LATERAL RESPONSE OF SINGLE PILES IN CEMENTED SAND AND WEAK ROCK

by

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DECLARATION

I hereby declare that this thesis is a presentation of my original research work. To the best of my knowledge it contains no materials previously published or written by another person, or substantial proportions of material which have been accepted for any other degree of qualification at the University of Western Australia, or any other university.

The research topic was raised by the candidature Fengju Guo and the coordinating supervisor Professor Barry Lehane. All the experimental work of the field cone penetration tests (CPTs), the field lateral load test, miniature CPTs, centrifuge model piles test, laboratory geotechnical testing, data processing and analysis of the results were developed and performed by the candidature. All the theoretical work of CPT calculation, simplified lateral capacity calculation and finite element modelling were performed by the candidature. Professor Barry Lehane provided full supervision for all the work of this thesis.

Signature:

Fengju Guo  Professor Barry Lehane
(Candidature)  (Supervisor)
ABSTRACT

Competent deposits of cemented sands distribute extensively along the coastline areas of Western Australia and many other coastline areas worldwide. These soils do not conveniently fall into any specific soil types of sand or clay and there are not yet generally accepted procedures for the design and analysis of piled foundations where lateral loads are comprised predominantly in these soils. In-situ cone penetration test (CPT) provides continuous profile data of the soils in question. Utilizing the cone data for design parameters for the lateral piled foundations has increasingly attracted interests.

This thesis investigates the lateral response of single piles in cemented sand and weak rock. The geotechnical behavior of the soils across a wide range from very weakly-cemented sands to well-cemented sands has been studied through a series of laboratory testing programs and in-situ testing programs. Full-scale lateral load experiments were performed with fully instrumented drilled and grout piles in a weak calcareous sandstone deposit at Pinjar, North West Perth in Western Australia. Centrifuge-scale load tests were also carried out on model piles preinstalled in very weakly-cemented sands artificially prepared with varying cement contents.

The results of the field test were used to develop the load transfer $P$-$y$ relationships as well as the limiting transverse pressures $P_u$ on piles for the well-cemented soils. The results of centrifuge model test in conjunction with finite element modelling were used to develop the lateral pressure-displacement $P$-$y$ relationships and the limiting transverse pressures $P_u$ for the very weakly-cemented soils. The field and centrifuge tests were modelled well using a hyperbolic $P$-$y$ relationship and the obtained ultimate lateral resistance. Existing methods to estimate $P_u$ and the formulations of $P$-$y$ curves were also evaluated and conclusive comments made to the comparisons.

The thesis then explores the potential of using CPT $q_c$ data directly for the design and analysis of laterally loaded piles. A simple CPT-based bi-linear $p$-$y$ approximation for the lateral response is proposed. This formulation is shown to provide good predictions for lateral pile response in a variety of cemented deposits.
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during my PhD journey.

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NOMENCLATURE

Latin symbols

\( a_0 \)  Initial radius of cavity
\( a \)  Current radius of cavity
\( a_i \)  Coefficients of curve fitting of moment data
\( A \)  Empirical factor of adjustment for ultimate resistance
\( B \)  Empirical factor of adjustment for ultimate resistance
\( b \)  Pile width or diameter
\( c' \)  Effective cohesion intercept
\( c_u \)  Undrained shear strength
\( C_e \)  Coefficient of gradation
\( C_u \)  Coefficient of uniformity
\( c_v \)  Coefficient of consolidation
\( c_H, c_1^H, c_2^H, c_3^H \)  Correlation parameters of CPT \( q_c \)-based sand
\( C1, C2, C3 \)  Coefficient of ultimate lateral resistance of API sand
\( D \)  Pile diameter
\( D_{50} \)  Mean effective particle size
\( D_{cone} \)  Diameter of cone penetrometer
\( e \)  Load eccentricity
\( f_s \)  Sleeve friction resistance of CPT
\( E_m \)  Operational modulus of soil or rock mass
\( e_{\text{max}} \)  Maximum void ratio
\( e_{\text{min}} \)  Minimum void ratio
\( E_p \)  Pile elasticity
\( E_{py} \)  Stiffness of the soil p-y spring
\( E_s \)  Operational modulus of soil
\( G \)  Shear modulus of soil or rock mass
\( G_0 \)  Small-strain shear modulus of soil or rock mass
\( H \)  Lateral load applied on pile
\( H_0 \)  Shear load at ground-line
\( H_u \)  Ultimate lateral load of pile
\( I_p \) Pile second moment of inertial of cross-area
\( I_{s(50)} \) Index of point load on chunk sample
\( K \) Curvature of pile
\( k_h \) Modulus of the subgrade reaction
\( K_0 \) Earth pressure coefficient
\( K_q \) Passive resistance coefficient for the cohesive component
\( K_c \) Passive resistance coefficient for the frictional component
\( K_R \) Pile flexibility factor
\( L_c \) Effective length of pile
\( L_e \) Embedment depth of pile
\( M \) Pile bending moment;
\( M_{\text{max}} \) Maximum bending moment
\( M_o \) Moment at ground-line
\( M_p \) Plastic moment
\( M_y \) Elastic yield moment
\( m \) Dimensionless material constant
\( n \) Dimensionless material constant
\( P \) Lateral transverse pressure
\( p \) Lateral resistance of soil and rock
\( p_a \) Reference stress that equals to one atmosphere
\( P_{\text{lim}} \) Limiting pressure of cavity expansion
\( P_u \) Ultimate transverse pressure of soil
\( P_{uc} \) Ultimate resistance by cohesive component
\( P_{uc\phi} \) Ultimate resistance of c-\( \phi \) soils
\( P_{ur} \) Ultimate transverse pressure of rock
\( P_{up\phi} \) Ultimate resistance by frictional component
\( q \) Deviator stress of triaxial test
\( q_c \) Cone tip resistance of CPT
\( q'_c \) Gradient of cone resistance, CPT
\( q_{ucs} \) Unconfined compressive strength
\( R_f \) Ratio of friction resistance to sleeve resistance in CPT
\( R_{\text{inter}} \) Strength reduction factor of interface element in Plaxis
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
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<tbody>
<tr>
<td>$SF$</td>
<td>Shear force in pile</td>
</tr>
<tr>
<td>$t$</td>
<td>Wall thickness of steel pipe</td>
</tr>
<tr>
<td>$V_s$</td>
<td>Shear wave velocity</td>
</tr>
<tr>
<td>$y$</td>
<td>Pile deflection</td>
</tr>
<tr>
<td>$y_0$</td>
<td>Pile deflection at groundline</td>
</tr>
<tr>
<td>$y_{50}, y_c$</td>
<td>Displacement at one-half the ultimate resistance of soil</td>
</tr>
<tr>
<td>$y_h$</td>
<td>Pile deflection at pile head where load applied</td>
</tr>
<tr>
<td>$y_{rm}$</td>
<td>Displacement at one-half the ultimate resistance of rock</td>
</tr>
<tr>
<td>$z$</td>
<td>Depth</td>
</tr>
<tr>
<td>$z_r$</td>
<td>Depth of pile rotation centre</td>
</tr>
<tr>
<td>$\Delta t$</td>
<td>Travel time</td>
</tr>
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**Greek symbols**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
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<tbody>
<tr>
<td>$\alpha$</td>
<td>Material constant in cavity expansion analysis</td>
</tr>
<tr>
<td>$\alpha_r$</td>
<td>Strength reduction factor of rock</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Material constant in cavity expansion analysis</td>
</tr>
<tr>
<td>$\delta$</td>
<td>Material constant in cavity expansion analysis</td>
</tr>
<tr>
<td>$\varepsilon$</td>
<td>Strain</td>
</tr>
<tr>
<td>$\varepsilon_{50}$</td>
<td>Strain at one-half the peak stress</td>
</tr>
<tr>
<td>$\varepsilon_a$</td>
<td>Axial strain</td>
</tr>
<tr>
<td>$\varepsilon_r$</td>
<td>Radial strain</td>
</tr>
<tr>
<td>$\varepsilon_v$</td>
<td>Volume strain</td>
</tr>
<tr>
<td>$\tau$</td>
<td>Shear stress</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson' ratio</td>
</tr>
<tr>
<td>$\xi$</td>
<td>Material constant in cavity expansion analysis</td>
</tr>
<tr>
<td>$\gamma'$</td>
<td>Specific unit weight of the soil or rock</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Density</td>
</tr>
<tr>
<td>$\sigma_1'$</td>
<td>Major principal stress</td>
</tr>
<tr>
<td>$\sigma_3'$</td>
<td>Minor principal stress</td>
</tr>
<tr>
<td>$\sigma_h'$</td>
<td>Horizontal effective stress</td>
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</table>
$\sigma'_v$ Vertical effective stress

$\phi'$ Effective friction stress or angle

$\phi_{\text{crit}}$ Effective friction angle at critical state

$\psi$ Dilatancy angle of soil

Abbreviations and acronyms

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tbody>
<tr>
<td>API</td>
<td>American Petroleum Institute</td>
</tr>
<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
</tr>
<tr>
<td>CEM</td>
<td>Methodology of Cavity Expansion Theory</td>
</tr>
<tr>
<td>CFA</td>
<td>Continuous Flight Auger</td>
</tr>
<tr>
<td>CHS</td>
<td>Circular Hollow Section</td>
</tr>
<tr>
<td>CPT</td>
<td>Cone Penetration Test</td>
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<tr>
<td>D&amp;G</td>
<td>Drilled and Grouted piles</td>
</tr>
<tr>
<td>DAQ</td>
<td>Data Acquisition System</td>
</tr>
<tr>
<td>DMT</td>
<td>Dilatometer Test</td>
</tr>
<tr>
<td>FEM</td>
<td>Finite Element Modeling</td>
</tr>
<tr>
<td>IGM</td>
<td>Intermediate GeoMaterial</td>
</tr>
<tr>
<td>ISRM</td>
<td>International Society for Rock Mechanics</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear Variable Differential Transducers</td>
</tr>
<tr>
<td>NC</td>
<td>Normally Consolidated</td>
</tr>
<tr>
<td>OCR</td>
<td>Over Consolidation Ratio</td>
</tr>
<tr>
<td>PSD</td>
<td>Particle Size Distribution</td>
</tr>
<tr>
<td>SCPT</td>
<td>Seismic Cone Penetration Test</td>
</tr>
<tr>
<td>SG</td>
<td>Strain Gage</td>
</tr>
<tr>
<td>UCD</td>
<td>The University College of Dublin</td>
</tr>
<tr>
<td>UCS</td>
<td>Unconfined Compression Shear Test</td>
</tr>
<tr>
<td>UWA</td>
<td>The University of Western Australia</td>
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CHAPTER 1 INTRODUCTION

1.1 Introduction

The aim of this thesis is to investigate the behavior of laterally loaded single piles embedded in cemented sands and weak rocks. Two main geotechnical problems have been addressed:

1. The geotechnical behavior of weak cementation soils and in-situ well cementation soils and weak rocks typically along the coastline area of Western Australia; and
2. The lateral response of single piles in these cemented materials.

A series of centrifuge model pile tests was performed in very weakly-cemented sands prepared with various cement contents. An experimental field test was conducted on drilled and grouted (D&G) piles in well-cemented sands and weak rocks. The cemented materials in this study do not contain any significant defects or discontinuities that can remarkably influence the lateral behavior of the test piles. In-situ cone penetration tests (CPT) and laboratory tests were carried out to characterize the soils for the centrifuge model test and the field test. Based on these test results, empirical model of load transfer relationship is developed with emphasis placed on the relationship directly correlated with the cone resistance $q_c$ data. The simple CPT based load transfer approximation for the lateral response is examined and shown to provide sufficiently good predictions for laterally loaded piles in a variety of cemented deposits.

1.2 Background

Competent deposits of cemented sands distribute extensively along the coastline areas of Western Australia (WA) and many other coastline areas worldwide including marine and terrace deposits. These areas are typically recognized as locations with the most potential for developing fossil fuel resources, as well as for clean and renewable energy due to infinite wind power. It is necessary to establish piled foundations to support
structures in a variety of offshore and onshore cases. The piled foundations must be able to resist considerable lateral loads applied by the wind or wave forces according to the specific contexts. Importantly, significant variability in the density and degree of cementation in these deposits poses challenges for the efficient design of pile foundations.

A salient feature of natural cemented sands is the high variability of soil density, cement agent and the cementation degree, resulting in an exclusive twilight spectrum between soils and weak rocks. This wide span from very weak cementation to well cementation of individual soil grains represents a challenging area of foundation design, for instance, for the in-situ characterization and subsequently interpretation of engineering parameters. To understand the behavior of the cemented sands, artificial cemented sands are mostly used in laboratory tests in addition to using the natural deposits if samples available. Typical geotechnical behaviors of cemented soils have been widely studied (Clough et al. 1981; Consoli et al. 2007; Coop and Atkinson 1993; Huang and Airey 1993; Puppala et al. 1995; Rad and Clough 1982; Saxena and Lastrico 1978). Cementation between soil particles always significantly increases the cone penetration resistance whilst decreasing the compressibility (Beringen et al. 1982; King et al. 1980; Rad and Tumay 1986; Schmertmann 1978).

A nonlinear load transfer approach \( p-y \) method is most commonly used for the lateral loading of piles. Current design procedure for lateral loading covers standard \( p-y \) criteria for default soils of sand and clay (Matlock 1970; O'Neill and Murchison 1983). The soils of cemented sands, with cohesive and cohesionless components, do not conveniently fall into any specific soil types. The models for the analyses of lateral piles in cemented sands or \( c-\phi \) soils, are essentially the transformed models of sand or clay (Ismael 1990; Reese and Van Impe 2001). However, there has no a generally accepted approach for the analysis of lateral loading in \( c-\phi \) soils.

A weak rock \( p-y \) criterion in power law function is recommended in the interim for the basis of limited field test data (Reese 1997). Alternative \( p-y \) criteria employing a hyperbolic function has also been proposed for rock mass (Gabr et al. 2002; Liang et al. 2009). The Reese weak rock \( p-y \) criterion is currently widely adopted in some
geotechnical codes such as ALP (ALP 2010) and LPILE (LPILE 2000). However, there are presently very few case histories supporting the Reese weak rock formulation.

This thesis aims to further our understanding of the lateral soil-pile behavior in a range of variably cemented sands. The research uses extensively experimental testing programs as the basis for furthering this understanding with the aid of numerical modelling method to explore the potential of applying CPT-based approaches for the analysis of laterally loaded in these \(c=\phi\) soils.

1.3 Research overview

The work presented in this thesis is divided into nine chapters.

Chapter 2 presents a literature review on the previous work related to this research. Typical characteristics of cemented sands indicated by past studies are summarized. The methodology of cavity expansion theory is briefly reviewed to give the theoretical basis for the CPT data interpretation and potential correlations between cone resistance and lateral response. Multiple \(p-y\) models for the analysis of laterally loaded piles are also reviewed and summarized.

Chapter 3 presents the soil properties obtained from in-situ CPTs, miniature CPTs and laboratory tests for the centrifuge model piles test and field test. Additionally small strain shear modulus data are derived for the soils in the present study. Effective stress strength parameters are determined principally through laboratory tests. Characteristic stress-strain relationships for these materials are also revealed in laboratory tests. In-situ soil testing and laboratory testing complement each other for the characterization of the soils.

Chapter 4 describes the full-scale field load testing program and analyses the test result. Details of the piles are given, including pile geometry and instrumentation. The experimental set-up and testing program are described. Results of the lateral load-displacement responses, strain gauge data and inclinometer data are presented in this chapter. Load test data are interpreted and used to derive the load transfer \((P-y)\)
curves for the piles tested. The $P-y$ relationship for the well cementation soils of this study is experimentally derived.

Chapter 5 describes the centrifuge-scale model load testing program and analyses the test results. Soil preparation and model pile installation, as well as the experimental setup of centrifuge modelling are detailed; test results of the load-displacement responses are presented and followed by relevant interpretation on these results data.

Chapter 6 analyses the CPT data using the cavity expansion theory methodology and states the numerical modelling of Pinjar field test. Closed form solutions are first employed to determine the limiting pressure for cavity expansion. Numerical simulation of field load test is detailed. CPT back-analysed soil parameters are proven to support three-dimensional finite element modelling using a simple soil constitutive model.

Chapter 7 analyses the CPT data following the similar procedure to that used for the field test and elaborates on the numerical modelling of centrifuge model pile tests in prototype dimensions. CPT back-analysed soil parameters form the inputs to the finite element analysis to derive the characteristic load transfer $P-y$ relationship as well as the limit lateral pressure $P_u$ expressed for very weak cementation soils.

Chapter 8 combines and examines the derived load transfer relationships. Through verification against the present tests and selected case history, the back-calculated $P-y$ curves are examined to be capable of replicating the lateral responses with accuracy. Conventional standard $P-y$ models and existing CPT-related $P-y$ models are evaluated and relevant comparisons are discussed. In view of the challenges of determining the material constants for soils having various cementations, idealization is made to the back-calculated $P-y$ curves and a methodology for constructing $p-y$ curves for general $c-\phi$ soils is proposed. The proposed $P-y$ method directly incorporates the CPT $q_c$ data and is shown to give reasonable good predictions of lateral behaviour in a variety of cemented materials.

Chapter 9 concludes the thesis, summarizing the main research findings drawn from this thesis. Future research work is recommended within this field of research.
CHAPTER 2  LITERATURE REVIEW

2.1  Introduction

There has been much recent interest in the design of single piles supporting offshore wind turbines which frequently experience very high lateral loads (Figure 2-1a). In these applications, some are more capacity sensitive, whereas others are deformation sensitive. When designing piled foundations whereby lateral loads are predominant, two key criteria must be satisfied: first, an adequate load factor for safety against ultimate failure; and second, an acceptable deflection at working loads (Poulos and Davis 1980).

In this chapter, a review of previous studies of cemented sands is presented. Following this, the methodology of cavity expansion theory will be reviewed to present theoretical support for a relationship between lateral soil response and the CPT end resistance (Figure 2-1b).

A review of the ultimate lateral resistance of single piles concentrates on the ultimate lateral resistance of the soil, that is, the geotechnical capacity of the soil surrounding a pile. In many practical cases, the design of piles for lateral loading also depends upon satisfying a limiting lateral deflection requirement, which may result in the specification of allowable lateral loads. The earliest and simplest representation of the problem was the subgrade reaction method with the beam on an elastic foundation (Broms 1964; Broms 1964; Hetényi 1946; Matlock and Reese 1960; Winkler 1867). However, the response of real soil is far from elastic. Non-linear soil response is a key factor in the behaviour of laterally loaded piles as the strain levels involved are very high. A series of independent non-linear Winkler springs, known as the $p$-$y$ method, replaces the linear soil springs for a more accurate representation of the soil behavior (Matlock 1970; Reese et al. 1974; Reese et al. 1975; Reese and Welch 1975; Welch and Reese 1972). With the development and wide application of in-situ testing techniques, $p$-$y$ curves correlated with data obtained from in-situ tests have been presented for the design of laterally loaded piles (Briaud 1997; Briaud et al. 1983; Dyson and Randolph 2001; Novello 1999; Robertson et al. 1986; Robertson et al. 1989; Suryasentana and Lehane...
Another analysis method assumes that the soil is an ideal elastic continuum and implements a boundary element analysis to develop solutions to lateral responses (Banerjee and Davies 1978; Davies and Budhu 1986; Poulos 1971; Poulos and Davis 1980). Using the finite element analyses, simple algebraic expressions have been presented in response to flexible piles under lateral loading (Randolph 1981). The solutions presented by these analyses generally require soil elastic parameters that may be assigned judiciously and are also limited to simple cases of low working stresses. Application of this method in a real problem is considered less flexible than the Winkler \( p-y \) method. The inelastic behavior of pile cannot be properly incorporated into this method.

In this chapter, a brief overview of the subgrade reaction method will be presented. This will provide necessary context for the development of the \( p-y \) method based on cone resistance data, which is undertaken in the present study. Conventional standard \( p-y \)
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curves and the existing advancement of $p-y$ curves from in-situ test result data are reviewed and presented in this chapter.

2.2 Typical behavior of cemented sands and weak rock

In the present study, full-scale pile load tests were carried out in well cementation soils and weak rocks. Centrifuge-scale model piles tests were also performed in very weak cementation soils artificially prepared, including un-cemented sand. Some of the typical geotechnical behaviors of these cemented material identified from previous studies have been reviewed and are summarized.

2.2.1 Strength characteristics of cemented sands and weak rock

Marine and terrace deposits are located along the coastline of Australia, particularly Western Australia (WA), the Pacific coast of the United States (US), as well as other parts of the world such as Canada, Italy and Norway. Loess deposits also exist in arid and semiarid areas such as Middle-Western US and China. An important common characteristic of these cemented sands is that they are capable of resisting compression and shear forces similar to un-cemented sands. However, they can also withstand at least some tensile stress due to a measurable cohesion, i.e. the cementation or bond or interlocking effects (Collins and Sitar 2009). The common cementing agents are carbonates, calcites, quartz and clays.

‘Cemented sands’ describes a wide range of materials, from a soil-weak rock spectrum at the weaker end of cemented sands, to the stronger end of cemented sands, as depicted in Figure 2-2 [after Brown (1981)]. It can also denote the lithified forming types of weak rocks. Unconfined compressive shear strength ($q_{ucs}$) has been proposed as a measure of the extent of cementation in a deposit. At the weaker end of the spectrum, the deposits exhibiting $q_{ucs}$ values of less than 300 kPa are classified as weakly cemented, whereas very weakly cemented deposits are those with $q_{ucs}$ values of less than 100 kPa (Clough et al. 1981). Where the materials do not have a well-defined structure, strength models around a Mohr-Coulomb (MC) model are usually adopted, with an effective cohesion $c’$ or cementation and an effective friction angle $\phi’$. 
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The ISRM classification scheme for intact materials (Brown 1981) is illustrated in Figure 2-2; while IAEG (1979) limit their ‘weak rock’ category to materials possessing unconfined compressive shear strengths \( q_{ucs} \) of 15 MPa. Consistent with the reported definitions for similar weak carbonate rocks (Abbs 1983; Abbs and Needham 1985; Erbrich et al. 2011; Erbrich 2004), weak rocks herein are defined as materials at the soil and rock boundary possessing \( q_{ucs} \) values in the range 0.5 MPa ≤ \( q_{ucs} \) ≤ 5 MPa. These values are similar to those of the intermediate geomaterials (IGM) (O’Neill et al. 1996).

Compared to the weakly cemented sands, weak rocks are harder, more brittle and heterogeneous.

One of the distinguishing features of naturally cemented sands is that they have variable densities and degrees of cementation. This variability and the challenge of recovering a sample without disturbing its cementation make it difficult to study the fundamental behavior of naturally cemented material. Understanding the behavior of cemented sands has developed primarily through the use of artificial sands within a laboratory environment using agents such Portland cement (Clough et al. 1981; Consoli et al. 2007; Puppala et al. 1995; Rad and Clough 1982), gypsum (Coop and Atkinson 1993; Huang and Airey 1998; Lee et al. 2010), or calcite (Ismail et al. 2000). The artificially cemented sands allow for an evaluation of the effects of the amount of cementing agent.

Figure 2-2 The cemented sands and weak rocks in the continuous geotechnical soil-rock spectrum [after Brown (1981)]
and of sand density on the soil response.

Saxena and Lastrico (1978) studied the behavior of a lightly-cemented sand under static loading and found that the soil strength from the cohesion component was predominant at a low strain level of below 1% whereas the strength from the friction component governed the soil behavior at high strain levels. Other researchers (Coop and Atkinson 1993; Huang and Airey 1998) observed similar behavior in cemented soils. Clough et al. (1981) investigated the behavior of cemented sands through extensive laboratory tests which were performed on four samples of naturally occurring cemented sands and artificially fabricated cemented sands. The test results for naturally cemented soils suggested that stiffness and peak strength increase with an increase in confining stress. The strongly cemented soil showed the brittle failure behavior at all confinements, whilst the moderate and weakly cemented soils showed a transitional response from brittle failure to ductile failure as confining stresses increased. This is depicted in Figure 2-3. Increasing the cement content leads to an increase in soil strength and stiffness, whilst decreasing the strain at mobilized peak strength.

For the key parameters controlling stiffness and strength of artificially cemented soils, Consoli et al. (2007) were the first to establish a unique dosage methodology based on a rational criteria of porosity/cement volume ratio. The influence of the cement content on the initial stiffness and strength of artificially cemented sands can be easily quantified as a function of this porosity/cement volume ratio (Consoli et al. 2012; Consoli et al. 2009). The MC failure envelopes can be assessed as a function of the porosity/cement volume ratio of cemented sandy soils, including natural cemented soils, through determination of splitting tensile strengths and unconfined compressive strengths (Consoli 2014).
Figure 2-3 Typical stress-strain behaviour of cemented sands (after Clough et al.1981): (a) natural weakly-cemented; (b) natural strongly-cemented; (c) artificially-cemented, 2% cement; and (d) artificially-cement, 4% cement.
2.2.2 Cone penetration resistance in cemented sands

Rad and Tumay (1986) were among the first to research the effect of cement on the penetration resistance of sand using a model study. It was found that cementation has a pronounced effect on cone resistance and sleeve friction, whilst decreasing the friction ratio. According to Puppala et al. (1995), the friction ratio is found to be indifferent to an increase in cementation at very weak cementation levels ($q_{ucs}$ values of less than 60 kPa), using a constant vertical stress. The tip resistance and the sleeve friction resistance are both found to increase with cementation, but at different magnitudes. Lee et al. (2010) found that the deformation modulus is dependant upon the degree of cementation, in addition to the effects of cementation on cone tip resistance.

![Figure 2-4](image)

Figure 2-4 (a) Effect of cementation on cone resistance (Rad and Tumay 1986); (b) Effect of cementation on cone resistance and friction resistance (Puppala et al. 1995)

Randolph et al. (2000) highlighted that at low stress levels, the cone resistance may be expected to be proportional to the degree of cementation; as the effective stress level increases, the level of cementation will have less influence. At high levels of cementation, that is, in the case of a more brittle rock-like material, the cone resistance no longer increases proportionally with the unconfined compression strength as the mechanism of failure changes.
2.3 Methodology of cavity expansion theory

The cavity expansion theory is reviewed here for two general purposes: first, cone resistance relies on the radial stress (i.e. essentially the horizontal stresses) to expand the cavity, and second, the cavity expansion theory provides a simple, yet reasonably accurate approach for the analysis of the CPT data.

2.3.1 The theory of cavity expansion

The theory of cavity expansion in finite and infinite media has been proven useful and has been widely applied in geotechnical engineering practice, such as in the area of interpretation of in-situ tests and the prediction of the behavior of piles. The cavity expansion theory was first applied in the case of metal indentation problems (Bishop et al. 1945). The application of the theory in geotechnical problems developed later. Through using increasingly realistic soil stress-strain models, significant progress has been made since then in developing accurate cavity expansion solutions for soils (Carter et al. 1986; Collins and Yu 1996; Salgado 1993; Salgado et al. 1997; Salgado and Randolph 2001; Vesic 1972; Yu and Houlsby 1991).

The theory of cavity expansion deals with the expansion of a cavity with given properties. Processes follow two basic types of expansion: expansion from a finite radius and expansion from an initial zero radius. In both cases, the pressure inside the cavity increases as the cavity radius increases, the soil deforms purely elastic at first and begins yielding at the cavity wall as the pressure further increases, as illustrated in Figure 2-5a. The cavity pressure does not increase infinitely when a cavity is expanded in a plastically deforming material—it will approach a limiting value, referred to as limiting pressure $p_{lim}$, as depicted in Figure 2-5b. The definition of limit pressure was first proposed as a closed form solution for the expansion of a spherical cavity in an elastic-plastic material with an associated flow rule ($\phi = \psi$) (Chadwick 1959), and later adapted for a material with a non-associated flow rule ($\psi < \phi$) (Yu and Houlsby 1991). The limiting pressure is also the steady-state pressure in the cavity when the ratio of the current cavity radius to the initial radius ($a/a_0$) approaches zero. An analysis of a cavity expansion problem yields the limiting pressure $p_{lim}$, and the cavity pressure-expansion
(pressure-strain) relationship during expansion, \( p-a \) curves.

![Diagram of cavity expansion problem](image)

Figure 2-5  (a) Cavity expansion problem (Carter et al. 1986); (b) cavity expansion-pressure curve

Based on a series of reviews and evaluations of the available theories for cone penetration analysis, Yu and Mitchell (1998) stated that cavity expansion theory is simple and yet reasonably accurate for the analysis of CPTs. The limiting pressure, \( P_{\text{lim}} \), in spherical cavity expansion is often applied to estimate the tip resistance, \( q_c \), in the cone penetrometer test.

2.3.2 Steps to determine cone tip resistance

To predict cone penetration resistance using cavity expansion theory, two steps need to be followed: first, determine the limiting pressure \( P_{\text{lim}} \) for cavity expansion in soil; and second, relate the cavity expansion limit pressure to the cone resistance \( q_c \).

Step 1 Determination of limit pressure

To estimate the cone tip resistance, the limiting pressure for the expansion of a spherical cavity should first be determined. Computation of limit pressure can be achieved through using either a numerical program of finite element analyses or an analytical approach. In the former approach, proprietary programs of finite element formulations
analysing steady-state cone penetration include CONPOINT (Salgado et al. 1997; Salgado and Randolph 2001), and the novel finite element code (Yu 2000). Alternatively, with the finite element codes PLAXIS and ABAQUS, cone penetration can be simulated assuming a spherical cavity expansion analogue (Suryasentana and Lehane 2014; Tolooiyan and Gavin 2011; Xu and Lehane 2008). In the latter approach, closed form solutions for the analytical analysis of spherical cavity expansion developed by (Carter et al. 1986) and Yu and Houlsby (1991) can also be used to evaluate limit pressure. Both solutions yield similar values.

Analytical solutions to closed form equations considered a simple approach to predicting limit pressure for its efficiency and accuracy in analyzing cone penetration test. Yu and Houlsby (1991) defined that a steady-state deformation mode is reached at very large deformations putting \( \frac{a}{a_0} \to \infty \) in the pressure-expansion relationship. The limiting value of the cavity pressure \( p_{\text{lim}} \) can be obtained analytically by finding the cavity pressure ratio \( R_\infty \) of an infinite series. The solutions are based on the non-associated MC failure criterion and take into account the dilatant behavior of the cohesive frictional soils.

**Step 2 Determination of cone tip resistance**

Randolph et al. (1994) used the analogy of spherical cavity expansion and bearing failure to analyze the end-bearing capacity of driven piles in sand, as depicted in Figure 2-6a and Figure 2-6b. Assuming that the stress of a rigid soil cone beneath the pile tip is equal to the limit pressure to expand the spherical cavity, a relationship between the drained end-bearing pressure and the limit pressure was found:

\[
q_b = p_{\text{lim}} (1 + \tan \phi' \tan \alpha) \quad (2.1)
\]

where \( \alpha \) is the angle of the rigid soil cone and equals to 45+\( \phi/2 \) (degree). This correlation can also be used for evaluating of the cone resistance \( q_c \) by taking \( \alpha \) as 60° (angle of a standard cone). This correlation has been widely applied in cone resistance \( q_c \) estimation (Suryasentana and Lehane 2014; Tolooiyan and Gavin 2011; Xu and Lehane 2008).
2.4 Ultimate lateral resistance of piles

Many researchers have presented estimations of the ultimate lateral resistance for cohesive soils (Broms 1964; Dunnavant and O'Neill 1989; McClelland and Focht 1956; Murff and Hamilton 1993; Randolph and Houlby 1984; Reese 1958; Reese et al. 1975; Welch and Reese 1972), and cohesionless soils (Broms 1964; O'Neill and Murchison 1983; Reese et al. 1974). In the general case of $c$-$\phi$ soils, derivations of the ultimate lateral soil resistance have been given (Brinch Hansen 1961; Evans and Duncan 1982; Reese and Van Impe 2001). Estimations of soil resistance using cone resistance in sands have also been proposed (Lee et al. 2010).

2.4.1 Lateral resistance of cohesive soils

In a discussion of the paper presented by McClelland and Focht (1956), Reese (1958) proposed two models for determining the soil lateral resistance against the pile below and near ground surface. One model assumes that a pile moves through saturated clay at some distance below the ground surface so that only a horizontal flow of the soil occurs at failure. Figure 2-7 depicts a vertical section though the pile and stress analysis of the
soil and pile assuming soil blocks are displaced horizontally when the pile is deflected. The ultimate field soil pressure \( (P_u) \) is analytically determined as \( 12c_u \), where \( c_u \) is the soil strength. The other model assumes that near the ground surface, a \( 45^\circ \) wedge of soil in front of the pile is passively moved up and out by the pile due to the lack of vertical confinement. Here, \( P_u \) would equal to \( 2c_u \). A limiting lateral pressure of \( 9c_u \) below ground surface has frequently been published for cohesive soils (Dunnavant and O'Neill 1989; Matlock 1970; Reese and Welch 1975; Welch and Reese 1972). The values were largely empirical and no theoretical justification was attempted.

Based on classical plasticity theory, Randolph and Houlsby (1984) presented precise solutions for the limiting lateral resistance of a circular pile loaded laterally in cohesive soil, using two approaches: the lower bound (LB) solution and the upper bound (UB) solution. The soil is modelled as a rigid, perfectly plastic cohesive material so that the calculation of the limiting resistance is reduced to a plane strain problem.

Figure 2-8 shows the characteristic mesh for soil flow around a perfectly smooth and a perfectly rough pile. These values are comparable to estimates by Poulos and Davis (1980) and Murff and Hamilton (1993).
Figure 2-8 Example of characteristic mesh for soil lateral flow: (a) $\alpha = 0.0$ for smooth pile, and (b) $\alpha = 1.0$ for fully rough pile (Randolph and Houlsby 1984), showing the higher resistance on the rough pile is due to the significantly larger deforming region; pile movement is in $y$-direction.

Further, based on observing the mechanism of cavity expansion in front of the pile, Randolph and Houlsby (1984) made a simple approximate calculation of the ultimate force per unit length. When a pile is laterally loaded, the pressure in front of the pile may increase from the in-situ horizontal stress level $\sigma_{h0}$ up to the limit pressure $P_{\text{lim}}$ obtained from a pressuremeter. The ultimate force per unit length is estimated to be between $(p_{\text{lim}} - u_0 + c_u)D$ and $(p_{\text{lim}} + p_a + c_u)D$. The limit pressure may be obtained from analyses of the pressuremeter test, expressed as $P_{\text{lim}} = \sigma_{h0} + c_u[\ln(G/c_u) + 1]$, in which $G$ is the shear modulus of the soil. For typical values of $G/c_u$, limit pressure $P_{\text{lim}} = \sigma_{h0} + 6c_u$.

The non-dimensional ultimate lateral resistance may be written as:

$$\frac{\sigma_{h0}}{c_u} + 7 < \frac{p}{c_u D} < \frac{\sigma_{h0} + p_a}{c_u} + 7 \quad (2.2)$$

The ratio of $\sigma_{h0}/c_u$ will typically be around 2 for normally or lightly consolidated clay (NC), and considerably lower for stiff, over-consolidated clay at shallow depths, which may be as low as 0.5. Thus, the ultimate resistance calculated from the cavity expansion analogue is comparable with or marginally deviates from that estimated by plasticity theory.
2.4.2 Lateral resistance of cohesionless soils

In predicting the behavior of sand around a laterally loaded pile, Reese et al. (1974) defined a limiting value of soil resistance for sand. The failure pattern of cohesionless soil was also defined through two cases: a soil wedge failure theory near the ground surface and a lateral flow failure model at depth, as illustrated in Figure 2-9a and Figure 2-9b respectively. MC failure theory was applied to the two models, as demonstrated in Figure 2-9c.

![Diagram of soil failure patterns](image)

Figure 2-9 Sand failure pattern in problem of laterally loaded pile: (a) assumed passive soil-wedge failure near ground surface, (b) assumed soil failure of lateral flow around the pile at depth, (c) Mohr-Coulomb diagram of pile in sand (Reese et al. 1974)

Bogard and Matlock (1980) realized that some terms in the formulation of \( p_u \) suggested by Reese et al. (1974) can be grouped into constants with little error, while others are less critical, they were able to simplify the equation. O'Neill and Murchison (1983) studied the lateral capacity of sand and evaluated the modified formulation of ultimate lateral resistance. This modified formulation was adopted as the guidelines of standard API sand (API RP2A-WSD 1987), and used by many geotechnical design offices. The ultimate lateral resistance varies from a value at shallow depths determined by Equation (2.3) to a value at deep depths determined by Equation (2.4). At a given depth \( z \), the
equation giving the smallest value of \( p_u \) should be used as the ultimate lateral resistance.

\[
p_{\text{us}} = (C_1 z + C_2 D)\gamma' z \quad (2.3)
\]
\[
p_{\text{ud}} = C_3 D\gamma' z \quad (2.4)
\]

where \( \gamma' \) is the effective soil weight; \( z \) is the depth; \( D \) is the pile diameter; and \( C_1, C_2, \) and \( C_3 \) are the coefficients as function of friction angle \( \phi' \).

2.4.3 Ultimate resistance of \( c-\phi \) soils

In the more general case of a \( c-\phi \) soils, deriving the ultimate lateral soil resistance based essentially on earth pressure theory has been given by Brinch Hansen (1961), who consider variation of resistance according to depth along the pile. On the soil pressure, Brinch Hansen (1961) take into account both the active and passive earth pressures. With the earth pressure coefficients known, the resultant pressure (i.e. passive minus active) at an arbitrary depth is given by

\[
p_u(z) = K_q(z)\sigma'_{v}(z) + K_c(z)c \quad (2.5)
\]

where \( K_q \) is the passive resistance coefficient for the frictional component of the soil and \( K_c \) is the passive resistance coefficient for the cohesive component of the soil, and \( \sigma'_{v} \) is the vertical effective stress at the depth under consideration.

Based on earth pressure theory, Evans and Duncan (1982) suggested an approximation of the ultimate soil reaction along a pile. Using the concept proposed by Evans and Duncan (1982), Reese and Van Impe (2001) recommended an estimation of ultimate soil reaction of a \( c-\phi \) soil. It is a summation of the ultimate resistances by the cementation/cohesive component \( (p_{uc}) \) and frictional component \( (p_{uf}) \), which is multiplied by an empirical adjustment factor \( (A) \).

\[
p_u = A p_{uf} + p_{uc} \quad (2.6)
\]

The ultimate soil resistance is taken as simply the passive soil resistance \( (\sigma_h) \) acting on
the face of the pile, in a horizontal direction of movement. The sliding resistance on the 
side of the piles and the active earth pressure will generally be small compared to the 
passive resistance and were considered to cancel each other out.

By comparison, the coefficients in Brinch Hansen (1961) are far larger than those of 
Reese and Van Impe (2001). The difference between the two estimated ultimate 
resistances becomes greater as the cementation degree (c’ value) increases.

2.4.4 CPT-\(q_c\) based resistance of sand

Cone resistance (\(q_c\)) and the lateral pile capacity (\(p_u\)) are commonly affected by the 
horizontal effective stress. Therefore, a certain correlation between the two is believed 
to exist. Lee et al. (2010) proposed a CPT \(q_c\)-based methodology for estimating lateral 
soil pressure \(P_u\) on rigid piles in sands, the \(P_u\) is given as \(p_u/D\). The lateral pressure \(P\) 
developed along pile embedment is typically given as a certain portion of limit lateral 
resistance \(P_u\). To investigate the correlation between \(q_c\) and \(P\), various sets of soils and 
stress conditions were prepared to calculate their values. The soil was assumed as 
Ottawa san, designed for various stress conditions and 60 drained cases were considered. 
The \(q_c\) values were estimated using the program Conpoint (Salgado and Randolph 2001). 
For each soil condition considered, the \(P_u\) values were obtained using three existing 
methods proposed by Broms (1964), Petrasovits and Awad (1972) and Prasad and Chari 
(1999). The \(q_c-P_u\) correlation was obtained

\[
P_u = c_H \frac{q_c}{\sigma_{mc}} p_a^{c_H}
\]

(2.7)

where \(p_a\) is the reference pressure and equals to one atmosphere pressure (100 kPa =0.1 
MPa); and \(c_H\), \(c^{1}_H\), \(c^{2}_H\) and \(c^{3}_H\) are the correlation parameters. Three sets of 
dimensionless factors were derived for \(q_c-P_u\) correlations.

2.5 \(p-y\) method of analysis of laterally loaded piles

Another important component for laterally loaded piles is the serviceability limit state
design. Two schools have predicted lateral deflection through two approaches: one, the subgrade reaction approach, and second, the elastic continuum approach. A brief description of these two schools has been provided in the section of introduction in this chapter. The following section will present an overview of the subgrade reaction method to provide some necessary background for the development of empirical $p$-$y$ relations.

2.5.1 $p$-$y$ method

The $p$-$y$ method is essentially a modified subgrade reaction method, replacing linear Winkler springs with non-linear Winkler springs to account for the non-linear response of real soil. The subgrade reaction method assumes an elastic beam on an elastic foundation, developed from the Winkler assumption (Winkler 1867). The soil reaction $p$ (force per unit length of pile) at any point along the pile is assumed to be proportional to the deflection $y$ at the same point, $p = (k_h D) y$, in which $D$ is the unit diameter. The proportionality $k_h$ (kN/m$^3$) is called the modulus of the subgrade reaction. The concept of the $p$-$y$ method in the form of a family of $p$-$y$ curves for various depths, was first suggested by McClelland and Focht (1956). The $p$-$y$ curves have been developed on the back-analysis of the full-scale lateral pile load test. The soil reaction $p$ on the pile (from the theory of beams) was modified by writing the following equation for the unit soil reaction $p$

$$p = E_{py} y$$

(2.8)

$E_{py}$ was termed the soil modulus of the pile reaction and was by definition the ratio of pile reaction at any point to the pile deflection at that point (Reese et al. 1974). The initial slope $E_{py, i}$ of a $p$-$y$ curve influences analyses only in the case of very small loads; in more normal cases, a secant modulus $E_{py, s}$ controls the analyses. The term does not uniquely represent a soil property, but simply a parameter for convenient use in computations. Soil resistance attains a limiting value defined as the ultimate soil resistance $p_u$ (force per unit length).

The fourth order differential equation governing the beam behavior can then also be modified for a laterally loaded long flexible pile, written as
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\[ E_p I_p \frac{d^4 y}{dz^4} + E_p y = 0 \quad (2.9) \]

where \( E_p \) is the modulus of elasticity of the pile, \( I_p \) is the second moment of inertia of the pile, and \( z \) is the depth below ground surface. McClelland and Focht (1956) linked the ‘horizontal’ soil modulus \( (E_s=k_hD) \) directly to the shear moduli of the triaxial cell. There have been various models estimating the modulus of subgrade reaction of the \( p-y \) curve from the soil modulus of elasticity \( E_s \) [e.g. Vesic (1961), Broms (1964), Randolph and Gourvenec (2011), Poulos (1971), Liang et al. (2009)].

Figure 2-10 represents a typical problem and analysis of laterally loaded piles. Assuming the basic information of soils can be mathematically expressed, the differential equation can be solved. These results would be satisfactory from the standpoint of structural analysis. The boundary conditions both at the top and the bottom of the pile are satisfied and the equations for the statics are also fulfilled. The successive curves are mathematically compatible; that is, starting from the deflection curve, it is possible to obtain the slope \( s \), the bending moment \( M \), the shear force \( SF \), and the soil reaction \( p \) (force per unit length) by successive differentiation; By starting from the soil reaction curve, the shear force, the moment, the slope and the deflection may also be obtained through successive integration. The results of a typical case are illustrated in Figure 2-11.
2.5.2 p-y curves

p-y curves can be defined graphically by a thin slice of soil-pile system, as illustrated in Figure 2-12, Figure 2-13 and Figure 2-14. It is assumed that the pile is vertically embedded without bending and the supporting soil is considered as a series of independent non-linear springs. Prior to loading, the soil pressure acting against the pile can be reasonably assumed to be uniform. If the pile is laterally loaded, the piled deflects along its length. With respect to the stresses and displacements of the soil and pile, equilibrium is established at all points along the pile. The soil behaves as a continuous medium as it is deformed and stressed. The resultant net soil reaction force at any given depth may be obtained by integrating the soil pressure around the pile. This process can be repeated in concept for a series of deflections, resulting in a series of reaction forces per unit length to form a p-y curve. In a similar manner, the set of p-y curves can be obtained for a range of depths.

Figure 2-13 shows a typical p-y curve (Reese et al. 1974). The initial slope, $E_{py-i}$, of the p-y curve influences analyses only for the very smallest loads; in most normal cases, a secant modulus, defined by $E_{py-s}$, controls the analyses. Figure 2-14 presents an idealized soil spring, bi-linear p-y curve for any depth, may be simplified to interpret the lateral pile response in practice (Schmertmann 1978).
Figure 2-12 Graphical definition of p-y concept (in sand): (a) a section of pile at depth below the ground surface, (b) soil and pile stress conditions at rest, and (c) soil and pile stress conditions after load applied (Reese et al. 1974)

Figure 2-13 Typical p-y curve (Reese et al. 1974)
The simplified bi-linear $p$-$y$ relation will make the calculation procedure more practical. Baguelin et al. (1977) presented a theoretical study of lateral reaction of piles assuming an elastic-plastic (EP) soil-pile system. Scott (1980) developed a simplified sand $p$-$y$ curve, consisting of two straight line segments and not applying ultimate soil resistance. Elastic-plastic $p$-$y$ curves for a variety of soils were also proposed by other researchers (Guo 2012; Schmertmann 1978; Scott 1981).

The non-linear EP $p$-$y$ curve is simply constituted by two main elements: the spring stiffness $E_{py}$ and the limit resistance $p_u$, as schematically depicted in Figure 2-14. For the general form of $p$-$y$ curve, a large number of parameters are required to be properly determined. In contrast, this simplified EP $p$-$y$ model is normally expeditious and sufficiently accurate (Poulos and Hull 1989).

The series of $p$-$y$ curves depend greatly upon soil types and are highly empirical. $p$-$y$ curves can be derived experimentally from full-scale loading tests conducted on instrumented piles for the type of soil in question. The procedures may follow the descriptions stated in section 2.5.1. Alternatively, if the successive slope curve of the pile section is measured with a tiltmeter or inclinometer, it is possible to obtain the diagrams of pile deflection, shear force, the bending moment and soil reaction. Table
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2.1 summarizes the variables used in the methodology of developing the \( p-y \) curves.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Formula</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embedment depth along the pile</td>
<td>( z )</td>
<td>([L])</td>
</tr>
<tr>
<td>Distance to the neutral axis within pile cross-section</td>
<td>( r )</td>
<td>([L])</td>
</tr>
<tr>
<td>Deflection</td>
<td>( y )</td>
<td>([L])</td>
</tr>
<tr>
<td>Slope or rotation of the pile section</td>
<td>( \phi = \frac{dy}{dz} )</td>
<td>dimensionless</td>
</tr>
<tr>
<td>Curvature of pile</td>
<td>( \kappa = \frac{d^2y}{dz^2} )</td>
<td>radians/L</td>
</tr>
<tr>
<td>Bending moment</td>
<td>( M = E_p I_{p} \kappa = E_p I_{p} \frac{d^2y}{dz^2} )</td>
<td>([FL^2])</td>
</tr>
<tr>
<td>Shear force</td>
<td>( SF = E_p I_{p} \frac{d^3y}{dz^3} )</td>
<td>([F])</td>
</tr>
<tr>
<td>Soil reaction (force per unit length)</td>
<td>( p = E_p I_{p} \frac{d^4y}{dz^4} )</td>
<td>([F/L])</td>
</tr>
</tbody>
</table>

2.6 Conventional p-y models

Pile design criteria for \( p-y \) curves of various soil types are based mostly upon back-computation from full-scale tests conducted several decades ago, supplemented with certain analytical investigations. With the development and wide application of versatile in-situ testing techniques, design criteria for laterally loaded piles using in-situ results data have been presented since the early 1980s. The invention of geotechnical centrifuge facilities has enabled small-scale model tests. The following sections present the existing conventional \( p-y \) curves for various soils and those from the in-situ test results.

Cemented soils do not fall easily into any specific category. They have been modelled as transformed clay or sand (Evans and Duncan 1982; Ismael 1990; Reese and Van Impe 2001). Some conventional \( p-y \) curves that may be considered for their comparison with the derived \( p-y \) curves have been reviewed. This section will summarize these models.
2.6.1 Stiff clay with no free water

Welch and Reese (1972) reported a detailed procedure for constructing $p$-$y$ curves for stiff clay with no free water. The $p$-$y$ curves of stiff clay with no free water were formulated through a field test and laboratory tests. The field experiment was conducted at a test site in Houston, Texas; the 0.76 m diameter drilled shaft was fully instrumented; the soil at the test site was over-consolidated clay with a well-developed secondary structure. Figure 2-15 depicts the normalized $p$-$y$ curve for stiff clay with no free water. The $p$-$y$ formulation is the same as that for API soft clay (Matlock 1970), but the exponent of the parabola is one-fourth, rather than one-third. The characteristic shape of stiff clay $p$-$y$ curves is thus stiffer than that of soft clay. Without a water table, no soil softening is observed on the characteristic shape of $p$-$y$ curves of stiff clay with free water. Computation of the ultimate soil resistance for stiff clay $p$-$y$ criteria is similar to that for API soft clay. The unit weight of soil should reflect the depth of the water table.

![Figure 2-15 Characteristic shape of p-y curve for stiff clay above water table (Welch and Reese 1972)](image-url)
Based on a statistical comparison of existing sand $p$-$y$ equations, O'Neill and Murchison (1983) proposed a simplified $p$-$y$ equation for laterally loaded piles in sand, which yielded relatively accurate results in comparison to the original $p$-$y$ curves. This modified $p$-$y$ criteria has been accepted as the guidelines for generating load transfer curves for sandy soils in the standard API RP 2A-WSD (API RP2A-WSD 1987). The lateral $p$-$y$ relationship may be approximated through a continuous hyperbolic tangent function and $p_u$ is defined previously (see section 2.4.2).

Figure 2-16 shows the characteristic normalized $p$-$y$ curve of API sand. The term $kz$ is the product of lateral subgrade modulus and depth used in sand $p$-$y$ model of Reese et al. (1974). This API sand model provides a limiting value of soil pressure and the $p$-$y$ curves are usually stiffer than those of Reese et al. sand model.

$$\frac{p}{Ap_u} = \tanh \left( \frac{kz}{Ap_u} y \right)$$

Figure 2-16 Characteristic shape of $p$-$y$ curve for API sand (O'Neill and Murchison 1983)
2.6.3 Cemented sand or c-φ soil

Ismael (1990) conducted a field experiment on bored piles in weakly cemented sands at a test site in Kuwait. The determined peak strength parameters $c'$ and $\phi'$ were typically 20 kPa and 35°. Evans and Duncan (1982) equation of $p_{u-c\phi}$ for c-φ soils was adopted to calculate the ultimate soil resistance $p_u$, and the cubic parabola function of API soft clay (Matlock 1970) employed to describe the slope of normalized $p-y$ relationship. Figure 2-17 illustrates the normalized $p-y$ curve for weakly cemented sand, incorporating a hyperbolic functional relation for API soft clay and an ultimate soil resistance for c-φ soil. The predicted load-displacement responses based on the $p-y$ curves of sand (Reese et al. 1974) were found to be much softer than the experimental response when ignoring the cohesion component.

![Figure 2-17 Characteristic shape of P-y curve for cemented sand (Ismael 1990)](image)

The procedure proposed by Ismael indirectly suggested that the cemented sand behaves more like cohesive soil than cohesionless soil. This is because it adopted the cubic parabola functional $p-y$ curve usually expressed for clay, reflecting cohesive
stress-strain behavior.

Ashford and Juinrarongrit (2005) conducted lateral load tests on full-scale large diameter Cast-In-Drilled-Hole piles (CIDH) at a site in San Diego, California. The test site consists of weakly cemented sands with cohesion levels ranging from approximately 15 kPa to 55 kPa, and a friction angle varying between 30° and 32°. Similarly, a simple parabola function was proposed for the $p-y$ relationships of cemented sands based on a back-calculation of the experimental results. The displacement in which the ultimate soil resistance is fully mobilized is estimated $3D/80$ using the suggestion of sand (Reese et al. 1974), rather than a multiple $y_c$ for clay.

In contrast to Ismael, Reese and Van Impe (2001) believed the stress-strain behaviour of $c-\phi$ soils may be closer to that of cohesionless soils than that of cohesive soils. To develop the $p-y$ curves of $c-\phi$ soils, the procedures described for sand (Reese et al. 1974) were recommended due to the stress strain of $c-\phi$ soils. Figure 2-18a displays the characteristic $p-y$ for general $c-\phi$ soils and sand in the case of zero cohesion. The $p-y$ curve of $c-\phi$ soils differs from that of un-cemented sand with a drop in resistance after peak resistance, as shown in Figure 2-18b. The $p-y$ curve of sand in $c-\phi$ soils comprises an initial linear section and a parabola that joins the linear section to an ultimate strength. Thus, the two-part procedure involving first, the determination of the $p_{uc,\phi}$ as a summation of $p_{uw}$ and $p_{uc}$ using an empirical factor and second, defining a sand-like $p-y$ relation, has been widely employed in commercial code LPILE for analyses of lateral pile.

It should be noted that the $p-y$ curves proposed by Reese and Van Impe (2001) were developed on the basis of a theoretical analysis and have not yet been validated by field test results. In a case history evaluation of laterally loaded piles, Anderson et al. (2003) stated that the silts, silty sands and clayey sands should use cohesive $p-y$ curves.
Figure 2-18 Characteristic shape of $p$-$y$ curve proposed for $c$-$\phi$ soils: (a) $c'=0$ kPa, sand (Reese et al. 1974), and (b) general $c$-$\phi$ soils (Reese and Van Impe 2001)
2.6.4 Weak rock

Reese (1997) extended the \( p-y \) method from the analysis of piles under lateral loading to analysis of single piles in rocks. Based on the results of load tests at two sites, the recommendations for computing \( p-y \) curves for rock were termed as interim due to the meagre amount of experimental data available. The concepts and procedures outlined in the recommendations similarly follow those for stiff clay above the water table, except that the ultimate resistance \( p_{ur} \) of rock, the slope \( K_{ir} \) (resistance \( p_i \) divided by \( y_i \)) of a straight line, and initial portion of the \( p-y \) curves are defined for weak rock.

\[
\frac{p}{p_{ur}} = 0.5 \left( \frac{y}{y_{rm}} \right)^{1/4}
\]

\[
p_{ur} = \alpha_r q_{ucs} (1 + 1.4z/D)D \quad 0 \leq z \leq 3D
\]

\[
p_{ur} = 5.2 \alpha_r q_{ucs} D \quad z \geq 3D
\]

\[
K_i = k_{ir}E_i
\]

\[
y_{rm} = k_{rm}D
\]

Figure 2-19 Characteristic shape of \( p-y \) curve proposed for weak rock (Reese 1997)

Figure 2-19 presents the normalized \( p-y \) curve for weak rock proposed by Reese (1997). The lateral soil resistance-deflection relationship for weak rock is represented by a three-segment curve: the initial linear portion of the curve at very small deflection and a straight line starting from the deflection where ultimate resistance is attained, and a transitional non-linear portion in the form of a further order parabola in between. The \( \alpha_r \) in Figure 2-19 is the strength reduction factor that may be assumed to be one-third for
RQD of 100 and to increase linearly to unity at RQD of zero; the $k_{rm}$ in is a constant ranging from 0.0005 to 0.00005 that serves to establish the overall stiffness of the $p$-$y$ curve; $E_i$ is the initial elastic modulus and $k_i$ is dimensionless constant of rock mass.

The $p$-$y$ criterion of Reese weak rock is based on a very limited number (two case studies) of full-scale field load tests. Recommendations for selecting values for the input parameters required are vague and unsubstantiated by broad experience (Turner 2006). The $p$-$y$ formulation is being used extensively despite these sources of uncertainty. Therefore, research should be undertaken with the objective of developing improved criteria for $p$-$y$ curves in weak rock. The $p$-$y$ curve parameters should be related to weak rock engineering parameters, which can be determined using the available site and material characterization methods.

2.6.5 Weathered rock and rock mass

Apart from the Reese’s weak rock $p$-$y$ criterion, other $p$-$y$ criteria for describing load transfer relationship for weathered rock and rock mass have also been developed (Gabr et al. 2002; Liang et al. 2009). The studies by Liang et al. (2009) and by Gabr et al. (2002) present recommendations for $p$-$y$ curves commonly based on a hyperbolic function, written as

$$p = \frac{y}{1 + \frac{y}{K_i p_{ur}}}$$  \hspace{1cm} (2.10)

in which $K_i$ is the initial subgrade modulus and $p_{ur}$ is the ultimate resistance; the two parameters correspond to the initial slope tangent slope and the asymptote of the hyperbola, as shown in Figure 2-20. Table 2-2 summarizes partly the variables for determining two parameters $K_i$ and $p_{ur}$ at deep depths. The $p_L$ in equations is the compressive strength of rock mass; the $m_b$, $s$, and $a$ are Hoek-Brown constants that depend on the characteristics of rock mass, $E_m$ is the modulus of rock mass and may be determined from the pressuremeter test data or dilatometer test data in two approaches.
Chapter 2 Literature review

Figure 2-20 Hyperbola p-y curve of weathered rock and rock mass (Liang et al. 2009; Gabr et al. 2002)

Table 2-2 Determination of two parameters characterizing a hyperbola of rock p-y curve

<table>
<thead>
<tr>
<th>Two parameters</th>
<th>Liang et al. (2009)</th>
<th>Gabr et al. (2002)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_i$</td>
<td>$K_i = E_\infty (D / D_{ref})e^{-2\pi \left( \frac{E_p I_p}{E_u D} \right)^{0.284}}$</td>
<td>$K_i = k_h D$</td>
</tr>
<tr>
<td>$p_{ur}$</td>
<td>$p_{ur} = \left( \frac{\pi}{4} p_L + \frac{2}{3} \tau_{max} - p_a \right) D$, Liang et al. (2009)</td>
<td>$p_{ur} = (p_L + \tau_{max}) D$, Gabr et al. (2002)</td>
</tr>
<tr>
<td>$p_L$</td>
<td>$p_L = \gamma' z + q_{acc}(m_h \frac{\gamma' z}{q_{acc}} + s)^a$</td>
<td>$p_L = \gamma' z + q_{acc}(m_h \frac{\gamma' z}{q_{acc}} + s)^a$</td>
</tr>
<tr>
<td>$\tau_{max}$</td>
<td>$\tau_{max} = 0.45 \sqrt{q_{acc}}$, $q_{acc}$(MPa)</td>
<td>$\tau_{max} = 0.2 \sqrt{q_{acc}}$, $q_{acc}$(MPa)</td>
</tr>
</tbody>
</table>

Liang et al. (2009) have presented two equations for evaluating $p_{ur}$ for rock mass. The first corresponds to a wedge failure mode that applies to rock mass near the ground surface, and the second applies to rock mass at depth and is given in Table 2-2; $p_{ur}$ takes the smaller of the two values obtained from the wedge analysis and at depth. The $p_{ur}$ at depth in Liang et al. (2009) approach is similar to that proposed by Gabr et al. (2002), but accounts for the active earth pressure acting on the pile. Both methods incorporate the Hoek-Brown strength criterion to evaluate the normal stress $p_L$ and both rely on correlations with GSI to determine the required Hoek-Brown strength parameters.
According to Liang et al. (2009), the Mohr-Coulomb strength parameters friction angle $\phi'$ and cohesion $c'$, which are required to evaluate the ultimate resistance of wedge failure, should be estimated from Hoek-Brown strength criterion.

By comparison, Liang et al. (2009) rock mass $p$-$y$ curves criterion gives stiffer initial response and relatively higher resistance capacity than those of Gabr et al. (2002).

Figure 2-21 compares the $p$-$y$ curves of weak rock, weathered rock and rock mass using the same rock properties. In contrast to the three-segment $p$-$y$ curve of Reese weak rock, the hyperbolic $p$-$y$ curves of Gabr et al. weathered rock and Liang et al. rock mass are much softer initial and inferior resistance capacity. The $p_{ur}$ value of Reese’ weak rock formulation depends on the $q_{ucs}$ value proportionally, whilst the $p_{ur}$ values in equations of Gabr et al. weathered rock and Liang et al. rock mass are a fraction or square-root the $q_{ucs}$ values.

Figure 2-21 Comparison of the $p$-$y$ curves of weak rock and rock mass

Use of the Reese weak rock model included within LPILE is supported by the observations for modeling short rock sockets in limestone from research by Parsons et al. (2010). The Reese’s weak rock model did predict somewhat less deformation than
was observed in working load. A relatively good fit was obtained between the weak rock model and the field test data with regard to the nominal resistance and ultimate pile head deformations, with the nominal model resistance being within about 10% of the observed ultimate load for both shafts.

2.7 CPT $q_c$-based p-y curves

The $p$-$y$ curves from in-situ tests have advanced the $p$-$y$ curves that conventionally employ the soil parameters obtained from routine laboratory testing for modelling the stress-strain relationship of soil. The CPT $q_c$-based $p$-$y$ curves utilize cone resistance directly to describe load transfer relationships, completely relying upon in-situ parameters. The $q_c$ is the most widely used measure of relative strength in both offshore and onshore investigations, and it is often used as a normalizing tool to relate soil properties across different sites over different depth ranges.

2.7.1 CPT $q_c$-based $p$-$y$ curves proposed by Novello (1999)

Based on the results of the centrifuge and model pile tests on un-cemented carbonate sand (Bass Strait sands), Wesselink et al. (1988) proposed a $p$-$y$ formulation for carbonate sand. The Wesselink $p$-$y$ curves have a parabolic shape where soil resistance $p$ is an explicit function of displacement as a fraction of the pile diameter. The Wesselink formulation captures non-linear stress-strain behavior in terms of effective stress and reflects variations in cone penetrometer resistance at a given site. Nonetheless, it is simply a best fit to the experimental data.

Considering the similarity of lateral pile loading to the cylindrical expansion of the surrounding soil mass, and using CPT $q_c$ as an expression of limiting plastic stress, Novello (1999) generalized this Wesselink-derived empirical relationship into a formulation directly linked with CPT-$q_c$, and effective mean stress $\sigma'_{v}$. This $p$-$y$ criterion takes the form expressed as

$$ p = (bD)\sigma'_{v} q^n_{c} \left( \frac{y}{D} \right)^{m} \quad < p_{\text{LIMIT}} \quad (2.11) $$
where \( b \) is a material constant; \( \sigma' \) is the overburden effective stress; \( n \) and \( m \) are exponents and \( p_{\text{LIMIT}} \) is the limiting lateral stress. For the carbonate sands of the Bass Strait (located in Victoria, Australia), the constants of \( b \), \( n \) and \( m \) from Equation (2.11) were a good fit at 2, 0.67 and 0.5 respectively. This \( p-y \) criterion was also made transferrable to other similar soil conditions at North Rankin, located on the North-West Shelf of Australia. To capture the variation in response from the individual pile tests, different \( m \) values of the \((y/D)\) exponent were trialed. Lower values of 0.25 and 0.33 were found and consistent with the clay \( p-y \) criteria from the studies by Reese and Welch (1975) and Matlock (1970). This suggested a possible cohesive response from the surficial muddy silt sand at North Rankin.

2.7.2 CPT \( q_c \)-based \( p-y \) curves proposed by Dyson and Randolph (2001)

Based on a series of centrifuge model pile tests, Dyson and Randolph (2001) proposed similar empirical \( p-y \) models for the calcareous sand and silt recovered from the seabed in the vicinity of the Goodwyn platform, on the North-West Shelf of Australia. The Wesselink’s \( p-y \) model was modified through use of a non-dimensional \( p-y \) expression, in which the depth term in Wesselink et al. (1988) was replaced by the quantity \( q_c \) normalized by \( \gamma'D \), i.e. a product of \( \gamma' \) and \( D \) in the \( p-y \) models. These modified \( p-y \) models are given by

\[
\frac{P}{\gamma'D} = R \left( \frac{q_c}{\gamma'D} \right)^n \left( \frac{\gamma}{D} \right)^m \tag{2.12}
\]

in which the force per unit length \( p \) has been replaced by the net pressure \( P \) (\( P = p/D \), in this case); \( R, n, \) and \( m \) are the dimensionless coefficients. The \( R, n, \) and \( m \) values are 2.84, 0.72, and 0.64 for the free-headed piles.

The above \( p-y \) models are derived from small-scale results, centrifuge tests on piles and numerical analyses, which relate the in-situ parameter CPT \( q_c \) to soil resistance for various calcareous sands. However, limiting soil resistance \( p_u \) is not explicitly given in these CPT \( q_c \)-based \( p-y \) models.
2.7.3 CPT $q_c$-based $p-y$ curves proposed by Suryasentana and Lehane (2014)

Recently, Suryasentana and Lehane (2014) numerically derived the CPT $q_c$-based power-law relationships specifically for sand, with consideration for modelling the imposed limit (ultimate) pressure. The unit force per length $p$ is normalised by a product of effective stress and pile diameter, and the $q_c$ is normalised by effective mean stress $\sigma_v'$ ($\gamma'z$). An exponent relationship, forming a superior format over the aforementioned formulations was introduced into the dimensionless $p-y$ model, determined as

$$\frac{p}{\gamma'zD} = 2.4\left(\frac{q_c}{\gamma'z}\right)^{0.67} \left(\frac{y}{D}\right)^{0.75} \left[1 - \exp\left(-6.2\left(\frac{z}{D}\right)^{-1.2}\left(\frac{y}{D}\right)^{0.89}\right)\right]$$

(2.13)

Equation (2.13) was validated against the measurements obtained in an independent field study in Hampton, Virginia. Agreement between the predictions and observed pile load-displacement at ground line was observed.

2.7.4 CPT $q_c$-based $P-y$ curves proposed by Phillips (2002)

In a field investigation of full-scale single piles subjected to a lateral load, or combined axial and lateral loads, Phillips (2002) derived best-fit $P-y$ relationships for the soft clay of Kinnegar sleech and the sandy fill. The $P-y$ relationship is written as

$$P = m q_c \left(\frac{y}{D}\right)^n$$

(2.14)

in which $m$ and $n$ are the dimensionless material constants with values of approximately 0.19 and 0.4 for Kinnegar sleech and about 0.44 and 0.50 for sandy fill. The lateral displacements predicted using back-fit $q_c$-based $P-y$ curves compared reasonably well with the measured displacements at the lower load (26 kN). However, a tendency for deviation at higher loads (60 kN) was observed for one of the piles. The predicted deflection for another pile under combined loads appeared conservative when compared with measured values. The predicted bending moment diagram indicated some deviations from those values measured for the laterally loaded pile, whereas they were...
relatively consistent when measured for combined loading.

2.8 Summary

From the literature review on cavity expansion theory’s methodology, ultimate lateral resistance of soil, \( p-y \) curves and typical behaviour of cemented sands and weak rocks, some concluding comments can be made:

- The cone resistance \( q_c \) (or \( p_{\text{limit}} \)) is affected by the soil state variables such as the effective horizontal stress \( \sigma' \) and soil density. The ultimate soil resistance is also given as a function of such state soil variables. From a theoretical perspective, it may be reasonable to expect certain correlations to exist between the ultimate resistance \( P_u \) of lateral pile and the tip resistance \( q_c \) of in-situ cone penetration.

- An estimate of ultimate lateral capacity using CPT \( q_c \) results have been proposed for cohesionless soils. However, this approach assumes various distributions of soil resistance and thus requires further validation. Estimation of lateral load capacity for piles in cohesive and frictional soils has not been generally accepted.

- CPT \( q_c \)-based models have shown promising potential and can be incorporated in a non-linear \( p-y \) formulation. The models are also straightforward for solving the geotechnical problems of lateral piles compared with conventional standard \( p-y \) models. However, most of them have not yet addressed the ultimate resistance in their equations.

- Significantly, there is still a considerable shortage of database for lateral soil strength and stiffness of cemented sands and \( c-\varphi \) soils.

- The variability and brittleness of \( c-\varphi \) soils cause difficulties in recovering samples for laboratory tests without disturbing cementation for laboratory tests. Such features suggest that an in-situ test based method is likely to be the most successful for those kinds of geomaterials.
CHAPTER 3 SOIL PROPERTIES

3.1 Introduction

This chapter describes the soil characterization, in-situ tests and laboratory tests that were used to evaluate two types of cemented soils: very weakly cemented sands (weak $c$-$\phi$ soils) and well-cemented sands (strong $c$-$\phi$ soils). The weak $c$-$\phi$ soils were artificially prepared for the centrifuge model pile investigations whilst the well-cemented sands were in-situ samples obtained from the Pinjar test site where the field experiments took place.

Soil characterization was undertaken to obtain mechanical data for the two types of $c$-$\phi$ soils and to determine their soil properties. The data were subsequently used to: (i) assist interpretation of the lateral load test results, and (ii) aid the development of a general model for predicting lateral pile response using in-situ test parameters.

CPTs are the most commonly accepted and used in-situ testing methods for determining geotechnical engineering properties. CPTs and SCPTs using standard electrical friction-type cone penetrometers were performed at the test site to characterize the strong $c$-$\phi$ soils, and CPTs using a miniature cone probe in the centrifuge were carried out for the very weak $c$-$\phi$ soils. Profiles of tip resistance $q_c$, sleeve friction resistance $f_s$, and shear wave velocity $V_s$ were obtained in the SCPTs. Only $q_c$ profiles, however, were obtained for the very weak $c$-$\phi$ samples as the centrifuge probe was not equipped with a friction sleeve.

The laboratory tests included index tests, unconfined uniaxial and triaxial compression tests. Effective stress soil strength parameters (i.e. cohesion $c'$ and friction $\phi'$ values) and the soil stress-strain behavior were derived from the shear tests. Considerable efforts and great care were taken in the laboratory testing to overcome sampling difficulties and meet the specimen criteria for the testing apparatus.
3.2 Artificial weakly-cemented sands

3.2.1 UWA superfine sand and cementing agent: general characteristics

Non-plastic uniform fine quartz sand, referred to as UWA superfine sand, was used for the centrifuge-scale model experiments. Artificial weakly-cemented sands were prepared by modifying the superfine sand with addition of Portland cement by per cent of dry unit weight (wt.%) at different amounts, producing various cementation degrees upon chemical hydration. The particle size distribution (PSD) and basic properties of the sand are presented in Figure 3-1 and Table 3-1. This sand is classified as SW in the unified soil classification system (USCS) and the mean effective particle size $D_{50}$ is 0.19 mm. The dominant mineral in this sand is quartz (SiO$_2$), and the maximum and minimum void ratios, $e_{\text{max}}$ and $e_{\text{min}}$, are 0.81 and 0.52 respectively.

![Figure 3-1 PSD of UWA superfine sand $D_{50} = 0.19$ mm](image)

Portland cement (Cockburn Grey Cement AS 3972 Type GP) was used as the cementing agent for the very weakly-cemented soil samples because cement-sand soils can show similar mechanical responses to those of natural cemented sediments (Puppala et al. 1995). The minimum compressive strength of Portland cement is about 35 MPa at
three days curing and cured moisture of approximately of 8 percent; the more water that is present, the lower the strength.

Table 3-1 Soil characteristics of UWA superfine sand [values from (Bienen et al. 2011; Rattley et al. 2008)]

<table>
<thead>
<tr>
<th>$G_s$</th>
<th>$D_{10}$ (mm)</th>
<th>$e_{max}$</th>
<th>$e_{min}$</th>
<th>$\gamma_{max}$ (kN/m$^3$)</th>
<th>$\gamma_{min}$ (kN/m$^3$)</th>
<th>Cu</th>
<th>Cc</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.647</td>
<td>0.19</td>
<td>0.81</td>
<td>0.52</td>
<td>18.0</td>
<td>14.9</td>
<td>2</td>
<td>0.12</td>
<td>SW</td>
</tr>
</tbody>
</table>

3.2.2. Soil preparation and characterization

Soil preparation took place in the first phase of the model pile experiment in the geotechnical centrifuge. The soil samples were prepared through dry pluviation of sand and cement mixture into a strongbox, using a sand pluviator device at 1 $g$, see section 5.3.2.

The soil samples had three cementation levels. Un-cemented sand was used as a reference whilst cemented sand mixtures were prepared at 2.5% and 5% cement contents, referred to as CS0, CS2.5 and CS5 throughout the following sections. The dry soil unit weight, $\gamma$, was 16.2 kN/m$^3$ ($\rho = 1.664$ g/cm$^3$) for sample CS0 and 15.8 kN/m$^3$ ($\rho = 1.608$ g/cm$^3$) for both CS2.5 and CS5. The calculation of the void ratio ($e$) for each sample was based on the overall weight of the dry sample and the volume of the sample in the strongbox when the dry samples were prepared.

Following the preparation of the dry samples, the addition of water to a moisture content of 16% and subsequent curing of the sample to achieve the required degree of cementation was conducted at 1 $g$ for all samples. After a curing period of 20 hours, the strongboxes containing samples were then mounted into the centrifuge channel in the arrangement shown in Figure 3-2a. They were then spun to the target gravitational level of 50 $g$ and water added to bring the level fully saturated for the soil characterization and lateral load test.
Figure 3-2 Strongbox arrangement in drum channel and CPT plan: (a) a schematic plan view of four strongboxes; (b) a schematic for the CPT and model pile plan layout in a strongbox; and (c) a photo showing CPT and model pile plan in a strongbox.

Soil strength profiles were obtained from the CPTs that were performed in-flight at a penetration rate of 1 mm/s, using the 5-mm diameter miniature probe. Cone penetration testing was conducted at the locations relative to the model pile locations shown in Figure 3-2. Two CPTs were performed in each strongbox for soil samples CS0 and CS2.5. One CPT was performed for soil sample CS5 due to its delicate cementation and hence need to minimize impact on adjacent lateral pile. Some local surface soil cracking was observed as the cone advanced to a great depth in the CS5 sample; such cracking...
can be expected to contribute to the fluctuate nature of the $q_c$ profiles measured.

3.2.3. Miniature cone penetration tests

The CPT was performed using a miniature cone penetrometer, as shown in Figure 3-3. The cone tip is 5 mm in diameter with a projected base area of 19.6 mm$^2$. When testing at 50 g, it represents a relatively large object (250 mm in diameter), which is nearly seven times the standard 35.7 mm diameter cone used in practice. A penetration rate of 1 mm/sec was used in the testing as this is a commonly used rate for testing in the UWA centrifuges. Cone penetration was performed on the other side to the loading direction of the model pile, approximately 80 mm (16 times the cone diameter or 16 $D_{CPT}$) from the internal wall, as shown in Figure 3-2c. The ratio of the equivalent chamber diameter to the cone diameter of 46 was greater than the value of 40 proposed by Puppala et al. (1995) to eliminate any significant boundary effects on the CPT data.

![Figure 3-3 The miniature quasi-static cone penetrometer (dimension in mm)](image)

Two strain gauges were attached to an aluminium inner rod to measure tip resistance, one on each side of the inner rod. They were connected in series to eliminate the effects of a possible bending of the inner rod. The gauges were calibrated by applying known forces at the tip and were found to behave linearly up to approximately 500 N, which is greater than the maximum tip resistance measured throughout this soil characterisation. The cone was not equipped to measure friction or pore pressures. Figure 3-4 presents the CPT results for samples CS0, CS2.5 and CS5. The cone requires several diameters of penetration to reach steady-state resistance (Bolton et al. 1993) and the depth of penetration required for a fully developed resistance can vary slightly, depending on the strength and stiffness of the soil (Ahmadi and Robertson 2005; Xu and Lehane 2008). In this study, the single-layered artificial soil samples were prepared to be homogenous. As can be seen from the figures, increasing the cementation degree increases the cone tip resistance.
Figure 3-4 Results from miniature-CPT tests for soil samples: (a) CS0 (uniform sand); (b) CS2.5; and (c) CS5 (in model dimensions; \( L_e \) = embedded depth and \( D \) = pile diameter)

For sample CS0, a uniform medium-dense sand, the cone end resistance \( q_c \) values increases proportionally with depth \( z \) for almost the whole depth range, at an average gradient \( q_c' (dq_c/dz) \) of about 0.83 MPa/m. For the cemented sand samples CS2.5 and CS5, cone end resistance \( q_c \) values increased sharply with depth \( z \) for the first 70 mm and 60 mm depth (3.5 m and 3 m depth in prototype dimension), at average gradients \( q_c' \) of 1.6 MPa/m in CS2.5 and 3.6 MPa/m in CS5. Below these depths, the \( q_c \) values tend to stabilize at about 5.2 MPa in CS2.5 and 13 MPa in CS5. The CPT data are summarized in Table 3-2.

Table 3-2 The gradient of end resistance \( (q_c') \) for all soil samples

<table>
<thead>
<tr>
<th>Soil type</th>
<th>CS0</th>
<th>CS2.5</th>
<th>CS5</th>
</tr>
</thead>
<tbody>
<tr>
<td>( q_c = dq_c/dz ) (MPa/m)</td>
<td>0.83</td>
<td>1.6</td>
<td>3.6</td>
</tr>
</tbody>
</table>
The cone resistances ($q_c$) measured in samples of cemented sands were much higher than those in uncemented sand, the values of the gradients $q_c'$ for shallow depths and limiting pressure for deep depths of CS2.5 and CS5 are greater than that of CS0. In other words, an increase in $q_c$ values owing to an increase in degree of cementation reveals a considerable contribution of the $c'$ component of strength to the resulting resistance of the penetrometer advancement over depth. Figure 3-5 shows the ratio of $q_c$ values recorded in CS2.5 and CS5 to the $q_c$ values in un-cemented sand CS0, suggesting the $q_c$ values increase with cement content for all depths. For a given relative density, the internal friction angle of cemented sands is similar to that of un-cemented sand, it appears that the penetration mechanism of cemented sand is different from that of un-cemented sand (Rad and Tumay 1986).

Relationships between cone tip resistance and soil strengths developed for un-cemented quartz sands were chosen and considered feasible to assess the effective stress friction angle for CS0 through normalising the cone tip resistance (Kulhawy and Mayne 1990). Correlations between cone tip resistance and shear strengths for cemented sandy ($c$-$\phi$) soils are still unavailable.
For sands, the effective stress friction angle can be evaluated through normalising tip resistance, \( q_{t1} = (q_t/p_a)/(\sigma_{vo}/p_a)^{0.5} \), where \( p_a = \) a reference stress equal to one atmosphere (\( p_a = 1 \) bar = 100 kPa) (Jamiolkowski et al. 2003). The relationship is written as

\[
\phi' = 17.6^\circ + 11.0^\circ \log(q_{t1}) \quad (3.1)
\]

Figure 3-6 presents its application to the uncemented sand CS0 for the depths where are the steady state of penetration. On average, the typical value of the friction angle determined from the miniature CPT profiles is approximately 37° for soil sample CS0.

3.2.4. Soil laboratory testing program

Parallel soil element tests were carried out to gain knowledge of the strength and stiffness of the very weakly cemented sands. The laboratory test program involved unconfined compressive strength tests and triaxial compression tests. Isotropic compression tests with shear wave velocity measurements using bender elements
allowed for the measurement of the very small strain shear modulus, $G_0$. Soil preparation procedures for all element tests were similar to those for the centrifuge model pile tests. First, a mixture of superfine sand and Portland cement at an appropriate ratio was carefully rained into a 72-mm diameter tube. Water was then introduced to gently permeate into the sand-cement mixtures. Curing at the standard indoor temperature (maintained at 23±2°C) was allowed for approximately 20 hours before testing; this was the same curing period employed before testing in the centrifuge samples. Soil samples were extruded from the soil tube to form specimens for each individual test.

As stated, the sand and cement mixtures were prepared with differing cement contents. The mixtures were rained into a 72-mm diameter and 220-mm high tube mould, with an inner wall coated with petroleum jelly to ensure smooth extrusion of the samples at a later stage. The bottom of the tube was sealed with clear acrylic base to prevent water leak and to clearly view the sample once water fully reaching bottom. The soil sample in the tube was generally prepared at 200 mm in height and the density was determined from the dry weight of the mixture. Then, water was added gently to the soil surface, which was covered with a filter paper to avoid any surface disturbance. Water permeating to the bottom of sample was viewed from the clear base plate. The tube soils were cured under effective stresses of 25 kPa and 54 kPa, applied on the top as a pre-consolidation process. Finally, the sample was extruded from the base after 20 hours curing for triaxial tests under isotropic stresses of 50 kPa and 200 kPa. The soils were prepared as isotropically normally consolidated (NC) soils. The length-to-diameter ratios of each specimen were prepared at two.

PSD tests were only conducted for the UWA superfine sand. The other samples were artificially prepared at the quoted densities and cement contents. Figure 3-1 shows the PSD information for the superfine sand. Other index parameters such as the maximum and minimum density and specific gravity were obtained from previous research at UWA (Bienen et al. 2011; Rattley et al. 2008).
3.2.5. Unconfined compressive shear (UCS) test

Unconfined compressive strength $q_{uc}$ has been proposed as a measure of the degree of cementation e.g. Beckwith and Hansen (1982); (Rad and Clough 1982). Deposits with $q_{uc}$ of less than 300 kPa are classified as weakly cemented, whereas very weakly cemented deposits have $q_{uc}$ values of less than 100 kPa. The unconfined compressive shear (UCS) test is a special version of the triaxial compressive shear test, in which the confining stress is zero and the state of stress is restricted to axial compression. Samples of any convenient size may be used. The testing apparatus used for these tests was the UWA triaxial cell chamber. The diameter and height of the specimens for the UCS test were 71.5 mm and 150 mm, respectively, as shown in Figure 3-7. Prior to each individual UCS test, there was no saturation treatment in water for each test specimen, and at least three specimens for each cement content (2.5% and 5%) were tested.

![Figure 3-7 UCS test on a sample CS2.5](image)

**Table 3-3** summarizes the results of the UCS tests. The strengths fall within the very weakly sand and weakly cemented sand category. As expected, the compressive strength increased with an increase in cement content. Some deviations in the measured $q_{uc}$
values of each soil type may be attributed to slightly different curing periods. These $q_{uc}$ values are only approximately half of the strength recorded in similar UCS tests on cement-sand mixtures reported by Rattley et al. (2008), who used early strength rapid hardening cement (Type III) as the cementing agent and the water content of 12%.

Table 3-3 Summary of unconfined compressive strength $q_{uc}$ (kPa)

<table>
<thead>
<tr>
<th>Samples</th>
<th>$q_{uc}$ (kPa)</th>
<th>$c'$ (kPa)</th>
<th>Curing (hrs.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS2.5</td>
<td>50</td>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td>CS5</td>
<td>125</td>
<td>30</td>
<td>24.5</td>
</tr>
</tbody>
</table>

3.2.6. Drained triaxial compression tests

Isotropic compression tests under fully drained conditions (CID) on all cemented sands and un-cemented sand were undertaken using a standard hydraulic triaxial cell. Pairs of bender elements were placed for each specimen to allow for the measurement of the shear wave velocity. Confining stresses ($\sigma_3$) of 50 kPa and 200 kPa were employed for each cement content (0%, 2.5% and 5%), giving a total of six CID tests. Soil samples at 200 mm in height were prepared in a similar procedure to that described above in a 72-mm diameter tube. Prior to transferring and trimming the samples into the triaxial cell, the samples were one-dimensionally compressed for 20 hours by applying a vertical loading equivalent to vertical stress of either 25 kPa or 54 kPa. The average dimension and height of the specimens were 71.5 mm and 150 mm. The lower part of the tube soil samples was used in the tests as some disturbance can occur to the upper section of the soil samples.

In the drained triaxial compression testing, a back pressure of 700 kPa was used to saturate the specimens. Saturation was checked by measuring the $B$ values ($B = \Delta u/\Delta \sigma_3$) and it was found that $B$ was greater than 0.95 in all cases. The samples were then sheared in a triaxial compression drained condition. A constant vertical displacement rate of about 0.025 mm/min (1%/hr) was used in all drained compressive shear tests. Specimen moisture was first measured using a cut-off sample from a specimen trim. Using a post-shear specimen upon test completion, the moisture content was measured a second time. Figure 3-8 shows a photo of a standard triaxial test upon completion (left)
The test results are presented in terms of the deviator stress \( q = (\sigma'_1 - \sigma'_3) \) [\( q' = (\sigma'_a - \sigma'_r) \)], where \( \sigma'_a \) = effective axial stress, \( \sigma'_r \) = effective radial stress, mean effective stress \( p' = (\sigma'_1 + 2\sigma'_3)/3 \) [\( p' = (\sigma'_a + 2\sigma'_r)/3 \)], axial strain \( \varepsilon_a = \Delta L/L \), volumetric strain \( \varepsilon_v = \Delta V/V \), radial strain \( \varepsilon_r = (\varepsilon_v - \varepsilon_a)/2 \), and the triaxial shear strain \( \varepsilon_s = (2/3)(\varepsilon_a - \varepsilon_r) \).

Figure 3-9 to Figure 3-11 present the triaxial test results, the deviator stress-axial strain relationships (i.e. plots of \( \varepsilon_a - q \) data), and the volume stress-axial strain relationships (i.e. plots of \( \varepsilon_v - \varepsilon_a \)) for all three types of soil samples. At both low and high confining stresses, peak strengths were attained for all samples at an axial strain of between 2.5% and 6.4%; these peaks were followed by rapid strain softening as a shear plane formed. As the confining pressure increases, the ratio of the peak strength to the ultimate (critical state) strength decreases. The axial strain that was required to develop the peak state also increases (corresponding to a transition from a shear plane failure to a barreling mode). The peak strength displayed on the stress-strain relationships for the un-cemented sand CS0 is indicative of a medium-dense sand. For the very weakly
cemented sands CS2.5 and CS5, the stress-strain responses indicated a significant cementation influence, which is consistent with previous studies reported (Clough et al. 1981; Coop and Atkinson 1993; Huang and Airey 1993). For normally consolidated samples, an increase in cement content results in an increase in stiffness and strength, and greater brittleness.

The peak stresses and critical stresses can be seen in Figure 3-9 to Figure 3-11. The respective values of the variables are listed in Table 3-6, which is presented at the end of this chapter.
Figure 3-9 Stress-strain behavior of soil sample CS0: (a) deviator stress-axial strain relationships, (b) volumetric strain-axial strain data under stresses of 50 kPa and 200 kPa.
Figure 3-10 Stress-strain behavior of soil sample CS2.5: (a) deviator stress-axial strain relationships, (b) volumetric strain-axial strain data under stresses of 50 kPa and 200 kPa
Figure 3-11 Stress-strain behavior of soil sample CS5: (a) deviator stress-axial strain relationships, (b) volumetric strain-axial strain data under stresses of 50 kPa and 200 kPa
Figure 3-12 presents the $q/p'$ ratio variations with axial strain for all soil samples under two confining stresses. The peak stress $q/p'$ ratios of cemented sands are generally higher than those of un-cemented sand under same stress level. For each soil sample, the value of the $q/p'$ ratio increases with mean stress. Under high mean stress, the weakly cemented sands transition from a less-defined brittle response to a ductile response; under lower mean stress, the soils behave with more brittleness than those under high stress.

Figure 3-13 presents the stress-strain relationships for all tested samples at two confining stresses. By comparison, these curves clearly show that the cement amount and the confining stress both influence the peak strength and the stiffness of the very weakly cemented sands. At a lower mean stress, the axial strain at peak strength is generally less than that at a higher stress level for each type of soil sample. The peak strength of a soil sample developed at a low stress requires a relatively smaller soil deformation than that of a high stress level.
Figure 3-12 Variations of $q/p'$ ratio with $\varepsilon_a$ for all soil samples under stresses of: (a) $\sigma'_3 = 50$ kPa; and (b) $\sigma'_3 = 200$ kPa
Figure 3-13 Stress-strain relationships for all types of soil under stresses of 50 kPa and 200 kPa, showing the effect of cement content on peak strength and initial stiffness.

Figure 3-14 plots the deviator stress normalized by the peak stress while the axial strain is normalized by a quantity $\varepsilon_{50}$, in which the developed stress equals to 50% of the peak stress. As can be seen in Figure 3-14, the normalized stress-strain behavior of these soil samples obtained from drained triaxial tests differ from each other. This is owing to some degree to the differences in the degree of cementation and confining stress. Even though some deviations exist for the normalised stress and strain data points after peak stresses, it is reasonably rational to use a single function to represent all the dimensionless stress-strain relationships before peak stresses. Non-linear regression analyses of the dimensionless stress-strain data, see Figure 3-14b, yield a best-fit relationship that can be written as

$$\frac{\sigma_3' - \sigma_3}{(\sigma_3' - \sigma_3')_{\text{max}}} = \tanh \left[ 0.546 \left( \frac{\varepsilon}{\varepsilon_{50}} \right)^{0.90} \right] \quad (3.2)$$

The least-squares method was used to evaluate the accuracy of the regression model, with a value of 0.997 calculated for the $R^2$, a coefficient of determination. This $R^2$ value
indicates a very good fit of the functional trigonometry Equation (3.2) with the non-dimensional stress-strain data.

Figure 3-14 Non-dimensional stress-strain relationships, $\frac{\varepsilon_a}{\varepsilon_{50}} - \frac{(\sigma_1' - \sigma_3')}{(\sigma_1' - \sigma_3')_{\max}}$, of all centrifuge soils: (a) complete curves; (b) the initial to the peak stress portion curves.
A MC criterion has been used to derive the shear strength parameters from the drained triaxial tests in the following discussions. The MC equation (criterion) is the most widely used to fit experimentally determined failure states for soil strength. The values of the derived peak strength parameters, the cohesion and peak friction angle, are listed as in Table 3-4.

Table 3-4 Values of MC strength parameters derived from triaxial compression tests

<table>
<thead>
<tr>
<th>Sample</th>
<th>Cohesion, $c'$ (kPa)</th>
<th>Friction angle, $\phi_{peak}'$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS0</td>
<td>0</td>
<td>39</td>
</tr>
<tr>
<td>CS2.5</td>
<td>28</td>
<td>33</td>
</tr>
<tr>
<td>CS5</td>
<td>26</td>
<td>39</td>
</tr>
</tbody>
</table>

In Table 3-4, the calculated $\phi'$ value of 33° for the soil sample CS2.5 is lower but the $c'$ value of 28 kPa, which was slightly higher than expected compared with values for CS0 and CS5. For soils at a similar density, the friction angles of shearing resistance are assumed to be similar to each other, irrespective of the degree of cementation (Bienen et al. 2011; Puppala et al. 1995; Rattley et al. 2008). The same precise densities for all samples can be challenging and may not be achievable in a small diameter tube mould. For the purpose of consistent comparisons, a reasonable cohesion value for sample CS2.5 is of interest if it has a similar friction angle of close to 39°, as seen in CS0 and CS5.

Assuming friction angles varying from 37° to 39° for CS2.5, the cohesion $c'$ values may be calculated ranging from 12 kPa to 16 kPa. In a similar manner for the samples of CS0 and CS5, the values of the MC strength parameters may also be calculated by varying the friction angle to be approximately 37–39°, which are summarized in Table 3-5. It indicates that the cohesion intercept increases with the cement content. Figure 3-15 shows $c'$ and $\phi'$ variations of derivations from triaxial tests, adjusted by assuming all of the soil samples have similar friction angles of 37–39°.

Table 3-5 Adjusted peak strength parameters for centrifuge soil samples

<table>
<thead>
<tr>
<th>Sample</th>
<th>Cohesion, $c'$ (kPa)</th>
<th>Friction angle, $\phi_{peak}'$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS0</td>
<td>0</td>
<td>38±1</td>
</tr>
<tr>
<td>CS2.5</td>
<td>12 – 15</td>
<td>38±1</td>
</tr>
<tr>
<td>CS5</td>
<td>27 – 33</td>
<td>38±1</td>
</tr>
</tbody>
</table>
Figure 3-15 Variations of $c'$ and $\varphi'$ values from triaxial tests and calculations fixing the modified friction angles of: (a) $37^\circ$; and (b) $39^\circ$

In addition to the peak strengths indicated in the drained triaxial tests, the ultimate
strengths (critical state) may also be determined. At large strain, it was found that the stress ratios $q/p'$ for all of these samples approach a constant, equivalent to a critical friction angle $\varphi_{\text{crit}}$, at the end of the tests, as shown in Figure 3-12. The $\varphi_{\text{crit}}$ value for cemented sands (34°) was slightly higher than that for un-cemented sand (32°).

Table 3-6 tabulates the values of the variables in the shearing tests. As the values of the initial modulus are rather scattered and considered less reliable, they are not presented herein. The secant moduli $E_s$ to the stress-strain curve are taken as the soil moduli at an axial strain of 50% of the peak deviator stress ($q_{\text{peak}}$). The values of the Poisson’ ratio $\nu$ are also taken at an axial strain of 50%. Assuming isotropic elastic behavior, Poisson’s ratios $\nu$ computed from the CID tests give values ranging from 0.15 to 0.36, with the values increasing with an increase in confining stress and strain level. It appears that the cementation slightly reduces the Poisson’ ratio, whilst the density slightly increases the Poisson’ ratio.

3.2.7. Bender element tests

The bender element (BE) tests involved the transmission of a single–shot sine wave from a BE at one end (bottom) of the sample and the measurement of its first arrival by BE at the other end (top) of the sample (travelling a distance, $L_{\text{specimen}}$). The BE is 4 mm in height, 6 mm in width and 2 mm in thickness. The shear wave velocity propagating across the specimen ($V_s$) and $G_o$ may be determined from

$$G_o = \rho V_s^2 = \rho \left( \frac{L_{\text{specimen}}}{\Delta t} \right)^2$$

where $\rho$ is the total mass density of the specimen, $L_{\text{specimen}}$ is the tip-to-tip length between the two BEs, and $\Delta t$ is the travel time of the shear wave through the specimen. Upon the completion of consolidation at specified stresses, three frequencies ranging from 4.5 kHz to 12.5 kHz were investigated to ensure that the measured time was not frequency-dependent. Following this, four travel times were measured in the initial stages of shear testing at incremental axial displacement of 0.25 mm, 0.5 mm, 1 mm and 2 mm, to obtain sufficient measures for an average value.
Table 3-7 summarizes the changes in values of the measured $G_{\tan}$ (including the $G_o$ that shearing strain less than 0.1%) with small strains $\varepsilon$ under stress levels of 50 kPa and 200 kPa. For the un-cemented sand sample CS0, the shear modulus appears to increase slightly with the axial strain, whereas the shear modulus tends to decline with strain in the cemented sands CS2.5 and CS5. For instance, at the axial strain level ($\varepsilon_a$) of 0.2%, the $G_{\tan}$ values, including the $G_o$ values, of soil sample CS0 at 72 MPa and 170 MPa, become 85 MPa and 192 MPa at a strain of 0.7% under two stresses. For example, in the case of cemented sands under stresses of 50 kPa, the initial shear modulus can be as high as 239 MPa for CS2.5 and 373 MPa for CS5. It degrades to 197 MPa and 299 MPa when strain reaches approximately 1.31%.

Figure 3-16 presents the $G_{\tan}$ values and the cement content of the three types of soil samples. It is evident that the cemented sand specimens of CS5 and CS2.5 have much higher values of small strain shear modulus than the un-cemented sand specimen CS0. The relationship between the $G_{\tan}$ and the cement content is shown to be nearly proportional. Meanwhile, the stress level is shown to have influence on the $G_{\tan}$ for both un-cemented sand and weakly cemented sands. For weakly cemented sands CS2.5 and CS5, however, the shear modulus will decrease even with the small strain as shearing destroys the cementation fabric. For uncemented sand CS0, the $G_o$ seems to be slightly improved as the soil contracts with initial shearing and consequently becomes denser.
Figure 3-16 Small strain shear modulus measured from bender element tests: (a) the effect of cement content on initial stiffness $G_o$ and small-strain stiffness $G_{tan}$; (b) the effect of stress level on $G_o$ and $G_{tan}$
3.2.8. Effect of cementation on shear strength

The relatively small dependence of $q_{peak}$ on the cement content indicated in the triaxial tests arises because this difference is only due to difference in $c'$ between the samples. Such differences lead to a significant dependence of the UCS strength on $c'$, as stress level effects are virtually absent.
### Chapter 3 Soil properties

#### Table 3-6 Results of drained triaxial tests for centrifuge model soils

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Confining stress $\sigma_a'$ (kPa)</th>
<th>Bulk density (g/cm$^3$)</th>
<th>$\varepsilon_{50}$ (%)</th>
<th>$\varepsilon_a$ (%)</th>
<th>$\varepsilon_v$ (%)</th>
<th>Peak strength</th>
<th>Ultimate strength#</th>
<th>Young’s moduli $E^*$ (MPa)</th>
<th>Poisson’s ratio* (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS0</td>
<td>50</td>
<td>1.648</td>
<td>0.67</td>
<td>169</td>
<td>4.4</td>
<td>-1.6</td>
<td>117</td>
<td>1.32</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>1.655</td>
<td>0.76</td>
<td>668</td>
<td>3.8</td>
<td>-1.1</td>
<td>350</td>
<td>1.29</td>
<td>56</td>
</tr>
<tr>
<td>CS2.5</td>
<td>50</td>
<td>1.570</td>
<td>0.38</td>
<td>228</td>
<td>2.9</td>
<td>-1.2</td>
<td>146</td>
<td>1.48</td>
<td>37</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>1.574</td>
<td>1.05</td>
<td>593</td>
<td>4.6</td>
<td>-0.4</td>
<td>458</td>
<td>1.30</td>
<td>54</td>
</tr>
<tr>
<td>CS5</td>
<td>50</td>
<td>1.563</td>
<td>0.37</td>
<td>284</td>
<td>2.5</td>
<td>-0.6</td>
<td>169</td>
<td>1.58</td>
<td>47</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>1.528</td>
<td>1.13</td>
<td>814</td>
<td>6.4</td>
<td>-0.5</td>
<td>554</td>
<td>1.44</td>
<td>45</td>
</tr>
</tbody>
</table>

\# Values at $\varepsilon_{axial} = 20\%$ (=18% CS2.5 50kPa =16% CS5, 50kPa)  
* Values for 50% peak deviator stress

#### Table 3-7 Small strain shear modulus $G_o$ measured from bender element tests for centrifuge model soils

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Confining stress (kPa)</th>
<th>$\varepsilon_a$ (%)</th>
<th>$G_{tan}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS0</td>
<td>50</td>
<td>0.00</td>
<td>69 (G_o)</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>0.00</td>
<td>169(G_o)</td>
</tr>
<tr>
<td>CS2.5</td>
<td>50</td>
<td>0.16</td>
<td>72</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>0.17</td>
<td>170</td>
</tr>
<tr>
<td>CS5</td>
<td>50</td>
<td>0.66</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>1.31</td>
<td>189</td>
</tr>
</tbody>
</table>

Note: The values for $G_{tan}$ are provided for different confining stresses and strain levels, illustrating the small strain shear modulus $G_o$ measured from bender element tests.
3.3 Natural well-cemented sands and weak rocks

3.3.1. Site geology

Lateral pile tests were performed in a limestone quarry in Pinjar, about 25 km north of Perth, WA. The quarry mine is classified as a 'medium grade limestone', which is part of the Tamala Limestone formation. Tamala Limestone (otherwise known as Tamala Eolianite) is the geological name given to the widely occurring eolianite limestone sediments on the western coastline of WA, stretching more than 1000 km from Shark Bay in the north and to near Albany in the south. In the Perth region, carbonate eolianites of Tamala Limestone extend up to 10 km inland of the modern coast (bounded in the east by the Darling fault) and up to 40 km or more offshore in their submarine extent, unconformably overlying the Cretaceous and Tertiary formations.

The Tamala Limestone formation was first known as the Coastal Limestone formation (Saint-Smith 1912; Teichert 1950) and described as a grainstone exhibiting large-scale cross bedding. The name 'Tamala Eolianite' was proposed to describe the formation on the Shark Bay area of the Carnarvon Basin (Logan et al. 1970). Eolianite is a consolidated sedimentary rock consisting of mineral granular material deposited by the wind. The current name Tamala Limestone refers to the lithological sequence rather than a genetic term, which was proposed subsequently by Playford et al. (1976). Brooke (2001) refers to the Australian carbonate eolianite as “the world’s most extensive eolianite deposit”.

The lithology of the Tamala Limestone comprises calcarenite and calcareous cemented, fine to very coarse, poorly sorted sand that has accumulated as coastal sand dunes during the Middle and Late Pleistocene and Early Holocene eras (approximately 1.8 million years ago) (Playford et al. 1976). Calcarenite was composed mainly of foraminifera and mollusk fragments. As a result of sedimentation and water percolating through the shelly sands, the mixture later lithified when the lime content dissolved to cement the grains together. The term 'aeolian' refers to the wind-blown origin of the sediment, and 'calcarenite' indicates that the majority of the sand grains in the original sediment were calcium carbonate (either calcite or aragonite). The degree of cementation of the sediments varies from fully
indurated (by calcite and calcrete) to weakly indurated, to totally un-cemented with predominantly quartz sand grains being dominant; see Figure 3-17.

A vast complex of former dunes has risen up to 258 m high, with outcrops tens of metres thick and perhaps over 150 m thick in the southern end. Dune units are often interspersed with shallow-marine units in shore-parallel ridges. The most extensive of these is the Spearwood Dune System, which is derived from the Tamala Limestone formation.

Figure 3-17 (a) Carbonate particles of marine origin in the Tamala Limestone from Shark Bay, including quartz, porcelaneous and Elphidium-like foraminifers, and red algae (Le Guern and Davaud 2005), and (b) scanning electron micrograph showing micritic meniscus cement in eolianite from Libya, precipitated after dissolution of aragonite and high-magnesium calcite particles by under-saturated percolating water (McLaren and Gardner 2004).

3.3.2. WA calcium cemented sands or weak rocks

No borehole data were available at the Pinjar site prior to the involvement of UWA in 2010. Steeper and taller slopes exposed in the quarry site demonstrate that considerable cement bonds contribute to the shear strength of the sandy soil masses and their ability to resist tensile stresses. The ground in the area of the lateral test piles was relatively flat. A mixture of cemented sands and un-cemented sands is present from the ground surface. Sample from two rotary cored boreholes close to the test site (Lehane 2010), indicate that the deposits
are a heterogeneous mixture of weakly cemented harder lenses interbedded with very poorly or un-cemented sandy soils. The soil profiles revealed from the excavation inspection (see Figure 3-18) showed cemented sediments, which can be described as laminated, very thinly-bedded and varying in thickness from 2.5–10 mm to 10–50 mm and in places over 50 mm thick. The formation is easily broken using a mechanical excavator tool, leaving harder cemented lumps and un-cemented sand. No groundwater was encountered within the depth penetrated by the test piles at the time of field testing.

Figure 3-18 Features observed for the deposits at Pinjar test site: (a) bedding structure (location of pile 340A), (b) interlayer of poorly cemented sands and well cemented sands or lithified (weak rocks)

The in-place density for the Pinjar material was assessed using the sand cone replacement method, see Figure 3-19. To execute the sand cone replacement method, a hole was first excavated within the excavated test pit to a specified dimension of 15.2 cm in diameter and 20 cm deep. A standard uniform dry sand ($D_{50} = 0.5 \text{ mm} \sim 0.6 \text{ mm}$) was used to fill the hole through a graduated bottle after the in-situ soil was removed. The volume of the hole was determined from the device’s calibration between volume filled and the weight of the sand placed. The measured in-place bulk unit density was determined as $1.58 \text{ g/cm}^3$ and the in-situ void ratio was 0.683.
The field test was conducted in late spring when the rainy season had passed. No groundwater table was observed at the time of testing. The measured moisture content was approximately 4.5%, implying a bulk (dry) unit weight of 15.0 kN/m$^3$.

3.3.3. Cone penetration test

In-situ CPTs were conducted by Probedrill Pty Ltd to characterise the site prior to the full-scale pile load tests. The electronic penetrometer used had a standard 10 cm$^2$ projected end area (35.7 mm in diameter) and a standard friction sleeve. No pore pressure measurements were available. Penetration refusal occasionally occurred, suggesting the lithified stratum is in places interlayered with very hard cemented sandy soil. A total of six CPTs were successfully penetrated to a target depth of 6 m at the test site.

Figure 3-21 presents the results of the CPTs that successfully penetrated to a target depth of 6 m, in form of the cone tip resistance $q_c$, the side friction resistance $f_s$, and the friction ratio $R_f$. The friction ratios fall consistently within 0.4% to 0.6% range. The average $q_c$ value is approximately 35 MPa for soils at a depth between 1 m and 2 m (i.e. in the most critical zone for the lateral load tests), with some peak $q_c$ values of up to 60 MPa recorded. The traces show no clear evidence of the bedded and cemented nature of the deposit observed at the test site (as seen on Figure 3-18). Without such knowledge and the shear wave velocity...
data, standard CPT classification charts based on $q_c$ and $R_f$ suggest the material is a medium-dense sandy soil (Robertson 2009).

3.3.4. Seismic cone penetration test

To obtain small strain shear modulus data, seismic cone penetration tests (SCPT) supplemented the CPTs upon the completion of the load tests. Eight SCPTs in total were successfully conducted to a depth greater than the embedment depth of the test piles.

The stiffness of soils and rocks at small strains is finite and commonly denoted by the low strain modulus $G_0$ that can be determined from $G_0 = G_{\text{max}} = \rho V_s^2$, where $\rho$ = the soil/rock mass density (kg/m$^3$), and $V_s$ = shear wave velocity (m/sec). Research shows that the $G_0$ applies to both static and dynamic properties, as well as both drained and undrained loading conditions in geotechnical situations. The shear wave velocity ($V_s$) is a fundamental property of geomaterials, and representative of a non-destructive response at very small strains ($\gamma_s < 10^{-6}$ decimal) for geomaterials and non-geomaterials. Since water cannot sustain shear, shear wave measurements of soils are unaffected by the presence of groundwater. $V_s$ measurements in soils can be achieved using in-situ field and/or laboratory tests. In the present study, SCPTs were used for the Pinjar field geotechnical investigation program.

SCPT is a hybrid in-situ geotechnical investigation method, which optimizes data through combining two techniques into a single sounding. A velocity geophone located within the penetrometer is used to measure the arrival times of the shear waves. Figure 3-20 shows the set-up of the shear wave source for the SCPT. A horizontal plank positioned parallel with the geophone axis on the surface of the ground is struck to generate a source rich in shear wave energy. The shear waves are polarised horizontally and emanate vertically as a downhole test.

For geotechnical characterization of the Pinjar site, in-situ SCPT tests were performed in accordance with AS1289.6.5.1 (1999) and IRTP 2001 with a friction reducer for a geotechnical characterization of the Pinjar site. Pseudo-interval downhole shear wave propagation during cone soundings was measured at intervals of 0.5 m.
Figure 3-20 Pseudo-static downhole SCPT: (a) a photo of shear wave source on ground surface, and (b) a schematic of SCPT

The values in the continuous profiles of \( q_c \), \( f_s \), and \( R_f \) closely match those measured in the standard CPTs shown in Figure 3-21. The seismic cone shear wave velocities and the standard CPT data recorded at Pinjar are presented on Figure 3-22. The \( q_c \) values between 1m and 2m (i.e. zone of significance for the lateral tests) are typically between 30 MPa and 40 MPa, with a mean value of 35 MPa. The friction ratios \( R_f \) (\( f_s/q_c \)) are 0.5% and occasionally 1% at local depths, which when combined with the \( q_c \) data simply indicates that the soil is classified as medium-dense sand.

The shear wave velocities, \( V_s \), do, however, reveal the cemented nature of the deposit and are typically in the range from 520 m/s to 1480 m/s (some up to 4230 mm/s) with an average value of 950 m/s. The values are slightly higher than the reported range of 365 to 765 m/s) for soft rock (Clark and Walker 1977).

The derived \( G_0 \) profiles are presented in Figure 3-23. The variations of \( G_0 \) values indicate that the stiffness generally increase with the depth, but there are occasionally looser pockets. Based on elastic theory, the small strain Young’s modulus, \( E_o \), can be determined from the link \( E_o = 2G_0(1 + \nu) \), where \( \nu \) is the Poisson’ ratio, which is empirically taken as 0.1 for drained soils or weak rocks. Figure 3-23 also includes the \( E_o \) profiles.
Figure 3-21 Site characterization using CPT, showing the profiles of: (a) tip resistance $q_c$, (b) sleeve friction resistance $f_s$, and (c) friction ratio $R_f (FR = R_f = f_s / q_c)$ with depth $z$ in in-situ CPT.

* The red dotted-line in plots indicates the level of the test pit base.
Figure 3-22 Site characterization using SCPT, showing the profiles of: (a) tip resistance $q_c$, (b) sleeve friction resistance $f_s$, (c) friction ratio $R_f \text{(FR} = R_f = f_s/q_c\text{)},$ and (d) velocity of shear wave $V_s$ with depth $z$

* The red dotted-line in plots indicates the level of the test pit base.
Figure 3-23 Small strain shear modulus and elastic modulus profiles of cemented sands and weak rocks at Pinjar site: (a) $G_o$-$z$; (b) $E_o$-$z$

* The red dot-line in plots indicates the level of test pit base.

3.3.5. Soil laboratory tests

The objective of the laboratory testing program presented here was to develop the parameters for cemented sands and weak rocks to assist in the interpretation of the lateral load tests. The laboratory tests include classification tests as well as element tests to assess the material’s strength and stiffness. Classification tests allow the test site material to be categorised as an appropriate soil/rock type for a single layer stratigraphy (or into a limited number of arbitrary groups for a multi-layered stratigraphy), each of which is evaluated for the presence of materials with similar geotechnical properties. These include the particle size distribution (PSD), calcium content tests ($C_aCO_3$), compaction tests (maximum density tests, $e_{min}$ and $e_{max}$), specific gravity determination tests ($G_s$) and point load tests [$I_{a(50)}$], and UCS, direct shear and triaxial compression tests were employed to derive the geotechnical parameters.
Specimens used for the classification tests, except in the case of the point load tests, were generally the disturbed soil samples taken from excavation at a depth of 1 m. Specimens used for strength tests were relatively intact weak-rock-like lumps taken from hand-carved block samples.

3.3.6. Index tests

Index tests were performed to obtain necessary data for classifying the soils of weakly cemented and un-cemented sandy soils. The particle size distribution, natural moisture content, maximum and minimum density and calcium content were determined in accordance with Australian standards and ASTM (6913).

PSD analyses reveal that the average deposit grain size is 0.3–0.4 mm for both well cemented lumps and un-cemented sandy soils. These were ground down (using a mortar and pestle) and oven-dried prior to testing. Chemical tests on the cemented and un-cemented materials indicate that both materials consist primarily of quartz, at an approximate content of 60±5% with calcium carbonate occupying typically 40±5%. It appears that the carbonate richness increases slightly with the hardness of the samples selected. The overall content of siliceous sand, however, is always greater than the calcium carbonate content. Therefore, according to the classifications proposed by Clark and Walker (1977), the material can be termed to be calcareous sandstone. Interestingly, a classification of sandstone conflicts with the category of a limestone, which it is generally named.

The water content of the natural cemented sands and weak rocks at Pinjar site was measured as typically 4%. The void ratios were measured from compaction tests and were approximately 1.004 as a maximum value ($e_{\text{max}}$) and 0.515 as a minimum value ($e_{\text{min}}$). The relative density was 66%. Table 3-8 summarizes the physical properties measured from the above tests. After grinding the cemented material using a mortar and pestle, its $D_{50}$ was measured to be about 0.3 mm, as shown in Figure 3-24. The uniformity coefficient was approximately 3. Based on the soil properties, the weakly-cemented sands were classified as SP according to the USCS specification.
The Franklin point load test (Broch and Franklin 1972; Franklin 1985) was used to determine the point load strength index ($I_s$). This index provides a quick measurement of the strengths of unprepared samples of rock fragment, both in the field and the laboratory. The point load index $I_s$ must be corrected to a standard equivalent diameter ($D_e$) of 50 mm to deduce the (size corrected) strength index $I_s(50)$. The index $I_s(50)$ provides a simple method for establishing a rock strength classification. It can also be used to estimate properties such as the compressive and tensile strengths and to evaluate a measure of anisotropy by using an index $I_a(50)$, which is a ratio of the greatest to the least $I_s(50)$, measured perpendicular and parallel to the existing planes of weakness, respectively.

Point load tests were conducted on both arbitrary lumps of Pinjar material and on carefully hand-carved cemented rectangular prisms using a 55-kN capacity, hydraulic hand pump,
model ELE77-0115 point load tester. The specimens were longitudinally loaded, loading direction parallel to the bedding plane (foliation); these tests are referred to as Group 1 tests. They were also diametrically loaded, loading direction perpendicular to the bedding plane (foliation); these tests are referred to as Group 2 tests. The rate of increase in the load during all tests was approximately 0.01 kN/s. The maximum force on the test specimen \(P\) was read from a calibrated readout unit. The length-to-breadth ratio of the prepared samples was generally 2 and a cross-section consistently square area. The strength indexes were deduced and corrected using the following expression (Brook (1985):

\[
I_{s(50)} = f \cdot P / D_e^2
\]

(3.4)

Where \(f\) = size correction factor = \((D_e/50)^{0.45}\), \(P\) = applied load, and \(D_e\) = equivalent specimen diameter. Figure 3-25 shows photos of in-situ samples under point load test, well cemented (left) and moderately cemented (right) sands.

Figure 3-25 Photos of point load test, loading direction along with the bedding (left) and loading direction perpendicular to the poorly formed bedding structure (right)

Table 3-9 presents the results and statistical data for the point load tests on the pre-prepared prismatic samples. The average value of index \(I_{s(50)}\) is 0.67 MPa. For specimens with moisture, the \(I_{s(50)}\) values range 0.02 MPa to 0.43 MPa, and 0.15 MPa on average under loads parallel to the bedding plane (Group 1); the values range from 0.13 MPa to 0.55 MPa, average 0.26 MPa under loads perpendicular to bedding plane (Group
According to ASTM (D5731-08) and Bieniawski (1974), these strength indices are consistent with the classification of a very low strength rock.

Table 3-9 Statistics of $I_{a(50)}$ (MPa) data

<table>
<thead>
<tr>
<th>Test</th>
<th>$n$</th>
<th>$I_{a(50)}$ Ave (MPa)</th>
<th>$I_{a(50)}$ Min. (MPa)</th>
<th>$I_{a(50)}$ Max. (MPa)</th>
<th>STDEV</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td>31</td>
<td>0.148</td>
<td>0.023</td>
<td>0.433</td>
<td>0.095</td>
<td>0.64</td>
</tr>
<tr>
<td>Group 2</td>
<td>17</td>
<td>0.259</td>
<td>0.126</td>
<td>0.545</td>
<td>0.100</td>
<td>0.39</td>
</tr>
<tr>
<td>Group 3</td>
<td>25</td>
<td>0.671</td>
<td>0.045</td>
<td>1.803</td>
<td>0.505</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Group 3 tests shown in Table 3-9 are the tests conducted well after sampling when samples lost their moisture and were dried. The shear strengths of dried well-cemented specimens, are significantly higher than those with moisture (Group 1 and Group 2).

The value of greatest-to-least index ratio $I_{a(50)}$ is about 2, showing that the weak calcareous sandstone is an anisotropic material (Broch 1983).

3.3.7. Sampling for well-cemented sands and weak rocks

Geotechnical sampling of weakly-cemented materials can be extremely difficult, owing to the high variability in the degree of cementation over relatively short distances. For very weakly cemented sands, the direct separation of some laminar masses may easily occur or even collapse in the case of a reduction or absence of material confinement, resulting in loss of the delicate bonding. On the other hand, the sampling method for soils will not work for strong materials such as well-cemented sands and weak rock. Two rotary cored boreholes reported approximately 100 m from the test site indicate that the deposit comprises weakly cemented ‘cobbles’ in an un-cemented sandy matrix (Lehane 2010); the drilling process evidently washed out the un-cemented material. Only approximately half of the cores recovered were intact and the solid core lengths were generally less than 100 mm. This means that the rock quality designation was close to zero (RQD = 0), as shown in Figure 3-26.
Undisturbed ‘block’ samples were manually taken from a depth of approximately 1.5 m, by forming an isolated “island” and then cutting it off. This process of ‘block’ sampling is shown in Figure 3-27. Only reasonably well-cemented material could be sampled in this manner. The hand-carved block samples (three in total) were carefully wrapped with cling film and aluminum film package to avoid any moisture loss. Sufficient protection measures were implemented and the samples were transported to the laboratory and stored...
in a humid room for laboratory testing. A high level of caution was exercised during the whole process.

Figure 3-27 Hand-carved intact block sample from Pinjar test site; the block-sample dimensions about 300 mm × 400 mm × 400 mm and taken at a depth of approximately 1.5 m below soil surface

The weak rock specimens for the standard triaxial tests and unconfined compressive tests were cut from the block samples with a diamond saw to form square prism sections at a height-to-diameter ratio of 2. Although the sampling and cutting operations for the samples were very time-consuming, it was felt that the time spent was necessary to obtain a reliable estimate of the intact strength of the cemented material at the Pinjar test site.

3.3.8. Unconfined compressive shear (UCS) test

Sixteen prismatic sections with moderately rounded four edges were tested under compression in a computer-controlled, servo-hydraulic, intermediate-stiff, 250-kN-capacity INSTRON Tension-Compression load frame. Twelve specimens with average dimensions of 50 mm × 50 mm × 110 mm were tested with the compression load orientated parallel to the bedding plane. Four shorter specimens were tested with average dimensions of 50 mm × 50 mm × 50 mm in which the compression load was perpendicular to the bedding plane. Figure 3-28 shows photos of the UCS tests. All
specimens were prepared according to the standard specifications for UCS tests, in accordance with recommendations from ISRM and ASTM D2166. The samples’ cross-section was approximately square; the height-to-width ratios for specimens with a loading parallel to the bedding were between 2 and 2.5. Tests in which loading was applied perpendicular to the bedding had an aspect ratio of about 0.5 and hence, compressive strengths recorded in these tests are not directly comparable to the other tests.

A 10-kN-capacity load cell was added at the base platen to ensure a satisfactory resolution for the load readout. Loads are transmitted to the specimen through a flat, hard-ended steel platen at a width similar to that of the test specimens. The top platen is free to rotate slightly to secure the specimen if the ground ends do not meet perfectly parallel. The load was increased by loading at a constant displacement rate of 0.1 mm/min, up until the specimen’s failure. The resolution of load measurement is approximately 1 μN, and the displacements is recorded to a resolution of approximately 10 μm.

For all specimens, the recorded load vs. displacement responses display consistently brittle, as typically illustrated in Figure 3-29. That is, peak states are followed by rapid decline to residual states. The initial loads produce sections of inelastic, concave-upward load-displacement, followed by elastic, linear relationships as loads increase till reaching the peak values of compressive strengths; the concave-upward portion of the graph reflects bedding-in effects at the end platens and compliance in the loading system. Brittle extensions along the bedding plane (longitudinal splitting) dominate almost all of the specimens’ failure in the loading direction of the bedding orientation; vertical fractures occur across the bedding plane in a loading direction perpendicular to the loading direction, as shown in Figure 3-28.
Figure 3-28 UCS tests of weak calcareous sandstone: (a) ruptured specimens under loading in direction parallel to bedding plane; (b) ruptured specimens under loading in direction perpendicular to bedding plane
Figure 3-29 Typical stress-strain curves of UCS test: (a) showing strength $q_{ucs}$ of 4 MPa; and (b) $q_{ucs}$ of 0.14 MPa

The peak values recorded for the 12 specimens tested for unconfined compressive strengths range between 0.14 MPa and 4 MPa, an average $q_{ucs}$ value of 1.12 MPa, as summarized in Table 3-10. The UCS tests appear to have failed by tensile splitting and
this failure mode clearly affects the measured UCS of the material. The standard deviation
and the coefficients of variation indicate that the strength distributions of the Pinjar weak
rock are rather dispersed.

Table 3-10 Statistics $q_{ucs}$ data of specimens of length-to-width ratio of approx. 2

<table>
<thead>
<tr>
<th>n</th>
<th>$q_{ucs}$ Ave (MPa)</th>
<th>$q_{ucs}$ Min. (MPa)</th>
<th>$q_{ucs}$ Max. (MPa)</th>
<th>STDEV</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>1.1</td>
<td>0.14</td>
<td>4.0</td>
<td>1.15</td>
<td>1.05</td>
</tr>
</tbody>
</table>

Figure 3-30 compares the mean $q_{ucs}$ values and $I_{s(50)}$ values that tested on Group 1 and
Group 2 specimens. Using the average values of $I_{s(50)}$, 0.15 MPa and 0.26 MPa for Group
1 and Group 2 respectively, a relationship between $q_{ucs}$ and $I_{s(50)}$ may be expressed simply
as

$$q_{ucs} = (4.3 - 7.5) I_{s(50)} \quad (3.5)$$

A factor of 4.3–7.5 is close to 7. This is comparable to the $q_{ucs}$ to $I_{s(50)}$ ratio for rocks
suggested by Look (2007), and the range ratios of 2.7–8.8 for Middle East weak porous
carbonate rocks suggested by Abbs (1985). The derived factor also infers that the
correlations (factor of 16–24) between the point load index and the uniaxial strength for
conventional strong rocks are less appropriate for the Pinjar weak rock.

The $q_{ucs}$ values ($q_{ucs} < 5$ MPa) measured from the UCS tests indicate that the calcareous
sediments at Pinjar may be categorized as hard soils, very weak rocks, and weak rocks
according to recommended strength for soils and rocks with the intermediate category
(IGMs), $q_{ucs} = 0.5$-5 MPa, has been adopted predominantly for drilled pile shaft design in
some states in the US, according to the criteria outlined by O’Neill et al. (1996), has not
been adopted universally for most of the states.

The UCS results for calcareous sandstone at the Pinjar test site are in the range of those
reported for weak offshore carbonate rocks in the Middle East (Abbs 1983; Abbs 1985;
Abbs and Needham 1985). The ratio of UCS to $q_c$ falls in the range of 0.025–0.037. These
values are slightly lower than the value of 0.05 suggested for typical soft rocks or calcarenites offshore in North-West Australia (Erbrich 2004).

![Figure 3-30 Normal distributions of the measured values of $q_{u cs}$ and $I_{s(50)}$](image)

$\frac{q_{u cs\text{-MEAN}}}{I_{s(50)\text{-MEAN}}} \approx 4.3-7.5$

$I_{s(50)\text{-MEAN}} = 0.15-0.26 \text{ MPa}$  
$q_{u cs\text{-MEAN}} = (1.1 \text{ MPa})$

3.3.9. Drained triaxial compression tests

Generally the test setup and testing procedures for standard drained triaxial tests on the cemented carbonate sediments are similar to those for artificial weakly cemented sands. After the specimen has been set up, the specimen is saturated, isotropically consolidated and compressively sheared. Dry densities were determined from off-cuts from the same block sample; at least three measures were made for each block. The average densities of the test specimens ranged from 1.60–1.62 g/cm³.

The diameter and height of the specimens were carefully measured using a calliper with a resolution of 0.01 mm in at least four different positions. Note that the trimmed specimens were not perfect standard cylinders due to hardness difference between parallel and perpendicular to bedding structure. The average diameters of the specimens ranged from approximately 60–72 mm and corresponding heights ranged from approximately 115–140 mm. Figure 3-31 shows a photo of the setup of a weak rock specimen for a triaxial test and
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a schematic of the specimen from the in-situ weak rock. The direction of the major (axial) principal stress is consistently parallel to the bedding plane of the specimens.

Figure 3-31 Photo and schematic of specimen orientation in lab tests: (a) a setup of triaxial compression test on weak rock specimen, major principal stress applied parallel to the bedding; (b) a schematic showing the specimen cut along bedding plane

During the saturation stage, a back pressure of between 500 kPa and 700 kPa was applied and the $B$ values ($B = \Delta u/\Delta \sigma_3$) were generally greater than 0.95 in most tests. At the stage of consolidation, mean effective stresses ($p'$) of 50 kPa, 100 kPa, and 200 kPa were used for the weak rock samples, which were loaded at a stress rate of between 25 kPa/hr. and 50 kPa/hr.. The sample consolidation took at least four hours; the valve for back-drainage was opened at the start of the shearing stage. The axial load was applied to generate deviator stress at a constant axial strain rate of 1%/hr, a displacement rate of 0.017 mm/min. The test was terminated when the axial strain reached 20%. The deformed sample was removed and weighted, and it was then placed into oven to dry in preparation for the moisture content measurement.

Table 3-11 summarizes the values of the relevant variables for the compressive shearing tests on the three specimens. The secant moduli $E_s$ and Poisson’s ratio $\nu$ are calculated at

Table 3-11

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Bedding structure of in-situ weak rock</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Bedding structure of in-situ weak rock</th>
</tr>
</thead>
</table>
50% mobilization of peak strength. The displacement of the samples was measured at the platens (refer to Figure 3-32), which measurements can result in a significant under-estimation of stiffness, especially as the material becomes stronger.

Table 3-11 Results data of drained triaxial tests for weak rock at Pinjar test site

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Confining stress p' (kPa)</th>
<th>Density (g/cm³)</th>
<th>ε₅₀ (%)</th>
<th>Peak strength Stress q (kPa)</th>
<th>εₐ (%)</th>
<th>εᵥ (%)</th>
<th>Critical strength Stress q (kPa)</th>
<th>Ratio q/p'</th>
<th>E* (MPa)</th>
<th>υ* (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C2</td>
<td>50</td>
<td>1.60</td>
<td>0.20</td>
<td>1472</td>
<td>0.44</td>
<td>0.20</td>
<td>200</td>
<td>1.60</td>
<td>325</td>
<td>0.06</td>
</tr>
<tr>
<td>C3</td>
<td>100</td>
<td>1.62</td>
<td>0.40</td>
<td>1686</td>
<td>0.91</td>
<td>0.26</td>
<td>183</td>
<td>1.25</td>
<td>231</td>
<td>0.09</td>
</tr>
<tr>
<td>C4</td>
<td>200</td>
<td>1.62</td>
<td>0.41</td>
<td>1740</td>
<td>1.11</td>
<td>0.31</td>
<td>568</td>
<td>1.42</td>
<td>282</td>
<td>0.15</td>
</tr>
</tbody>
</table>

# Values at εₐ = 20%
* Values for 50% peak deviator stress

The stress-strain relationships for the Pinjar weak rock are presented in Figure 3-32. Under the stresses of 50 kPa and 100 kPa, the peak deviator stresses were measured to be 1.47 MPa and 1.69 MPa at axial strains of 0.44% and 0.91 respectively. Under a stress of 200 kPa, the peak deviator stress was recorded as 1.74 MPa.

As shown in Figure 3-32, the deviator stresses for all tests first increase linearly and then nonlinearly until a well-defined yield point is reached (peak value). This is followed by a rapid drop and then a slow decline towards an ultimate state. The concave-upwards initial component of the deviator stress-axial strain plots is likely to reflect some slack between the specimen and the top cap or base pedestal in set-up. The non-linearity of the secant modulus is a typical feature of geomaterials.
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Figure 3-32 Drained triaxial test results of the cemented carbonate sediments at Pinjar: (a) deviator stress vs. axial strain relationship; (b) volumetric strain vs. axial strain data.

Figure 3-33 demonstrates the normalised stress-strain relations. In this event, the deviator stress is normalised by the peak stress and the axial strain is normalised by a quantity $\varepsilon_{50}$, in which the developed stress equals to 50% of the peak stress. This occurs even when the...
scattered stress-strain curves for different mean stresses closely merge into a backbone curve. An appropriate single relationship cannot be obtained through regression analyses of these non-dimensional stress-strain data due to some false axial strains recorded at the initial shearing portions.

Figure 3-33 Dimensionless relationships of principal stress difference vs. axial strain: (b) complete curve; (b) initial portion curve of response
The strength parameters can be derived using the MC model criteria, as was used for the artificial weakly-cemented sands. The peak strength parameters, $c'$ and $\phi'$, determined from the drained triaxial compressive tests on ‘hard’ well-cemented lumps of weak rocks are typically 270 kPa and 43°, as shown in Figure 3-34.

Figure 3-34 Peak strength parameters cohesion and friction angle of Pinjar weak rock

Figure 3-35 shows the ratio of deviator stress to effective mean stress, $q/p'$, with axial strain. The stress ratios at peak state decrease with mean stress level. The final stress ratios vary significantly, owing to rupture planes that developed in all of the specimens. However, the $q/p'$ ratio of specimen C2 under lower mean stress approaches an apparent final constant of 1.60, which is equivalent to a friction angle ($\phi'_{\text{crit}}$) of 39°, showing a complete loss of cementation and a high friction angle typical of carbonate rich angular particles.
This critical friction angle is compatible with the reported friction angle of 37° for cemented material by Coop and Atkinson (1993), for the naturally cemented carbonate soils in the North West Shelf of Australia.

It is noteworthy that the equivalent UCS values derived from triaxial tests [assuming $q_{\text{ucs}} = 2c \tan(45° + \varphi'/2)$ for $\sigma'_1 = \tan^2(45° + \varphi'/2)\sigma'_3 + 2c \tan(45° + \varphi'/2)$] are consistent with the mean UCS values measured. In addition, the $c'$ value measured in the triaxial tests is consistent with that of the $q_{\text{ucs}}$ values.

3.4 Summary

In-situ soil characterization and laboratory tests have been conducted for the artificial weakly cemented sands and the in-situ material from the Pinjar test site. CPTs and SCPTs provide continuous profiles of penetration soundings for the subsurface soils from the cone (for which the measured resistance corresponding to the limiting pressure at failure states). The laboratory tests facilitate the determination of selected parameters of interest, such as the effective cohesion and friction angle.
Very weakly-cemented sands, that is, weak $c$-$\varphi$ soils, are relatively homogenous (owing to the sample preparation procedures). Compared to these very weak $c$-$\varphi$ soils, in-situ strong $c$-$\varphi$ soils display a very high degree of variability in cementation and density (i.e. a heterogeneous nature) and anisotropic due to the presence of bedding planes (laminations). With a higher degree of cementation, the cone tip resistances of hard $c$-$\varphi$ soils are significantly higher than those of very weakly cemented sands. The same is true in the case of the small strain shear modulus.

The test results of weakly-cemented sands indicate that peak strength and stiffness increase with cement content, as well as the confining stress. It was also found that the weakly cemented sands show a transitional response from brittle failure to ductile failure as confining stresses increase, whereas the well-cemented sands and weak rocks exhibit brittle failure behaviors at all confinements (within the range investigated).

The weakly-cemented sands, artificially prepared with cement contents of 2.5% and 5%, have peak cohesions of approximately 12 kPa and 30 kPa respectively, and peak frictions range from $37^\circ$ to $39^\circ$. The un-cemented sand has similar peak friction angle. At large strains, both cemented samples and un-cemented sample approach an ultimate state with a critical state friction angle of between $32^\circ$ and $34^\circ$.

For the natural well-cemented samples of weak rocks, the peak cohesion is revealed in the range from 70 kPa to 270 kPa and the friction angle $43^\circ$. At large strain, the weak rock sample approaches an ultimate state, with the critical friction angle determined as $39^\circ$ at low mean stress.
CHAPTER 4 FULL-SCALE LATERAL PILE TEST AND ANALYSIS

4.1 Introduction

One straightforward method to establish reliable load transfer relationships for particular soil and rock model is through a field load testing program. There have been numerical and analytical approaches to derive such relationships. However, there will always be some uncertainties and assumptions made in the process of derivation. In particular, insufficient knowledge of the strength characteristics of the heavily cemented sands and weak rocks (strong $c$-$
\phi$ soils) can complicate these approaches. Load testing programs produce direct measurements of load-displacement responses under particular conditions. The experimental data provide a sound basis on which reliable relationships can be deduced, against which the analytical and numerical models can also be assessed and calibrated. In this chapter, the field experiment and its corresponding results are presented.

Site characterization which is a vital part of the experiment has been described in CHAPTER 3. This chapter first presents the pile description including the pile geometry, instrumentation and pile installation. The test set-up, the testing program and the corresponding field experiment results will then be discussed. Next, based on the bending moment data gathered from strain measures and inclinometer data, the experimental load-displacement transfer relationship, i.e. $P$-$y$ curves, were derived for Pinjar material. A non-linear trigonometry function was tentatively formulated for the $P$-$y$ curves for the in-situ soils at Pinjar.

4.2 Design of lateral load test

The test site was selected within a quarry site in Pinjar, about 25 km North–West of Perth, WA. This site features hard calcareous deposits of sandy soils and weak rocks formed by extreme variable cementation. These deposits closely match the soil types in question and were deemed a representation of such soil types. The site’s geotechnical
condition was defined and described extensively in section 3.3.

Figure 4-1 shows the plan for the four test piles together with the locations of the in-situ CPT and SCPTs. Each pair of piles of identical diameters was constructed centre-to-centre at 1.5 m apart; and the two different diameter pairs were 3 m apart. A typical ground profile was observed clearly from the wall of the test pit, displaying its weak rock characteristics coupled with un-cemented bands of sandy soils.

The lateral load tests developed in three phases: (1) pile instrumentation and pile installation, (2) on-site lateral pile calibration testing, and (3) lateral load tests.

4.2.1. Pile geometry and instrumentation

This full-scale pile experiment test was conducted on two pairs of drilled and grouted (D&G) piles at different diameters in very weak rock profiles. The two pairs of D&G test piles were 340 mm and 450 mm in diameter, 5 m in length, and labelled 340A, 340B, 450A and 450B. Each of these D&G piles was comprised of a steel pipe with grout core inside the pipe and grout in the annulus between the pile and soil/rock. A typical cross-section of the D&G piles is illustrated in Figure 4-3b. The length of the steel pipe and grout were extended to the full length along with the pile. The outer
diameter and wall thickness of the steel pipe was 219 mm and 5 mm for the 340 mm shafts, and 356 mm and 6.4 mm for the 450 mm shafts. The steel pipes were cut from standard Grade C350 Pipe (AS1163) in circular hollow sections. Table 2-1 tabulates the geometry details of the D&G shafts.

<table>
<thead>
<tr>
<th>Shaft No.</th>
<th>Diameter $D$ (mm)</th>
<th>Outer tube diameter$^a$ $OD$ (mm)</th>
<th>Tube thickness$^b$ $t$ (mm)</th>
<th>Length $L$ (m)</th>
<th>Embedment depth $L_e$ (m)</th>
<th>$L_e / D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>340A, B</td>
<td>340</td>
<td>219</td>
<td>5</td>
<td>5</td>
<td>4</td>
<td>11.8</td>
</tr>
<tr>
<td>450A, B</td>
<td>450</td>
<td>356</td>
<td>6.4</td>
<td>5</td>
<td>4</td>
<td>8.9</td>
</tr>
</tbody>
</table>

$^a$ Outer diameter of steel pipe.  
$^b$ Wall thickness of steel pipe.

Electrical-resistance strain gauges were attached diametrically opposite to the outer wall of the four steel pipes, along their full length. Under horizontal loading, the two sides correspond to tension and compression zones. The strain gauges on the outer wall of the steel pipe were used to measure the bending moment. The location of the strain gauges is demonstrated in the schematic of the test setup in Figure 4-3d. A standard inclinometer access tube, 85 mm in diameter (see Figure 4-3c) supplied by Duham Geoslope, was attached to the centerline of each steel pipe for inclinometer probe access to measure the rotation of pile along the pile length. The diametrically arranged strain gauges and the internal grooves of the inclinometer casing (uppermost $A$ Groove) should be aligned in the same plane (in line) along the direction of the soil-pile movement. The inclinometer tube provides access for the inclinometer probe, allowing it to obtain subsurface measurements, and deforms with the pile so that inclination measurements of the casing accurately represent pile movements. Linear variable differential transducers (LVDT) with a resolution of $10^{-2} \text{ mm}$ were used to measure the motion of the pile head for each pile.

The piles were constructed using a CFA process by the piling company, Belpile Pty Ltd. These $D&G$ piles were constructed using the CFA technique and post-boring installation of the tube into the (grouted) bore, as shown in Figure 4-2. Whilst the auger was drilled into the ground, the flights of the auger were filled with soils, providing lateral support and maintaining the stability of the hole. At the same time, when the auger was
withdrawing from the hole, standard Portland cement grout was placed inside by pumping the grout through the hollow centre of the to the base of the auger. Simultaneously pumping of the grout and withdrawing the auger provided continuous support to the hole. After auger extraction, the steel pipe was inserted in the centre of the grout-filled hole. The steel pipe and cement grout was then left for curing.

Figure 4-2 Installation of drilled and grouted pile: drilling a hole for grouted pile (left), and installing the tube and filling with grout (right)

The compressive strength of cement grout is about 35 MPa at age of 20 days. The steel pipe is standard Grade C350 Pipe (AS1163) with a minimum yield strength of 350 MPa.

21 days following the pile construction, the ground around the piles was excavated to a depth of 1 m to expose the piles 1 m above the soil surface. During excavation, the soil material was logged and sampled for some laboratory tests typically the index tests, such as the PSD test, point load test, and carbonate content test. This formed a test pit measuring approximately 5 m × 5 m. The pit facilitated the application of the lateral loads at the required level of eccentricity above the ground surface. It also eliminated the influence of the very variable upper metres of the calcareous deposits on the test results. Thus, the effective embedment depth of each shaft is 4 m, resulting in the embedment depth-to-diameter ratios of 11.8 and 8.9 for two diameters.
4.2.2. Experimental setup and pile calibration test

The conventional method for conducting a lateral load test was employed. This involves pushing or pulling the pile head of one test pile against a reaction pile (ASTM D3966), using two piles and applying the load so that both piles were tested simultaneously. A load cell was used to measure the applied lateral load and a displacement transducer attached to a reference beam was used to monitor lateral deformation (Reese 1984; Reese and Van Impe 2001).

Details of the experiment setup are shown in Figure 4-3. One pair of identical diameter shafts was simultaneously horizontally loaded using a jack. The load level was measured using two separate systems: the load cell device and the hydraulic pressure device. The hydraulic pressure was monitored by a gauge and a pressure transducer. Two load cells with a capacity of 300 kN and 1,000 kN (SPX) were used for tests on 340-mm and 450-mm diameter shafts. At a level close to load application on the shaft, a reference beam was set and a dial gauge measured the top lateral movements when each shaft was loaded transversely.

Data from various instruments were collected through a high-speed data acquisition system (DAQ) with the computer software Labview to acquire and manipulate data during the test. The system was housed inside a mobile field testing laboratory on the site. It consists of a signal conditioner of Signal Conditioning Extensions for Instrumentation, and a DAQ board to convert the conditioned analog into a corresponding digit number with maximum scan rate of 100 kHz. Through this DAQ system, the raw transducer signals of load cell and linear transducer were recorded and changed into a standardized voltage output.

Pile deformation during lateral loading tests was measured using an inclinometer survey. The inclinometer survey comprise four components: (1) the inclinometer casing, (2) inclinometer probe, (3) control cable, and (4) digital tilt readout unit. The grooved inclinometer probe tracks the longitudinal grooves in the casing. It consists of two force-balanced servo-accelerometers, measuring the tilt in the plane of the inclinometer wheels (A axis) and a plane perpendicular to that of the wheels (B axis), as schematically shown in Figure 4-3c. A control cable is used to control the depth of the
inclinometer probe, metric control cables are graduated at intervals of 0.5 m. The digital tilt readout unit displays the measurement of inclination obtained from the inclinometer probe. Digitilt DataMate II is used to store the readings from the inclinometer survey. Output from the probe is directly proportional to the sine of the angle of inclination of the long axis along the centreline of the pile length. The differences in subsequent readings with the initial base reading provide angular data logger at the corresponding depth. The inclinometer measures angular movement (tilt) rather than lateral movement. Data reduction from the tilt measure to lateral movement uses standard trigonometry. A profile of the lateral deflection with the depth of the pile for each load increment is then obtained.

The inclinometer survey was carried out tracking the A₀ and A₁₈₀ casing grooves in the plane of the inclinometer wheels along the loading direction, refer to Figure 4-3c. No tracking measurement of the B₀ and B₁₈₀ casing grooves was made for shear movement. The inclinometer casing of the 450-mm diameter shafts unexpectedly buckled and no tilt surveys were made of this pair of piles. Surveys for the 340-mm pair shafts ended when a load level of 130 kN was reached. The inclinometer data are presented in section 4.3.3.

Prior to the lateral load testing program, trial lateral pile tests were conducted for both pairs of piles. Low load levels were applied to obtain the moment–curvature relationships of the composite piles. Eccentric loads were applied at 0.9 m above ground level from the four piles, i.e. \( e = 0.9 \) m. Strain information collected from the strain gauges (see Figure 4-3d) above the soil surface was converted into bending moments and exercise calibration factors were derived. The tension/compression strain is taken as the average value of measured tension and compression strains at diametrical sides of the steel pipe, \( \varepsilon = (|\varepsilon_t| + |\varepsilon_c|)/2 \), where \( \varepsilon_t \) and \( \varepsilon_c \) are the tension and compression strains. The bending moments \( (M) \) associated with the specific incremental load level are calculated as a product of the applied load (force), multiplied by the distance of the strain gauge to the load point \( (M = H \times e) \). Curvature, \( \kappa \), is determined from expression: \( \kappa = \varepsilon / y \), where the deformation \( y = D_{outer}/2 \) (radius \( r \)) of steel pipe. Deduced curvature-moment \( \kappa–M \) relationships are presented in section 4.5.1.
Figure 4-3 Layout of the pile test: (a) a photo of pile calibration test setup, (b) schematic cross sections of test pile, (c) cross section of casing tube, and (d) elevation view of test setup.
Due to the lack of confinement imposed by soil rock mass, the cement grout covering above the soil surface was easily fractured during calibration phase of testing when a maximum load of 40 kN for 340-mm piles and 140 kN for 450-mm piles was applied and had to be removed. Consequently, the cross-section area for the upper portion of the pile above the soil surface comprised a steel pipe and grout core. Upon the lateral load removed thoroughly, permanent displacement at the pile head \((e = 0.9 \text{ m})\) was registered as 1.6 mm and 1.5 mm for 340A and 340B piles, and 1.5 mm and 1.4 mm for piles 450A and 450B, see Figure 4-4a and Figure 4-5a. The difference of section property (flexural rigidity \(EI\)) between the upper portion and the portion below the soil surface is considered negligible.

4.2.3. Testing program and testing procedure

The lateral load experiment program consists of static loading tests for 340-mm diameter and 450-mm diameters piles and an episode of cyclic loading test at small cycles for 340-mm diameter piles. Information about the testing program is provided in Table 4-2.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Pile</th>
<th>Embedment (L_e/D)</th>
<th>Load eccentricity (e/D)</th>
<th>Lateral loading</th>
<th>Inclinometer survey (Y/N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calibration test C1</td>
<td>340A, B</td>
<td>12</td>
<td>2.7</td>
<td>Static</td>
<td>N</td>
</tr>
<tr>
<td>Calibration test C2</td>
<td>450A, B</td>
<td>9</td>
<td>2</td>
<td>static</td>
<td>N</td>
</tr>
<tr>
<td>Load test L1</td>
<td>340A, B</td>
<td>12</td>
<td>0.9</td>
<td>Static &amp; cyclic</td>
<td>Y</td>
</tr>
<tr>
<td>Load test L2</td>
<td>450A, B</td>
<td>9</td>
<td>0.4</td>
<td>Static</td>
<td>N</td>
</tr>
</tbody>
</table>

The testing procedure for the two pairs of piles was essentially the same. Standard testing procedures were followed. The static load test was performed to obtain the load-displacement data so as to develop \(P-y\) curves for each pile. Removal of the grout cover for the upper portion of the pile resulted in some reduction in the pile rigidity. The load points were lowered close to the soil surface to minimize the influence \(e = 0.3 \text{ m}\) for 340-mm piles and \(e = 0.2 \text{ m}\) for 450-mm piles. The loading procedure was conducted in accordance with standard ASTM recommendations. First, one test pile was pushed against another reaction pile of the same diameter. Next, the load was
maintained for approximately 10 minutes to allow the pile displacement to stabilize before the next loading step took place. Then, the next load increment was applied by 20 kN and the same procedure was repeated.

During the static loading test for 340-mm piles at a load of 130 kN, a cyclic loading test was performed to study strength degradation of the soil-pile system under an increase in the number of load cycles. Lateral loading was cycled one-way six times between loads of 130 kN and 15 kN. Static loading was then resumed; the displacement would continue to develop with further increments in the load. Gradually, the load on the pile was lessoned by a similar fraction to the maximum load. At each unloading step, the load was maintained for 10 minutes and then unloaded to zero.

4.3 Experimental results

Table 4-3 summarizes that data of some typical results for the lateral load tests. For the pair of 340-mm piles, displacements were recorded at 8.4 mm and 23.1 mm for a maximum load of 160 kN. Displacements continued to increase to 8.8 mm and 49.6 mm at load with no further increase of load. Upon test completion, permanent displacements were registered at 2.6 mm and 34.2 mm for the 340A and 340B piles, respectively. For the 450-mm pair piles, displacements were recorded at 11.5 mm and 40 mm at a maximum load of 480 kN. Displacements continued to increase to 13.8 mm and 66.9 mm at load with no further increase. When the piles were unloaded to zero and upon completion of the load test, permanent displacements were registered as 3.9 mm and 48.7 mm for the 450A and 450B piles, respectively. Regarding the strain responses during lateral loading, the maximum strain levels for both type B piles went over 2040 με and 2012 με, beyond which some strain gauges lost their workability. The maximum strain levels for piles of type A were 1279 με and 887 με.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Shaft No.</th>
<th>Load eccentricity e (m)</th>
<th>Load H (kN)</th>
<th>Displacement yh (mm)</th>
<th>yh (mm)</th>
<th>Strain ε (10^-6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1</td>
<td>340A</td>
<td>0.3</td>
<td>160</td>
<td>8.4</td>
<td>2.6</td>
<td>1279</td>
</tr>
<tr>
<td>L1</td>
<td>340B</td>
<td>0.3</td>
<td>160</td>
<td>23.1</td>
<td>34.2</td>
<td>2040</td>
</tr>
<tr>
<td>L2</td>
<td>450A</td>
<td>0.2</td>
<td>480</td>
<td>11.5</td>
<td>3.9</td>
<td>887</td>
</tr>
<tr>
<td>L2</td>
<td>450B</td>
<td>0.2</td>
<td>480</td>
<td>40</td>
<td>48.7</td>
<td>2012</td>
</tr>
</tbody>
</table>
In this section, the results of individual lateral load test for pile 340A, 340B, 450A and 450B are presented. $H-y_h$ graphs of load-displacement responses under lateral loading are first presented. Following these, the $\mu\varepsilon-H$ response for strain variations data for all levels under varying loads is detailed. Finally, the distributions of pile deflexion for the 340-mm piles are presented.

4.3.1. General load displacement response

Pile 340A was tested under both static and cyclic loads using 340B as the reaction pile. Figure 4-4 depicts the load-displacement responses of the calibration and load tests for piles 340A and 340B, including episode responses to cyclic loading through $H-y_h$ graphs. As is visible from the curves, the pile stiffness for the calibration test ($e = 0.9$ m) is lower than that of the load test ($e = 0.3$ m). This is due to a larger deflection resulted from a relatively larger moment loading, in addition to shear force loading. Comparing the $H-y_h$ graphs displayed in Figure 4-4a, the lateral stiffness of pile 340A is shown stiffer than that for pile 340B. The deflection at load point of pile 340B is approximately ten times that of 340A, 23.1 mm and 8.4 mm at a maximum load of 160 kN.

The six cycles of one-way cyclic lateral loads have a small influence on the pile head displacement of pile 340A, whilst the cyclic loading causes an increase of total deflection of pile 340B, as shown in Figure 4-4b. The first cycle causes significant degradation than the next other cycles.

Figure 4-4c presents the complete lateral load-displacement curves for 340 mm piles. The yield displacement was estimated as 24 mm for pile 340B at a lateral load close to 160 kN, whilst no yield displacement indicated for pile 340A. Similar to the observation in pile calibration test, pile 340B deflected more than 340A under same loads. Figure 4-4d presents the lateral response simply showing the static loading.

Figure 4-5 shows the load-displacement responses of the calibration and load tests for pile 450A and 450B. Coincidentally, the tests yield a similar soil-pile response: a stiffer pile 450A and a softer pile 450B. The maximum displacements measured are 11.5 mm and 40 mm when the load reaches a maximum of 480 kN. Unlike the 340-mm-diameter piles, elastic (linear) behaviour of the 450-mm diameter piles is demonstrated until a
load of 300 kN which is more than half of the maximum (yield) load.
Figure 4-4 Load-displacement transfer relationships $H-y_h$ graphs for 340-mm piles: (a) pile calibration test, $e = 0.9$ m; (b) complete load-displacement curves of lateral load test, $e = 0.3$ m; (c) episode of cyclic loading; and (d) lateral load test simply showing the responses under static loading.
By comparison, the maximum loads applied on 450-mm piles were significantly increased as an increase in diameter but the same embedment depth, yet slightly decreased $L_e/D$ ratio.

Figure 4-5 Load-displacement transfer relationships $H-y_h$ graph of piles 450A and 450B: (a) pile calibration test, $e = 0.9$ m; (b) lateral load test, $e = 0.2$ m
4.3.2. Strain gauge data

The strain $\varepsilon = \frac{\left| \varepsilon_t \right| + \left| \varepsilon_c \right|}{2}$ responses for loading at various levels along the pile length for all four piles are illustrated in Figure 4-6 and Figure 4-7. All strain gauges worked during the loading and unloading stages except for the gauge SG-0.3 m of 450B (a gauge at 0.3 m below soil surface), which went beyond its capacity of 2,012 με and failed to resume its workability, see Figure 4-7b. The strain for the gauge SG-0.5 m of 340B went as high as 2,040 με. This gauge resumed its workability upon unloading.

![Figure 4-6](image-url)

Figure 4-6 Strain variations over lateral loading, με-H graphs: (a) pile 340A and (b) pile 340B (The gauges at lower levels do not show significant responses and are omitted.)
For each test pile at Pinjar site, only two or three of six levels of strain gauges of these four piles responded to the incremental loads. Very little change was observed for more than half of the gauges, implying that there was very little ground pressure or zero
pressure on the pile at those levels. For the 340-mm diameter piles below depths of approximately 1.0–1.3 m from the soil surface, virtually zero strain responses occurred. For 450-mm-diameter piles, approximately 1.5–1.7 m from the soil surface was the defining or critical boundary depth for an effective soil rock pressure distribution range. In other words, the soil rock mass within these boundary depths made a crucial contribution to resistance to the lateral loads, whilst the soil rock mass below these depths made a small contribution or no contribution to the resistance. Therefore, the strain data reveal that these piles behaved as flexible pile (long pile) which the lateral stresses develop along an effective length of pile. This effective length is defined as a length of a long flexible pile where the bending moment, pile deformation, shear force and soil/rock pressure gradually decay to zero (Randolph 1984).

Nevertheless, the location of the maximum moment ($M_{\text{max}}$) could not be determined precisely from the strain data due to insufficient gauges arranged along the pile length, the determination of $M_{\text{max}}$ will be discussed in section 4.5.

4.3.3. Inclinometer data

Inclinometer data were measured along the full length of the 340A and 340B piles during the lateral load tests. However, no inclinometer measurement was made for 450 A and 450B piles due to the unexpected failure of the inclinometer access tubes in both piles.

Figure 4-8 shows the profiles of lateral displacement vs. depth for 340-mm piles with load increments reaching 130 kN. The deflection data demonstrate that most of the lateral displacement occurred at a depth within the upper 1 m of pile embedment. This distribution pattern of rotation (deflection) and depth is consistent with the observation of the strain data. It can be inferred that the 340-mm pair piles behave as long flexible piles and the effective length is identified to be approximately 1–1.5 m. Despite the absence of displacement profiles for 450-mm diameter piles, based on the strain observation it is certain that the 450-mm pair piles also behave as long flexible piles, with an effective depth of approximately 1.5–1.7 m.
Chapter 4 Full-scale lateral pile test and analysis

4.4 Other observations from field experiment

As identified in the previous chapter, the deformation and bending moment induced by lateral loads were confined only to the upper portion of the pile and the overall length of the pile doesn’t exhibit significantly impact on the lateral response of the pile. These observations suggest that the full-scale test piles have a plastic hinge formation, i.e. the lateral behavior of long flexible piles.

The grout cover started cracking from the top portion of piles above ground surface during the calibration test at a horizontal load of 40 kN for 340-mm piles and 140 kN for 450-mm piles. These loads are relatively low and indicate a very low tensile strength of the grout cover. The B piles consistently behaved softer than the A piles in this experiment. Inspection on completion of testing indicated that, in the cemented soil, a significant gap between the soil/grout and steel pipe of pile occurred on the active side of the piles and cracks emanated radially from the pile along the soil surface on the passive side. Figure 4-9 shows the observed tensile cracks and gap between grout and...
steel pipe of piles 340B and 450B. The cracks consistently occur in the grout cover and then the soil interfaced in a plane perpendicular to the loading direction and more cracks develop, radially emanating to the soil and weak rock at some distance. The onset of such cracks, however, cannot be precisely determined as a function of the applied lateral load. As for A piles, similar cracks were also observed but at relatively small size, indicating the A piles are stiffer than B piles generally or the strength difference of soil surrounding the piles.

Figure 4-9 Observed cracks for 340 mm pile (left) and 450 mm pile (right)

The evolution of cracking observed implies a failure mechanism of the soil/pile system. Figure 4-10 illustrates the propagation of soil/grout cracking. The observed failure mode can simply be described in three stages:

(a) Under certain load, the soil/pile deforms, forming a gap between the back face of steel pipe and soil and (an approximately 90°) crack that develops in grout and then the ground, as shown in Figure 4-10a. At this stage, the flexural stiffness is considered to degrade but does not necessarily fail since the pile system sustains greater loads after the initial crack appears.

(b) As the load increases, the soil starts cracking due to tensile failure (as well as stress concentration) at two opposite sides of piles along loading direction; simultaneously, a series of small cracks in the front side of pile develops due to the compressive pressure exhibited by the displacement of soil/pile (Figure 4-10b). At considerable deformation facilitated by the cracking, the soil resistance of compressive side is considered to be mobilised to its ultimate strength at depths.
(c) As the size of crack increases, the soil/pile is reaching its maximum load capacity, which can be assessed from the load-displacement curve where flattens off, as demonstrated in Figure 4-10c.

![Figure 4-10 Sketch of soil/pile failure mechanism for shallow depth](image)

This is consistent with the observations by Faro *et al.* (2015) but distinguishes in that the noticed soil cracks in the present study initiate in the grout of pile and propagate in the direction of approximately 90° to the loading. The fan of soil wedge, (i.e. the mobilised effective stress friction angle) of the assumed passive soil-wedge failure model by Liang *et al.* (2009) and ‘rock chippers’ of theoretical analysis by Erbrich (2004) may develop to be larger than 45° to the loading direction.

Upon completion of the load test, the ground next to passive side of each pile was excavated to investigate the cracking patterns along the pile length, and to locate the plastic hinge elevations of the piles. Figure 4-11 displays the photos displaying the profile of excavated soil and pile upon test completion, 340B pile on the left. The ground soil surrounding pile 340B was the easiest to dig, indicating a relatively soft soil mass in that location. From the small-sized cracked pieces of grout, it can be estimated that the crack size decreases with depth below the ground surface. The cracks become insignificant at a depth greater than approximately 1 m and 1.5 m for the two types of diameter piles. The inclination degrees measured for 340B and 450B were approximately 2.1° and 2.3°, respectively.
In this section, the development of the load transfer relationship of the strong cemented soils at Pinjar test site using the results from full-scale experiment is presented. An assessment of inelastic moment-curvature relationships is first provided for estimating the bending moment diagram along the pile that is required for the derivation of soil resistance. This is followed by the process of back-calculation of the pressure-displacement $P-y$ curves based on the results from static lateral load testing. Furthermore, the derived $P-y$ curves of Pinjar material are presented.

4.5.1. Determination of moment-curvature relationship

The theoretical elastic yield moments, $M_{p-340}$ and $M_{p-450}$, were calculated to be approximately 67 kNm and 228 kNm. The plastic moments, $M_p-340$ and $M_p-450$, were calculated to be approximately 90 kNm and 300 kNm for piles with diameters of 340 mm and 450 mm, respectively. Table A-1 in APPENDIX A gives the details of the pile geometry and pile property.

For small bending strains, the moment of the pile can be computed according to the following expression:

$$M = EI\kappa = EI \frac{\varepsilon}{r} \quad (4.1)$$
where $EI$ is the flexural rigidity of the pile, $\kappa$ is the curvature of the cross-section ($\kappa = \frac{\varepsilon}{r}$), $\varepsilon$ is bending strain and $r$ is the radius of the steel pipe of pile. The maximum elastic strain, for which the relationship in Equation (4.1) applies, was identified up to approximately 1600 $\mu\varepsilon$ for the full-scale test piles. Beyond this strain level, the pile cracks in tension and the stress-strain behavior of the pile became non-linear, causing a reduction in the Young’s modulus, shifting the neutral axis of the pile, and hence leading to a degradation in the pile flexural rigidity. Therefore, a non-linear moment and curvature relationship is required to compute the bending moment.

The relation of moment and curvature still holds as $M = V\kappa$, in which the curvature $\kappa$ is still given as $\kappa = \frac{\varepsilon}{r}$, but the pile rigidity would no longer be constant and is replaced with a variable $V$. The non-linear $M-\kappa$ relation had to be developed for the bending moment to be appropriately determined.

The non-linear $M$ vs. $\kappa$ relations for piles with two different diameters were estimated using Oasys-Adsec (AdSec 2010), a program for non-linear analyses of sections under multiple loads. The cross-section of the composite pile was assigned with three concentric sections: grout core, steel tube and grout cover. The grout and steel pipe material were modelled as elastic-perfectly plastic material, with an initial slope equal to their Young’s modulus and a constant stress once yielded.

Figure 4-12 and Figure 4-13 present the moment–curvature relationships estimated by Oasys-Adsec along the measured data points at the initial portion of the curve for small strains. There is adequate agreement between the Oasys-Adsec estimates and the measured data points of moment and curvature at the initial portion of the estimated curve for the piles with two different diameters.

The slope of the moment-curvature curve is simply the flexural rigidity of the composite pile which due to the cracking of the pile at a large curvature (strain). The flexural rigidities (i.e. the initial slope of $M-\kappa$) before the onset of the cracking were calculated to be approximately 7,200 kNm$^2$ (340-mm pile) and 31,000 kNm$^2$ (450-mm pile). The capacities of the elastic yield moment ($M_y$) and plastic moment ($M_p$) calculated by Oasys-Adsec are coherent with the theoretical capacities. This suggests that the
non-linear moment–curvature relationships are capable of providing sound moment–
curvature predictions for the piles.

Figure 4-12 AdSec estimated moment-curvature relationship and measured data: (a) complete \( \kappa \)-M curve, and (b) initial portion of \( \kappa \)-M curve, 340 mm piles
Figure 4-13 AdSec estimated moment-curvature relationship and measured data: (a) complete $\kappa$-$M$ curve, and (b) initial portion $\kappa$-$M$ curve, 450-mm piles
4.5.2. Method of back-calculation of P-y curves

Derivation of the experimental P-y curves using results data from fully instrumented pile test generally involves mathematical procedures. First, the bending moment profile along pile length needs to be established by using curve-fitting technique. Then, the soil resistance per unit length on the pile may be deduced by using double differentiation process of the bending moment data.

A continuous function for the bending moment of depth

With the bending moment data obtained from the strain gauge measures, basic beam theory was used to back-calculate the lateral soil resistance per unit length along the pile, \( p \), and associated soil–pile displacement, \( y \), for obtaining experimental load transfer P-y curves (Reese and Matlock 1956). The soil resistance and soil–pile displacement at any point of embedment depth can be derived according to the following equations:

\[
p = \frac{d^2 M}{dz^2} \quad (4.2)
\]

\[
y = \int \frac{M}{EI} dz \quad (4.3)
\]

It should be stressed that the Equation (4.3) is not applicable to calculating lateral displacement for piles with variations in pile flexural rigidity. It is only workable for elastic piles. In the Pinjar field test, the lateral displacement data was obtained from an inclinometer survey (for pair of 340-mm piles). Hence, the measured displacement data can be conveniently used for obtaining load transfer relationships.

To apply the Equation (4.2) for deducing the soil resistance, it is necessary to first establish a continuous relation of the bending moment versus embedded depth, based on the experimental moment from strain measures. The bending moments induced at the strain gauge levels of each test pile were estimated using the \( M-\kappa \) relationships on Figure 4-12 and Figure 4-13. Numerical approaches are required to determine a continuous function for discrete data points.
A solving method of non-linear curve fitting, employing a combined function of an exponent function and a polynomial, was adopted for this study. A simple least-squares method was used to evaluate the errors between the fitted moments using Equation (4.4) and measured moments \( M_i \). Best estimate moment profiles with depth \( z \) were then fitted to these data using the following relationship form:

\[
M = \left(1 - \frac{1}{e^{a_i z} + 1}\right) \sum_{i=1}^{n} a_i z^{i-1}
\]  

(4.4)

where the \( a_i \) are the coefficients. The polynomial function has been frequently used in curve fitting (Ashford and Juinnarongrit 2005; Dou and Byrne 1996; Doyle et al. 2004; Kong and Zhang 2007; Matlock and Ripperger 1956; Yang 2006). An exponential function is incorporated to effectively model the decay phenomena of moment, shear force and/or deflection over the embedded depth of pile. Such lateral behavior is typical observed for a long flexible pile. Equation (4.4), a product of a polynomial function multiplied by the exponential function has been successfully applied in interpretation of the bending moment and pile rotation data (Phillips 2002). The details of the data process for curve fitting of the moment data as well as the description of the least-squares method for curve fitting are provided in APPENDIX D.

**Moment profiles of the test piles**

The best fit moment profiles along the measured data for the two diameters are shown in Figure 4-14 and Figure 4-15. Although measurement was achieved by different strain gauge levels on each individual pile, the \( M \) profiles should be treated as approximate due to the shortage of strain gauge levels in the upper metres of soil; this is because depth plays a crucial role in resistance to the application of horizontal loads. It can be clearly seen that the depths of the maximum bending moments gently migrate downward with an increase in loading for all piles.
Figure 4-14 Moment profile of piles: (a) 340A; and (b) 340B
Chapter 4 Full-scale lateral pile test and analysis

As shown in Figure 4-14a and Figure 4-14b, the critical depth of zero moment for pile 340A (approximately 1.3±0.2 m) is smaller than that of pile 340B (about 1.5 m). These differences may be attributed to the local variations in weak rock (soil mass) and/or the minor differences in the stiffness of piles. Under a lateral load of 130 kN, the peak moment induced at approximately 61 kNm and 70 kNm are generally compatible with the theoretical yield moment \( M_y \) of 67 kNm. This indicates that both piles may not yet yield at loads less than 130 kN. The depth of the peak moment is observed at approximately 0.3 m and 0.4 m (i.e. the ratio of this depth of peak moment to embedment depth \( z_{max,M/L_e} = 0.08 \sim 0.10 \)). The downwards migration of plastic zones for soft pile 340B at loads of 140 kN and 160 kN should be treated with care due to the considerable deflections (severe cracking) occurring at a large strain, therefore making the fitted moment diagram less reliable.

For 450-mm piles, the critical depth of zero moment for pile 450A (1.5± m) is observably smaller than that of pile 450B (about 2 m), as shown in Figure 4-15. The depths of peak moments are found to be approximately 0.4 m and 0.5 m, with corresponding moments of approximately 201 kNm and 238 kNm for the stiffer and softer piles respectively. The ratio of this depth of peak moment to embedment depth, \( z_{max,M/L_e} \) ranges from 0.10 to 0.13. The depth of zero moment of pile 450A (2± m) is smaller than that of pile 450B (approximately 2.2 m). The pile 450A was in its elasticity throughout the loading phase, whilst pile 450B fell into plasticity at a load of 450 kN.

The elastic continuum method and subgrade reaction method were examined for predicting the bending moment diagrams for two diameters (Hetényi 1946; Poulos 1971). Based on the results of the on-site pile calibration, the back-calculated Young’s modulus of cement grout was back-calculated to be approximately 5.5±0.5 GPa. This value is close to the values reported for similar materials (Li 2007; Luff 2007). The equivalent modulus of pile elasticity was calculated to be approximately 11.4 GPa and 13.2 GPa, according to the relation \( E_p = (EI)_{p}/(\pi D^4/64) \), for the piles of 340-mm pair and 450-mm pair. The values of soil and weak rock modulus are the BE \( E_s \) values discussed in section 6.2. The modulus of subgrade reaction were calculated using the formulation of Vesic (1961).
Figure 4-15 presents the comparison of predicted moment distributions by these methods with the best-fit moment diagram of the 340-mm piles under working loads of 40 kN and 80 kN from the field test data. The agreement between these is reasonable for...
the 340-mm piles under working loads of 40 kN and 80 kN. For the selected loads, the solutions of the subgrade reaction modulus yield consistently larger moments and greater depth than those given by the elastic continuum method.

Figure 4-16 Moment comparison of the existing elastic solutions and experimental for 340-mm piles under horizontal loads of: (a) 40 kN; and (b) 80 kN
Figure 4-17 presents the comparison of predicted moment distributions by these methods with the best-fit moment diagram of the 450-mm piles under working loads of 100 kN and 200 kN from the field test data. The moments given by Poulos elastic continuum method are consistently smaller than the best-fit of experimental data, while the moments predicted below peak moment depths by the subgrade reaction method are considerably larger than the experimental values.

For the selected loads, the solutions of the subgrade reaction modulus consistently yield larger moments and greater effective depth than those given by the elastic continuum method. Furthermore, the distinguish between the derived diagrams from curve fitting of moment data and the predictions from the elastic continuum methods grow larger with the increase in horizontal load. This is demonstrated by the curves presented in Figure 4-17b, which are in contrast to those deviations indicated by the curves in Figure 4-17a.
Figure 4-17 Moment comparison of the existing elastic solutions and experimental for 450-mm pair piles under horizontal loads of: (a) 100kN; and (b) 200 kN
4.5.3. Derived P-y curves

The shear forces \((SF)\) and soil resistances \((p)\) along the pile length were calculated by differentiating and twice differentiating the moment data \((M)\) with respect to the depth \((z)\). The methodology of deducing soil resistance is described in APPENDIX D. The lateral soil-pile displacements \(y\), can be obtained directly from the measures of inclinometer, see section 4.3.3. The soil resistance \(p\) force per unit length in the \(p-y\) curves is replaced by transverse pressure \(P\), which is given by \(P = p/D\). Relating the soil lateral pressures \(P \ (p/D)\) to the associated soil-pile displacements \(y\), the characteristic \(P-y\) curves of cemented calcareous sand/weak rock were developed for various depths. Sensitivity analyses on the fits achieved to the moment data suggested that the estimated pressures were accurate to within about 15%.

Figure 4-18 presents the \(P-y\) curves deduced using static load testing results data for 340-mm piles. The \(P-y\) curves obtained were limited by depth near the ground surface because the pressures developed to resist the lateral loads were confined within a relatively small, effective length for the present piles. Experimental \(P-y\) curves for 450-mm piles were not developed as the data of soil-pile lateral movement were unavailable.

As can be seen in Figure 4-18a and Figure 4-18b, the maximum lateral pressures on pile 340A and pile 340B at dimensionless depths \(z/D\) of 0.5 and 0.75 (0.17 m and 0.23 m) are shown approximately 2,400 kPa and 1,500 kPa, corresponding lateral displacements of 1.90 mm and 2.77 mm. For the 450-mm piles, the maximum pressures are observed to be 3,600 kPa and 2,600 kPa at depths of 0.25 m and 0.3 m for the stiffer pile A and softer pile B. The deduced maximum pressures may reasonably be regarded as the yield pressure \((P_u)\) of the cemented calcareous sand and weak rock. For a long flexible pile, whilst the overall failure mechanism is governed predominantly by a pile structural yield, before the lateral load reaches its real ultimate value, relatively high pressures are developed near the soil-pile top portion. The soils and weak rocks are likely to yield at even relatively low loads and consequently increased displacements will occur.
Figure 4-18 Characteristic $P$-$y$ curves of Pinjar materials: (a) 340A; and (b) 340B
Inspecting the maximum lateral pressure \( P_{\text{max}} \) in the P-y curves for a number of depths and the measured cone resistance \( q_c \) at that depth, the \( P_{\text{max}} \) values are found approximately 13% - 20% of the \( q_c \) values for the same depths. So the \( P_u \) values of the soils near ground level can be determined falling in the range between 13% and 20% of the \( q_c \). With the values of limit pressure \( P_u \) determined, the transverse pressure \( P \) may be normalised by the \( P_u \), and the corresponding lateral displacement \( y \) normalised by pile diameter \( D \). Thus, dimensionless P-y curves is formed, as shown in Figure 4-19.

![Figure 4-19 Dimensionless P-y data of Pinjar material based on the results data of 340 mm piles and the regression curve simply showing the portion response from the origin till the peak pressure](image)

It is noted that the normalized P-y data in Figure 4-19 exclude the pressures data beyond the peak values of derived lateral pressure. However, the non-dimensional P-y response of pile 340A appears to be slightly softer than that of 340B, this may be attributed to the accuracy of data processing of the moment data.

Based on the non-dimensional P-y data for very shallow depths near the ground surface \( (z < 0.5D) \), a best-fit P-y response for the 340A and 340B piles was obtained through non-linear regression analyses. The best-fit P-y relationship from regression analyses
was a tangent hyperbolic equation and written as

\[
\frac{P}{P_u} = \tanh \left[ 60 \left( \frac{v}{D} \right)^{0.75} \right]
\]  \hspace{1cm} (4.5)

Figure 4-19 also includes the P-y curve that calculated using the regression model of Equation (4.5). It is evident the calculated P-y curve agrees well with the derived values if neglecting the pressures data after peak values.

4.6 Summary

The field experimental results including lateral load tests and inclinometer survey have been presented in this chapter. The observations indicate that the lateral pile behavior depends on the strengths of soil rock, the pile rigidity, and the geometry of soil/pile system as well.

Comparison of load-displacement results of 340-mm and 450-mm piles suggests that both the pile stiffness and the pile capacity increase as the pile diameter increases.

The strain profiles and the pile deflection profiles measured (excluding 450 mm diameter) reveal the test piles consistently behave as long flexible piles. For the two varying diameters, the critical depths of zero soil reaction (zero soil displacement) revealed from strain data were identified to be approximately 4D.

The non-linear flexural stiffness of pile is crucially important to determine the moment-curvature relationship of pile, as well as the subsequent derivation of soil reaction. A best-fit P-y response of weak rock material was deduced on basis of the experimental results of Pinjar field test (340 mm piles). However, the fitted P-y curve flattens off at the limiting pressure \( P_u \), ignoring the portion curve of strain softening.
CHAPTER 5 CENTRIFUGE-SCALE MODEL PILE TEST AND INTERPRETATION

5.1. Introduction

Full-scale pile tests are expensive and time-consuming. Centrifuge modelling using reduced-scale model tests are economical alternatives to solve geotechnical problems; the behavior of a foundation can be investigated in a soil specimen with known parameters. A geotechnical centrifuge is typically used to test physical models for geotechnical problems such as the strength, stiffness and capacity of foundations. Centrifuge modelling is a physical modelling technique in which the weight stresses of a prototype are simulated by the placement of a small-scale model in a centrifugal field.

There have been extensive centrifuge tests on lateral piles using sand (i.e. soils with frictional strength components $\varphi$) (Alderlieste 2011; Bienen et al. 2011; Dyson and Randolph 2001; Klinkvort 2012; Kong and Zhang 2007; Scott 1980; Wesselink et al. 1988) Examples of centrifuge tests in clay (i.e. soils with cohesive strengths $c'$) have also been reported (Brant and Ling 2007; Doyle et al. 2004; Guo et al. 2014; Jeanjean 2009; Zhang et al. 2010). However, there are few investigations on laterally loaded pile in cohesive and frictional soil samples ($c$-$\varphi$ soils), using either a full-scale field test or reduced-scale model experiments.

This study extends the investigation of responses of lateral loaded piles in sand and clay to $c$-$\varphi$ soils through centrifuge modelling. The modelled $c$-$\varphi$ soils were artificial very weakly cemented sands. Reduced-scale model piles were designed as short, single piles with slenderness ratios of 5 and 10 and strong section property. Strain gauges, however, were not attached to the model piles. Three types of soil samples were reconstituted with various content levels of Portland cement by dry weight. The geotechnical characteristics of the artificial cemented sands including uncemented sand have been presented in section 3.2 of CHAPTER 3.

This chapter describes the centrifuge modelling of laterally loaded piles in very weakly cemented sands. A general overview of centrifuge testing for solving geotechnical
problems is first described. Following this, the testing programs, testing procedure and the results are presented. Lastly, the lateral load-displacement responses of the model piles in various soil conditions are generalised via non-linear regression analyses of the measurement data.

5.2. Geotechnical centrifuge at UWA

Schofield (1980) played a major role in the development of modern geotechnical centrifuge modeling. The history and basic principles of the small-scale model, and centrifuge modeling have been reviewed by Joseph et al. (1988). More details in centrifuge technology were covered by Taylor (1995). Information about centrifuge modelling will not be replicated here. Nonetheless, it is helpful to briefly present the scaling law related to the present study of laterally loaded pile. Following this, information about the centrifuge facility at UWA is provided.

If the reduced-scale (1/N) model and full-scale prototype are made of materials with identical mechanical properties, then the stresses (σ) and strains (ε) in the model and prototype will also be identical (e.g. σ_p = σ_m, ε_p = ε_m). The subscripts ‘m’ and ‘p’ stand for model and prototype. The scaling factors for the quantities such as the pile length and pile diameter, flexural rigidity, bending moment, are listed in Table 5-1.

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Prototype</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear dimension and displacement</td>
<td>1</td>
<td>1/N</td>
</tr>
<tr>
<td>Area</td>
<td>1</td>
<td>1/N^2</td>
</tr>
<tr>
<td>Volume</td>
<td>1</td>
<td>1/N^3</td>
</tr>
<tr>
<td>Mass density</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Stress, and strain</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Force</td>
<td>1</td>
<td>1/N^2</td>
</tr>
<tr>
<td>Pile flexural rigidity</td>
<td>1</td>
<td>1/N^4</td>
</tr>
<tr>
<td>Bending moment</td>
<td>1</td>
<td>1/N^3</td>
</tr>
<tr>
<td>Time (for inertial force similarity)</td>
<td>1</td>
<td>1/N</td>
</tr>
</tbody>
</table>
Chapter 5 Centrifuge-scale pile test and interpretation

The geotechnical centrifuge used for this study was the geotechnical drum centrifuge at UWA; this facility has been described in detail by Stewart et al. (1998). Above ground level, this well-balanced drum centrifuge shown in Figure 5-1 is composed of three major units: a drum channel, multi-axis actuator system (VHM axis motion), and outer casing. Dynaservo motor and clutch components are built underground below the machine. Concentric shafts connect the two sections above ground and underground. Some additions to the drum include wireless data acquisition with software (DigiDAQ) and a high-speed data capture system. The drum channel is 1.2 m in diameter and can hold a soil area of 300 mm wide by 200 mm deep. The maximum test acceleration is approximately 300 g. The rate of vertical and horizontal movement for the multi-axis actuator system is about 3 mm/sec with a maximum payload of 10 kN. The circumferential speed can reach 540 degrees/sec, which is equivalent to the capability of a continuous 300 Nm. The drum centrifuge specifications are summarized in Table 5-2.

![Figure 5-1 A photo of drum centrifuge at UWA with no outer casing in place](image)

The key feature of the drum centrifuge is a central tool table that may be stopped or started independently from the soil channel. This is achieved with two concentric shafts.
Operational work on the central tool table can be performed without stopping the channel, such as when checking or modifying models.

<table>
<thead>
<tr>
<th>Table 5-2 Drum centrifuge specification</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Property</strong></td>
</tr>
<tr>
<td>Outer diameter</td>
</tr>
<tr>
<td>Inner diameter</td>
</tr>
<tr>
<td>Maximum acceleration</td>
</tr>
<tr>
<td>Maximum acceleration</td>
</tr>
<tr>
<td>VH displacement of actuator</td>
</tr>
<tr>
<td>VH load of actuator</td>
</tr>
<tr>
<td>Circumferential displacement of actuator</td>
</tr>
<tr>
<td>Circumferential load of actuator</td>
</tr>
</tbody>
</table>

Typical centrifuge tests can be performed using either full sample testing with soil placed over 360 degrees within the drum channel, or by strongbox sample testing whereby the soil is contained in a strongbox mounted in the drum channel. If the strongbox is mounted in the drum channel asymmetrically, dead weights at the opposite side of the strongbox location are placed to counterbalance the system. In the present study, the centrifuge testing was performed using the small strongbox placed in the drum channel. The structural model can be installed either in-flight using specially fabricated installation parts at $N_g$ or installed at 1 g, depending on the specific requirements of the test.

Soil characterization is essential for centrifuge model tests for determining soil parameters relevant to models. Different in-flight miniature probes of different diameters are used such as a miniature cone penetrometer or miniature T-bar. Two main uses for such probes in centrifuge testing are: (i) to check the uniformity or repeatability of the sample and more importantly, (ii) to obtain some absolute measure of the continuous in-flight strength profile of the specimen (Bolton et al. 1999). In this study, a miniature CPT was used to characterize the soil samples (see section 3.2.3).
5.3. Design of model pile test

The centrifuge model experiment involves four broad phases of operation:

1. Preparation of soil sample, in which the model pile was pre-buried at 1 g;
2. Curing of soil sample for hydration to react at a duration of about 20 hours;
3. Soil characterization in-flight; and
4. Lateral load application on model pile in each soil sample.

In total, six load tests were designed for the centrifuge modelling, in which model piles with the same diameters but at embedment depths in three types of very weakly cemented soil samples were modelled including the un-cemented soil sample. Table 5-3 summarizes the experimental test program.

### Table 5-3 Testing program at acceleration level of 50 g

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Box No.</th>
<th>Model pile</th>
<th>Soil sample</th>
<th>Dry unit weight $\gamma$ (kN/m$^3$)</th>
<th>Gradient of $dq_c/dz$ (MPa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>CS0-S</td>
<td>5D</td>
<td>CS0</td>
<td>16.3</td>
<td>0.83</td>
</tr>
<tr>
<td>T2</td>
<td>CS0-L</td>
<td>10D</td>
<td>CS0</td>
<td>16.3</td>
<td></td>
</tr>
<tr>
<td>T3</td>
<td>CS2.5-S</td>
<td>5D</td>
<td>CS2.5</td>
<td>15.8</td>
<td>1.60</td>
</tr>
<tr>
<td>T4</td>
<td>CS2.5-L</td>
<td>10D</td>
<td>CS2.5</td>
<td>15.8</td>
<td></td>
</tr>
<tr>
<td>T5</td>
<td>CS5-S</td>
<td>5D</td>
<td>CS5</td>
<td>15.8</td>
<td>3.50</td>
</tr>
<tr>
<td>T6</td>
<td>CS5-L</td>
<td>10D</td>
<td>CS5</td>
<td>15.8</td>
<td></td>
</tr>
</tbody>
</table>

* Notation is described below.

5.3.1. Testing equipment

As shown in Figure 5-2a, two pairs of models with identical diameters at different lengths (a total of four models) were used. Table 5-4 summarizes the model specification. The diameter of all the model piles ($D$) was 10 mm and the embedment lengths ($L_e$) were 50 mm and 100 mm. The lengths of the upper parts above the soil surface were consistently 50 mm for all the model piles. When tested at 50 g, these samples would represent a prototype pile diameter $D = 0.5$ m, with embedded lengths $L_e = 2.5$ m ($5D$) and $L_e = 5$ m ($10D$), respectively, resulting in length-to-diameter ratios of...
5 and 10 \( (L_e/D = 5 \text{ and } 10) \). The model piles were made from solid steel that cut from a standard steel rod at a minimum cost and effort for fabrication.

![Model piles and lateral loading arm and in-line load cell](image)

Figure 5-2 Experiments on centrifuge-scale piles: (a) model piles; (b) lateral loading arm and in-line load cell

With a model pile diameter of 10 mm, the ratio of model diameter to soil particle diameter, \( D/D_{50} \), is approximately 55 \((10/0.19)\), refer to CHAPTER 3. The effect of particle size has been identified as the potential scaling issue in centrifuge modeling. Even though it is seldom possible to replicate all of the precise details of the prototype, a minimum ratio of 15 should be satisfied to avoid any inconsistency in test data (Taylor 1995). A model diameter-to-grain size diameter of 50 was recommended by Hoadley et al. (1981). However, an even higher ratio of 88 has been considered necessary for poorly graded soils by Klinkvort (2012). Hence, the \( D/D_{50} \) ratio of 55 for this centrifuge modeling program was considered within a satisfactory range.

Table 5-5 provides the details of the properties of model and prototype piles. In the case of the model pile and prototype pile where materials with either different mechanical properties or different cross-sections were used, equivalent dimensions and pile properties should be determined accordingly.

---

Table 5-5 provides the details of the properties of model and prototype piles. In the case of the model pile and prototype pile where materials with either different mechanical properties or different cross-sections were used, equivalent dimensions and pile properties should be determined accordingly.
Table 5-4 Model pile specification

<table>
<thead>
<tr>
<th>Model piles</th>
<th>Qty.</th>
<th>Diameter D (mm)</th>
<th>Length L (mm)</th>
<th>Embedment L_e (mm)</th>
<th>Slenderness L_e/D</th>
<th>Load eccentricity e (mm)</th>
<th>e/D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shorter pile</td>
<td>2</td>
<td>10</td>
<td>100</td>
<td>50</td>
<td>5</td>
<td>30</td>
<td>3</td>
</tr>
<tr>
<td>Longer pile</td>
<td>2</td>
<td>10</td>
<td>150</td>
<td>100</td>
<td>10</td>
<td>30</td>
<td>3</td>
</tr>
</tbody>
</table>

Table 5-5 Properties of model and prototype piles at 50 g

<table>
<thead>
<tr>
<th>Properties</th>
<th>Model</th>
<th>Dimension</th>
<th>Prototype</th>
<th>Dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile diameter, D</td>
<td>10 mm</td>
<td>mm</td>
<td>0.5 m</td>
<td>m</td>
</tr>
<tr>
<td>Pile length, L^1</td>
<td>50 mm</td>
<td>mm</td>
<td>2.5 m</td>
<td>m</td>
</tr>
<tr>
<td>Pile length, L^2</td>
<td>100 mm</td>
<td>mm</td>
<td>5 m</td>
<td>m</td>
</tr>
<tr>
<td>Ratio, L^1/D</td>
<td>5</td>
<td>-</td>
<td>5</td>
<td>-</td>
</tr>
<tr>
<td>Ratio, L^2/D</td>
<td>10</td>
<td>-</td>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td>Inertia, I</td>
<td>49 mm^4</td>
<td>mm^4</td>
<td>0.003 m^4</td>
<td>m^4</td>
</tr>
<tr>
<td>Pile modulus, E</td>
<td>200 GPa</td>
<td>GPa</td>
<td>200 GPa</td>
<td>GPa</td>
</tr>
<tr>
<td>Pile rigidity, EI</td>
<td>98.2 Nm^2</td>
<td>Nm^2</td>
<td>614 MNm^2</td>
<td>MNm^2</td>
</tr>
</tbody>
</table>

It is likely that a geotechnical failure, referred to as a short pile, will control the ultimate lateral load. A rigid pile tends to rotate around a point, thereby generating soil pressure over the total length of the pile. Thus, the failure mechanism will be governed predominantly by soil strength - that is, by a geotechnical failure rather than a structural failure of a slender pile.

The centrifuge-scale model piles were not instrumented with strain gauges to measure bending moment. The main reason for this omission was that there was insufficient time for the strain gauge work due to the packed schedule for the centrifuge device. However, the total lateral response of the single piles in cemented sand was successfully obtained.

A loading arm was designed to transversely load the pile (see Figure 5-2b). The loading arm was fabricated from solid steel rod, 165 mm in length and 14 mm in diameter. One tip of the loading arm was directly connected with the actuator on the tool table of the
drum centrifuge; the other tip was equipped with an in-line load cell that fitted at a right angle. A half-steel cylindrical part was attached at the end of load cell; the round surface of this half-cylinder part would enable a point contact to the model pile when the loading occurred. The lateral load was applied by pushing the model pile at a constant displacement rate when the loading arm vertically or circumferentially moved with the actuator. The flexibility of the loading arm had to be eliminated from the measurement of displacement.

5.3.2. Soil preparation with pile installation

Soil samples were prepared in three types:

(1) Un-cemented sand designated CS0;
(2) Very weakly-cemented sand with cement added at a content of 2.5% (by dry weight) designated CS2.5;
(3) Very weakly-cemented sand with cement added at a content of 5% (by dry weight) designated CS5.

As the first step, the pre-buried model piles were installed at 1 g in the soil sample contained in a strongbox during soil preparation. The strongbox was made of steel and had an internal dimension of 258 mm × 165 mm × 200 mm. Fabric cloth covering the inside bottom and two inside walls was placed as the drainage path for the saturated soil sample; appropriate anti-erosion measures by applying grease had to be done for protecting the inside surfaces of strongbox wall and base. Prior to dry pluviation for soil preparation, each single model pile was hung in the middle of an empty strongbox using a string passed through a tiny hole close to the pile head; the levels for the tiny hole from the pile head for all four models were the same to enable the models to achieve the required embedment. Thus, the piles were maintained precisely in a vertical orientation in the soil mixture when the pluviation was completed (see Figure 5-3).

This 1 g installation procedure simulates a non-displacement installation. Thus, the installation effect on the pile response is minimised and one can expect that the initial lateral stresses prior to cementation corresponded to $K_0$ conditions with $K_0=1-\sin\phi'$.
Effect of installation is considered important for the lateral response of laterally loaded piles.

Figure 5-3 (a) A photo of soil prepared in a strongbox with model pile pre-buried at 1 g; and (b) a schematic of typical setup of cement-sand mixture pluviation (unit in mm)
With the diameter of 10 mm for all small-scale model piles, the equivalent diameter ratio of approximately 24 achieved in the strongbox is considered a suitable boundary condition for tests on a non-slender pile, compared with some similar model tests (Moreno et al. 2011; Sorensen et al. 2015).

The soil specimens were prepared through dry pluviation of the mixture of the dry sand and cement into a strongbox using a sand pluviator device, as shown in Figure 5-3. The mixture of UWA superfine sand and grey cement was mechanically mixed at different ratios with a mixer and this process was thought to make the mixture sufficiently uniform. At the time of transferring the dry mixture into the sand chamber of the sand pluviator, a very slight amount of water was sprayed to the dry mixture (<1% by weight), which would not affect hydration afterwards. This dampening treatment aimed to avoid any loss of fine cement particles or any undesired aggregation of cement throughout the pluviation process, producing a homogenous soil sample (Clough et al. 1981; Lee et al. 2011; Lee et al. 2010; Lee et al. 2010; Puppala et al. 1993; Puppala et al. 1995; Rad and Tumay 1986).

The soil sample density is determined primarily by the height of the sand pluviation chamber to the strongbox base, and the rate of the movement of the sand chamber along the shutter. The size of the diffuser seam also influences the density. Trial pluviations were exercised to find the controlling parameters. The density (soil mass divided by its volume) achieved was 1.66 g/cm$^3$ for sample CS0 and 1.61 g/cm$^3$ for both CS2.5 and CS5, see section 3.2.2. The finishing soil depth was 120 mm for all three types of samples contained in the strongboxes and the embedment depths for the shorter and longer model piles were 50 mm and 100 mm respectively.

For all samples, water was added by gently pouring it on the surface along the four edges of the strongbox; it then permeated into the soil mixture following the assumed permeation direction of the edge to the middle of the sample volume. A drainage path of the soil sample was guaranteed with a particular-purpose cloth that securely placed along the internal walls and base of each strongbox prior to the pile installation and the pluviation process of sand-cement mixture. Great care was exercised during the water addition process. The moisture content was controlled at 16% for all soil types.
Chapter 5 Centrifuge-scale pile test and interpretation

After a curing period of approximately 20 hours at an indoor temperature of 23±3°C, the strongbox containing the soil and pre-buried single pile model was carefully mounted into the drum channel for testing. Two identical sample types in two separate strongboxes were used with pre-buried model piles at a depth of 5D and 10D. Four strongboxes containing two types of soil samples and four embedded single pile models were symmetrically arranged in the channel at four locations (see Figure 3-2a). It was assessed that this layout would not only keep the drum channel in balance, but also allow the tests to be performed in an efficient sequence.

5.3.3. Experimental program

Once the strongbox was mounted in the drum channel, the channel was ramped up to 50 g. Next, the drum channel was filled with water at 50 g; this took approximately one hour for the whole system to become stable and the soil samples were assumed to be fully saturated. Soil characterization was then performed prior to the lateral load tests to determine soil strengths (see section 3.2.3).

The final phase of the centrifuge experiment consisted of performing lateral load tests on single model piles in the saturated samples. The model piles were monotonically loaded in a free-headed condition, with loads applied consistently at 30 mm (\( e = 3D \)) above the surface for model piles. All the soils were submerged with free water about 10 mm above the soil surface. Displacement in the lateral loading phase was applied at a rate of 0.1 mm/s.

The transverse load was applied on the model pile by vertically moving the tool table, at a loading rate of 0.1 mm/s which corresponded to a normalized velocity \( V \) of about 0.02, \( V = vD/c_v \), where \( v \) = loading rate, \( c_v \) = coefficient of consolidation (approximately \( 6\times10^{-5}\) m²/s on average for all samples). This loading rate defines the test under drained conditions. The transition from partially to fully drained conditions was as low as 0.01 \([ V=0.01\sim30 \text{ (Finnie and Randolph 1994)}; \ V=0.1\sim10 \text{ (House et al. 2001)}]\).

Figure 5-4 demonstrates the model test set-up. The magnitude of the load was measured with an in-line load cell, which was integrated into the loading arm. The lateral
displacement at the point of load application was inferred from the actuator movement. All data were recorded with a DigiDAQ program. It should be noted that the recorded movement did not exactly correspond to the pile displacement due to the flexibility of the loading arm. This flexibility of the steel load arm was taken into account and corrections were made to the recorded data of the actuator to deduce the correct lateral displacement of the model pile.

Figure 5-4 Set-up of model test: (a) Photo of a load test; (b) Schematic of profile view of load test (unit in model dimension mm and N)
Chapter 5 Centrifuge-scale pile test and interpretation

Tests for sample CS0 ended at a pile head lateral movement of 2D. Tests for samples CS2.5 and CS5 were terminated at a pile head displacement of up to 5D, at which level the soil had experienced very large deformations. Soil collapse was observed, implying that a limiting state in the soil sample around the model pile was reached.

5.4. Experimental results

During the loading for the longer model pile in sample CS5, the load cell with a capacity of 350 N was placed with a load cell with a higher capacity of 1 kN. As a consequence, the time required for the replacement influenced the hydration degree more than had been anticipated. All the rest load test results were continuously recorded.

5.4.1. Lateral load–displacement results

The lateral load results are presented with model dimensions and prototype dimensions. Figure 5-5 shows the load-displacement results of the lateral load tests for both $L_e = 5D$ and $L_e = 10D$ model piles in samples CS0, CS2.5 and CS5. Figure 5-6 shows the corresponding results in prototype scales. Figure 5-7 shows the combined load-displacement curves in two groups of shorter piles and longer piles. The lateral load at the pile head is denoted as $H$ and the pile head displacement denoted as $y_h$. A peak value on the $H$-$y_h$ curves is seen in tests on shorter piles in samples CS2.5 and CS5. However, the remaining tests did not indicate a peak lateral capacity.

For the $L_e = 5D$, the general lateral responses for sample CS0 were relatively ductile, the displacement increased with the load and no peak load was indicated. The maximum load applied on the pile in sample CS0 was 32 N in the model dimension, or 150 kN in the prototype dimension, with a corresponding large displacement of 2.3D at pile head. The shorter model pile tests for cement-sand samples CS2.5 and CS5 reached their peak loads at a movement of $y_h = 0.45D$ at the pile head. The loads then gradually dropped as the displacement continued. The peak values were 60 N and 76 N in the model dimension, or 150 kN and 190 kN in the prototype dimension. As expected, increasing the cement content resulted in an increase in lateral resistance to the loads. Some jagged
sections appear on the $H$-$y_h$ curves, presumably indicating the breakage of inter-particle bonding through cementation.

Compared with the deflection-load responses for shorter piles, the loads applied on the piles with $L_e = 10D$ at the same displacement levels were consistently higher than those of shorter piles. The improved lateral response of the longer piles is clear on inspection of Figure 5-5 to Figure 5-7. Significant increases in stiffness can be found with the longer piles as well.
Figure 5-5 Response of centrifuge model piles to lateral loading for three types of soil samples in model dimensions, $D=10$ mm and $e=3D$
Figure 5-6 Response of centrifuge model piles to lateral loading for three types of soils: (a) CS0; (b) CS2.5; and (c) CS5 (in prototype dimensions, $D = 500$ mm and $e = 3D$)
Figure 5-7 Combined load-displacement responses of prototype piles at embedded lengths: (a) $L_e = 5D$; (b) $L_e = 10D$ (in prototype dimensions)
5.4.2. Normalized lateral load-displacement results

The results of the experiment in dimensionless form are presented in Figure 5-8. The use of non-dimensional variables makes it possible to represent a wide range of real conditions by means of a single relationship. To form these non-dimensional relationships, the lateral load and corresponding displacement are divided by appropriate quantities. The quantities used for normalizing load $H$ are the pile diameter $D$, the embedment length $L_e$, and the gradient $q_c' = dq_c/dz$ of the cone tip resistance measured; the lateral displacement at pile head, $y_h$, is normalized by pile diameter $D$. Though the tip resistance $q_c$ in the profiles of cemented samples CS2.5 and CS5 approached a constant value at depths of approximately 3.5 m, the $q_c$ values over most of the pile embedment depths were revealed to be proportional to the depth at different gradients. It is therefore considered to be rational to use the gradient resistance rather than the constant resistance as a quantity for normalization.

For the embedments of $10D$, the normalised lateral load, $H/(D^3 q_c')$, with normalised displacement, $y_h/D$, is virtually unique and independent of cement content. This represents a characteristic lateral response for a range of soil samples with various cement contents.

The lateral load tests indicate that the cement content significantly contributes to pile capacity. The test results also show that the cement content contributed to initial stiffness. The relative uniqueness of normalized load-displacement curves for the lateral responses of model piles in all soil samples demonstrate the suitability of the CPT cone resistance for characterising the lateral response. The cone tip resistance $q_c$ is demonstrated to be not only a simple parameter measuring the relative strength in soil characterization but also a useful normalizing tool to relate soil properties across various soil conditions and over different depth ranges.
Figure 5-8 Normalized load-displacement responses of (a) $L_e=5D$; and (b) $L_e=10D$ piles
5.5. Other observations from model test

Besides the foregoing presented experimental results, some visualised observations provide supportive evidence for the soil-pile failure mechanism. Figure 5-9 shows photos upon completion of the load tests. For the un-cemented soil sample CS0, the soil close to the surface in front of the model pile was pushed up by excessive deformation; no gap formed behind the model pile under lateral loads. For the very weakly cemented soil samples CS2.5 and CS5, a gap between the soil behind the model pile was evident and this was due to their $c'$ component of strength.

As seen in Figure 5-9, for the $L_c=10D$ in CS2.5 and CS5, the boundary of soil area with cracks at the front side is generally perpendicular to the loading direction, suggesting that shear resistance at the sides of the piles is fully mobilized ahead of the compressive resistance. Meanwhile, from the distribution of the tensile cracks at the surface, the fan of an assumed soil wedge-type failure near the surface may develop as large as a semicircular area.
Figure 5-9 Observations of laterally loaded model piles in centrifuge soil samples of un-cemented sand CS0 and very weakly-cemented sands CS2.5 and CS5
Chapter 5 Centrifuge-scale pile test and interpretation

5.6. Analysis of centrifuge model piles test

As theoretically expected, the soil-pile displacements of these free-head model piles were observed as a result of pile rotation about certain depths below the soil surface for all model piles in the centrifuge test. The lateral capacities of these rigid piles will first be analyzed in the following section. Then the load-displacement responses are interpreted. A prototype dimension will be used for analysis of the centrifuge-scale model test in the following sections.

5.6.1. Ultimate lateral load capacity

The lateral load capacity, denoted as $H_u$, from the lateral load tests is determined as a load based on the excessive lateral displacement of the pile head. However, it is quite difficult to assess an ultimate lateral load or limiting load because the line of load-displacement transfer is mostly curved all the way. A point in the load-displacement curve (moment-displacement curve) where the curve becomes linear (flattens) or substantially linear is defined as the ultimate lateral load (Chari and Meyerhof 1983; Dickin and Laman 2003; Meyerhof et al. 1981; Petrasovits and Awad 1972; Prasad and Chari 1999). Beyond this point, deflection of the pile is more even with a marginal increase in the load. In the present study, identification of ultimate lateral loads will follow similar criteria to that described above if there is no obvious peak load indicated; if indicated, the peak loads will be regarded as the ultimate values. For instance, the shorter piles in samples CS2.5 and CS5 were mostly indicated by the peak loads.

For piles with a dimensionless embedment depth of 5 ($L_e/D = 5$), the ultimate lateral loads $H_u$ are identified as 56 kN, 150 kN and 190 kN in prototype scale, with corresponding dimensionless displacements ($y_h/D$) of approximately 0.96, 0.76 and 0.91. For longer piles at a dimensionless depth of 10, the $H_u$ values are 280 kN, 480 kN and 1,182 kN, respectively. These measured load values suggest that the ultimate lateral loads increase with embedment depth. The corresponding soil deformations also increase to mobilize more ultimate resistance. The observations are consistent with the features of the stress-strain behavior revealed in the drained triaxial tests. Table 5-6
summarizes observed ultimate lateral loads and displacements at pile head.

Table 5-6 Observed ultimate lateral loads and corresponding displacement

<table>
<thead>
<tr>
<th></th>
<th>$L_e/D = 5$ pile ($e = 3D$)</th>
<th>$L_e/D = 10$ pile ($e = 3D$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$H$ (kN)</td>
<td>$y_h/D$</td>
</tr>
<tr>
<td>CS0</td>
<td>56</td>
<td>0.9</td>
</tr>
<tr>
<td>CS2.5</td>
<td>150</td>
<td>0.6</td>
</tr>
<tr>
<td>CS5</td>
<td>223</td>
<td>0.8</td>
</tr>
</tbody>
</table>

5.6.2. Dimensionless lateral load-displacement response

The ultimate lateral capacity of these short rigid piles can be found as a function of the soil property in terms of the cone resistance slope $q_c'$, the pile embedded length $L_e$ and pile diameter $D$, as shown in Figure 5-10. Under the same loading eccentricities of $3D$, the lateral capacity is proportional to the product of cubic-pile diameter and gradient of cone tip resistance for each group of piles with same embedment depths. With increased embedded depth, the lateral capacities of the piles with embedment of $10D$ are nearly square more of those with embedment of $5D$.
Using the least-square method, regression analyses on the centrifuge test data, including the ultimate loads on pile, dimensionless depths of 5 and 10, and normalised quantities of pile diameter and cone resistance, yield a best-fit relationship that is expressed as:

\[ H_u = 0.0146 (D^3 q_c') \left( \frac{L_e}{D} \right)^{2.24} \]  \hspace{1cm} (5.1)

Figure 5-11 combines the load transfer relation measures for centrifuge piles. Based on the non-linear dimensionless lateral load-displacement responses presented in section 5.4.2, a single functional form can be employed to represent all normalised lateral-displacement relations for piles with dimensionless depths of 5 and 10.

Figure 5-11 Dimensionless lateral load-pile head displacement responses of centrifuge model piles at an eccentricity of \( e = 3D \)

A simple tangent hyperbolic function models was yielded from regression analyses on the normalized load-displacement data using the least-squares method. At a load eccentricity of \( e = 3D \), the characteristic relation was expressed as:

\[ \frac{H_o}{D^3 q_c'} = 4.04 \tan h \left[ 0.07 \left( \frac{L_e}{D} \right) \left( \frac{Y}{D} \right)^{0.56} \right] \]

\( R^2 = 0.9646 \)
\[ \frac{H}{D^3q_c} = 4.04 \tanh \left[ 0.07 \left( \frac{L_e}{D} \right) \left( \frac{y_h}{D} \right)^{0.56} \right] \]  \hspace{1cm} (5.2)

As presented in Figure 5-11, the calculated \( \frac{H}{(D^3q_c)} \), \( \frac{L_e}{D} \) and \( \frac{y_h}{D} \) values using Equation (5.2) were compared with the measured results data. This comparison reveals a reasonably good fit for both embedded depths of \( 5D \) and \( 10D \). If taking the dimensionless depth \( \frac{L_e}{D} \) equals to 10 or 5 and a displacement \( \frac{y_h}{D} \) where the ultimate loads is attained, Equation (5.2) will be reduced to an expression of Equation (5.1). This comparison also shows a reasonably good fit for both embedded depths of \( 5D \) and \( 10D \) piles.

From the dimensionless relationships of the lateral load-displacement responses, the in-situ cone resistance parameter \( q_c \) is again revealed as an important quantity for normalizing the various dimensional variables. A characteristic relationship between the non-dimensional variables can be conveniently developed, bringing together multiple divergent responses.

5.7. Summary

Six model tests in centrifuge-scale have been conducted in order to investigate the general soil-pile response in very weakly-cemented sands and un cemented sand as well. The model piles were laterally and monotonically loaded to considerable large displacement. Some conclusions can be drawn as follows:

The soil cementation has shown significant effect on the cone penetration resistance as well as the soil resistance under lateral loading. As soil cementation increases, the soil easily behaves as a material with pronounced brittle feature at relatively low stresses. For soil having a certain cementation, it tends to exhibit a stress strain behavior more ductile as the stress increases.

The normalized shear load transfer relationships, using the quantity of \( q_c \), can merge the multiple \( H-y \) curves into a characteristic load-displacement relation for the soils having varying cementations.
For the centrifuge scale model piles, i.e. short rigid piles, additional embedment depth enhances the initial soil-pile stiffness but also improves lateral capacity significantly, meanwhile decreasing soil-pile displacement.
CHAPTER 6  NUMERICAL ANALYSIS OF PINJAR FIELD TEST

6.1 Introduction

In this chapter, the in-situ CPT data for the Pinjar field test are analysed using the cavity expansion solutions methodology (CEM), which has been described in section 2.3. Operational strength and stiffness parameters for the in-situ material at Pinjar are theoretically assessed through the interpretation of the CPT cone data. Following this, simplified numerical simulations of the lateral load tests are performed using a finite element program to assess the representativeness of the operational parameters deduced from the CPT data.

6.2 CEM analysis of CPT data

Upper bound (UB), lower bound (LB), and a best estimate (or mean) (BE) Mohr-Coulomb (MC) strength and stiffness properties were assessed from UB, LB, and BE CPT $q_c$ data prediction using the procedure described in section 2.3 and the closed-form solutions developed by Yu and Houlsby (1991).

Based on the laboratory tests described in section 3.2, the values of $43^\circ$, $37^\circ$, and $40^\circ$ were assumed for $\phi'$ of UB, LB and BE. Applying the procedure described in section 2.3 gave the predictions shown in Figure 6-1, assuming the moduli $E$ given in Table 6-1. From the measured in-situ soil density, the overall soil was regarded as a relatively compressible material. Therefore, the dilation angle was not taken into account and a value of zero was set, $\psi = 0^\circ$. The $c'$ values are given in Table 6-1. These $E$ values are typically 5% to 15% of the small-strain values derived from the SCPTs but were selected as they were equivalent to typical $E_s/q_{ucs}$ values of 200 to 500 quoted for correlations (Goodman 1989).

It is evident that CEM-predicted $q_c$ profiles using both UB and LB parameter sets deviate away from the main-lines of the measured $q_c$ profile. The $q_c$ predictions using
UB values are around eight times those using LB values. As a compromise for the deviations predicted, a best estimate (BE) was made taking some intermediate values within the range. It should be noticeable that the $q_c$ predictions by the BE values are compatible with the measured $q_c$ values.

![Figure 6-1 Comparison of $q_c$, predictions and measures of Pinjar field material](image)

Table 6-1 MC parameters for CEM analysis of CPT data of Pinjar field test

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Lower bound (LB)</th>
<th>A best estimate (BE)</th>
<th>Upper bound (UB)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight, $\gamma$ (kN/m$^3$)</td>
<td>15.8</td>
<td>15.8</td>
<td>15.8</td>
</tr>
<tr>
<td>Cohesion, $c$ (kPa)</td>
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<td>160</td>
<td>270</td>
</tr>
<tr>
<td>Angle of friction, $\varphi$ (°)</td>
<td>37</td>
<td>40</td>
<td>43</td>
</tr>
<tr>
<td>Angle of dilation, $\psi$ (°)</td>
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<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Operative modulus, $E_s$ (MPa)</td>
<td>300</td>
<td>400</td>
<td>1000</td>
</tr>
<tr>
<td>Poisson’s ratio, $\nu$</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
</tr>
</tbody>
</table>

In the back-analysis of the CPT data of samples of field material, the ratio of the operative soil modulus $E_s$ of BE to the cone resistance $q_c$, $E_s/q_c$, was typically $12\pm3$. This is shown in in Figure 6-2.
Chapter 6 Numerical analysis of Pinjar field test

Figure 6-2 The $E_s/q_c$ ratio of Pinjar field material

Figure 6-3 The $E_s/q_{ucs}$ and $q_c/c'$ relationships for Pinjar material, compared with theoretical predictions, as well as empirical range
The values of ratio $E_s/q_{ucu}$ range 200 to 444 for Pinjar material and the corresponding value of ratio $q_c/c'$ is approximately 130 shown in Figure 6-3. This range is compatible with the theoretical predictions and reported empirical range (Goodman 1989). Figure 6-3 is derived assuming $\phi' = 40^\circ$ and $q_{ucu} = 2c'\tan(45^\circ + \phi'/2)$ at a normal stress of 100 kPa.

6.3 Finite element modeling of the lateral load test

Numerical experiments of three-dimensional finite element modelling (3D FEM) simulating the field load test were performed to investigate lateral responses. Numerical experiments followed three main steps for the investigation. First, the numerical model of piles were defined and calibrated with respect to the measured stress-strain response (refer to section 4.5.2) and the Oasys-Adsec predicted bending moment–curvature relationships. At this stage, input parameters for the properties of full-scale test piles were established for the numerical experiments, taking into account the cross-sections and the non-linear stress-strain behavior of the composite pile. The finite element modelling of the field load test was then performed, applying the soil and weak rock parameters assessed in the CPT analysis. Finally, the results of the numerical modelling were evaluated with respect to the measured values.

6.3.1. Numerical models with Plaxis 3D

Numerical simulations of the field test were performed using the commercial three-dimensional finite element analysis (3-D FEA) code Plaxis 3D 2011, which is a commercial 3-D FEM program used to perform deformation and stability analysis for various geotechnical applications (Brinkgreve et al. 2012). Plaxis 3D is a static, implicit, finite difference solver. To carry out the finite element computations, the geometry has to be meshed into elements. The soil volume in the Plaxis program is modelled through 10-node tetrahedral elements, which correspond to 6-node elements at each side of the tetrahedron. This is shown in Figure 6-4. In the present study, both the soil and pile were modelled using the 10-node tetrahedral elements. Interface elements are applied to model the interaction between the (pile) structure and the adjacent soil. This consists of 12-node elements and is based on the area elements formed by 6-node triangular
elements, which are compatible with 6-node soil and structural elements, as shown in Figure 6-5. The interface elements are formed by pairs (i.e. two nodes at each node position), one at the structure side and one at the soil side.

![Figure 6-4 Local numbering and position of nodes (*) and integration points (×) of a 10-node wedge element](image)

![Figure 6-5 Local numbering and position of nodes (*) and integration points (×) of a 6-node triangle element](image)

The associated strength and stiffness of the interface elements can be modified by a strength reduction factor, $R_{\text{inter}}$, which is a fraction of the strength of the adjacent soil effectively mobilized at the interface. In general, for real soil-pile interaction the interface is considered weaker and more flexible than the surrounding soil, which means that the value of $R_{\text{inter}}$ should be smaller than unity. An elastic-plastic model is used to describe the interface behavior for the modelling of the soil-structure interaction. The Coulomb criterion is used to distinguish between elastic interface behavior, where small displacements can occur within the interface, and plastic interface behavior, where plastic permanent slip may occur. Interface elements allow for differential
displacements between the node pairs.

To minimise the boundary effects on the predicted pile response, the size of the soil domain must be set sufficiently large. Considering computation time, a soil domain is set at approximately 15 m × 15 m (∼ 40 \(D\)) with a depth of 7.5 m (∼ 1.9 \(L_e\)), balancing the weight of boundary effects and time spent for computation. The bottom of the soil domain was restrained from any movement in all directions. The symmetrical vertical \(xz\) plane (or \(yz\) plane) is restrained from any movement in the \(z\)-direction. No displacement restraints were applied on the ground surface, allowing the soil to move freely. A finer mesh was used in the vicinity of the pile by defining a lower mesh factor in the pile elements. In total, there were 38,000 elements and 48,752 elements generated in the pile calibration test and lateral load test models, respectively.

The lateral load using a prescribed displacement approach rather than a direct load as calculation phases can be conveniently defined. Results of the pile response showing load-displacement development over successive lateral loading phases can be extracted directly from the output. Unlike a linear elastic pile, the induced bending moment for an inelastic pile cannot be calculated in a straightforward and direct manner.

The lateral displacement, \(y\), at a specific depth was calculated through averaging the nodal displacements around the circumference of the pile (in \(xy\) plan). The induced bending moment, \(M\), was deduced by extracting the bending strain data from the output and then reading a corresponding moment value from the established moment–curvature relationships (refer to Figure 6-8). This approach was used for the present numerical modelling. Alternatively, the moment can be calculated by numerically integrating the stress components over the cross-section of the pile, given by the formula

\[
M = \sum \sigma_{bi} A_i x_i
\]

where \(\sigma_{bi}\) is the bending stress for the \(i^{th}\) element, \(A_i\) is the area of the \(i^{th}\) element, and \(x_i\) is the coordinate (parallel to loading direction) of the centre of the \(i^{th}\) element.

6.3.2. Calibration of composite piles

The composite pile was installed in a sufficiently rigid medium with isotropic linear elastic properties and non-porous material, and various moments were applied to the top
of the pile, as demonstrated in Figure 6-6. Without any displacement or rotation at and below the fixity at ground level, the pile would behave like a cantilever under a horizontal load. The bending moment, $M$, increases linearly along the length of the column portion above ground level and simply equals the applied load times the distance from the load application point to the location of strain data extraction. The curvature $\kappa$ is constant and can be calculated as $\kappa = \frac{\varepsilon_b}{r} = \frac{\varepsilon_b}{0.5D_{\text{steel tube}}}$, where $\varepsilon_b$ is the bending strain, $r$ is the radius of the steel tube, and $D_{\text{steel tube}}$ is the outer diameter of steel tube. Bending strain, $\varepsilon_b (\varepsilon_{zz} \text{ in Plaxis 3D})$, is determined as the values of the strain on the opposite sides of the equivalent steel tube along the loading direction. $\varepsilon_b = \text{ABS}(\varepsilon_1 - \varepsilon_2)/2$, where $\varepsilon_1$ indicates tensile strain at the pile back (taken to be positive) and $\varepsilon_2$ indicates compressive strain (taken to be negative) at the pile front.

![Figure 6-6 Plaxis analysis of moment-curvature relationship (left) and a schematic of the mechanism (right)](image_url)

In the case of the full-scale lateral load test at Pinjar, the ultimate lateral capacity was predominantly governed by the yielding characteristics of the piles. When the induced moment was larger than the yield moment of the piles, the cracks initiated in the weaker material of the pile components and then propagated continuously as deflections increased. The elastic-perfectly plastic MC model was used to simulate the piles in FEM with Plaxis 3D rather than a linear elastic model. If a linear elastic model was used, its stiffness will be overestimated when the tensile strain is large enough to crack the grout.
and even the steel tube (Ardiaca 2009; Tand and Vipulanandan 2008; Voottipruex et al. 2011; Voottipruex et al. 2011).

A series of concentric elements forming the Pinjar piles were explicitly modelled in Plaxis 3D using the solid cylinders. Circular solid cylinders, which were defined by pile diameter, were specified for three individual pile components: the outermost grout cover (A), the steel tube (B), and the internal grout core (C), as sketched in Figure 6-7. To avoid mesh failure resulting from the very small detail of thin wall thickness of the steel tube compared to the large dimension of the pile length, the wall thickness dimensions were scaled up moderately to permit success in mesh generation, whilst keeping the steel pipes’ bending stiffness unchanged.

![Schematic of the numerical cross-section of test piles at Pinjar: (a) 340-mm; and (b) 450-mm](image)

A small modification was also made to the Young’s modulus of the grout because of its reduced thickness in the transformed section of the grout. At contact between the stiff steel tube and the grout cover, special attention should be given to those points where there is a high concentration of stress and strain. Interface elements may be assigned along the outside boundaries of the equivalent steel tube and grout cover to prevent such problems. At the composite pile-soil contact, interface elements were also assigned to
the outside boundary of the grout cover.

A Tresca criterion, a reduced MC criterion by setting the slope of the failure envelope \( \varphi' = 0^\circ \) and using one parameter as the intercept of the failure envelope \( c' \), was assumed for pile materials. The cohesive strength under uniaxial compressive strength, \( c \), was assumed to equate to its tensile strength, \( \sigma_y \). The parameters were first estimated and eventually determined through a trial-and-error approach until the stress-strain behavior yielded a response reasonably comparable to the Oasys-Adsec predicted \( M-\kappa \) relations (refer to section 4.5.2) and the in-situ measurements.

Table 6-2 summarizes the material parameters. The parameters for the grout cement and the equivalent steel tube are similar for the two diameters, except that the cohesive strength for 450-mm piles is slightly lower than that of the 340-mm piles. In view of a very smooth surface at the point of equivalent steel tube-grout cover contact, a value of 0.7 was defined for the \( R_{\text{inter}} \) of grout cover. It was unity for the values of the \( R_{\text{inter}} \) of grout core and the equivalent steel materials.

Table 6-2 MC pile parameters in Plaxis

<table>
<thead>
<tr>
<th>MC pile parameters</th>
<th>Cement grout</th>
<th>Equivalent steel tube</th>
<th>Actual steel tube</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight, ( \gamma ) (kN/m(^3))</td>
<td>24</td>
<td>78</td>
<td>78</td>
</tr>
<tr>
<td>Young’s modulus, ( E ) (GPa)</td>
<td>6</td>
<td>60</td>
<td>200</td>
</tr>
<tr>
<td>Cohesion, ( c ) (MPa)</td>
<td>2.1</td>
<td>100(^a)-110(^b)</td>
<td>-</td>
</tr>
<tr>
<td>Angle of friction, ( \varphi ) ((^o))</td>
<td>0</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>Angle of dilation, ( \psi ) ((^o))</td>
<td>0</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>Poisson’s ratio, ( \nu )</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>2.1</td>
<td>110(^a)-110(^b)</td>
<td>350-450</td>
</tr>
<tr>
<td>( R_{\text{inter}} )</td>
<td>0.7(^c)-1</td>
<td>1</td>
<td>-</td>
</tr>
</tbody>
</table>

\(^a\) Value for 450-mm piles.
\(^b\) Value for 340-mm piles.
\(^c\) \( R_{\text{inter}} =0.7 \) is for grout cover and \( R_{\text{inter}} =1 \) for grout core.

The numerically developed \( M-\kappa \) relationships for MC volume piles in Plaxis, together with the AdSec predictions, are shown in Figure 6-8. It is evident that the developed relationships are consistent with AdSec predictions (refer to section 4.5.2), suggesting that the appropriate parameters were determined for MC volume piles. A non-linear \( M-\kappa \) relation is indicated after maximum elastic strain, at \( \kappa = 0.0137 \) m\(^{-1}\) and 0.0019 m\(^{-1}\) for the 340-mm and 450-mm piles, respectively.
Figure 6-8 Moment-curvature relationships of Plaxis MC model piles of diameters: (a) 340-mm, and (b) 450-mm
6.3.3. Numerical modelling of field load test at Pinjar

Despite the clear non-linear nature of the Pinjar soil (see section 3.3), the FEA presented here sought to examine the potential of employing a simple linear elastic MC model, where the modulus of elasticity is obtained from the CPT data using the CEM approach described.

To reflect the test site conditions, the cluster of soil elements to a depth of 1 m adjacent to the piles was deactivated to simulate the excavated test pit. The composite piles were installed wished-in-place without considering the effects of pile installation. These results were expected to be suitable for the non-displacement piles.

Figure 6-9 depicts a vertical cross-section of the Plaxis 3D model of the lateral load test. The embedded lengths for two pairs of piles were 4 m. Lateral displacements were applied at 0.3 m and 0.2 m above ground line for the 340-mm and 450-mm diameter piles. Plaxis 3D FE models of the 340-mm and 450-mm piles were constructed for analyses of the field tests, without consideration of the stiffer pile $A$ and softer pile $B$ (see Figure 4-4 to Figure 4-8 in CHAPTER 4) i.e. the ground in the vicinity of each pair of piles had the same MC soil/weak rock parameters.

Figure 6-9 Plaxis 3D model for the Pinjar field test: (a) geometry boundary (left), and (b) plot of displacement vector of the pile under lateral loading (right)
The simple linear elastic MC model involves five input parameters: \( E \) and \( \nu \) for soil elasticity; \( \varphi \) and \( c' \) for strength; and \( \psi \) as an angle of dilatancy. The MC model is commonly implemented (recommended) as the first analysis for relatively fast computations. The values of the CPT back-analysed parameters of MC soil/weak rock properties were used as the input for the finite element modelling (i.e. the three defined properties of LB value, UB value and a representative BE value). The values of theoretical MC tensile strength were estimated to be 149 kPa, 70 kPa and 235 kPa for BE, LB and UB sets respectively. Tensile stress was allowed to develop by selecting tension cut-off and entering an appropriate value of tensile strength for Pinjar material throughout the Plaxis modelling. The soil and weak rock at pile-soil contact were at full strength, i.e. \( R_{\text{inter}} \) had a value of unity in all property-sets. Table 6-3 summarizes the soil and weak rock properties employed in Plaxis for the simulations of the Pinjar field test.

### Table 6-3 CPT back-analyzed parameters used as input for FEM with Plaxis

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Lower bound (LB)</th>
<th>A best estimate (BE)</th>
<th>Upper bound (UB)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight, ( \gamma ) (kN/m(^3))</td>
<td>15.8</td>
<td>15.8</td>
<td>15.8</td>
</tr>
<tr>
<td>Cohesion, ( c ) (kPa)</td>
<td>70</td>
<td>160</td>
<td>276</td>
</tr>
<tr>
<td>Angle of friction, ( \varphi ) (°)</td>
<td>37</td>
<td>40</td>
<td>43</td>
</tr>
<tr>
<td>Angle of dilation, ( \psi ) (°)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Operative modulus, ( E_s ) (MPa)</td>
<td>300</td>
<td>400</td>
<td>1000</td>
</tr>
<tr>
<td>Poisson’s ratio, ( \nu )</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>Tensile strength (kPa)</td>
<td>70</td>
<td>149</td>
<td>235</td>
</tr>
<tr>
<td>Strength reduction factor, ( R_{\text{inter}} )</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

6.3.4. Predicted lateral load-displacement response

Figure 6-10 presents the predicted lateral load-displacement responses at the pile head (i.e. \( H-y_h \) relations) and compares these with the experimental observations. The three sets of input parameters (LB, UB, and BE) yielded distinctive \( H-y_h \) responses under specified lateral loads (or displacements) for each diameter pile. As expected, the UB and LB stiffness and strength parameters produced the softest and stiffest lateral response respectively. The differences of \( H-y_h \) responses between three value-sets become more apparent as the lateral load (or displacement) increases.
Figure 6-10 Numerical $H-y_h$ responses with observations from field experiment: (a) 340-mm piles, $e = 0.3$ m and (b) 450-mm piles, $e = 0.2$ m
6.3.5. Predicted deflection profiles

The predicted profiles of displacement for the 340-mm piles using BE parameters are compared on Figure 6-11 with the measured profiles of lateral loads of 80 kN and 120 kN. At a horizontal load of 80 kN, the shape and magnitude of the Plaxis-predicted $y$ profile is consistent with that observed in both piles. At a higher load of 120 kN, the ground line deflection is approximately 18% smaller than the measured for pile 340A. This difference of ground line displacement may be because the FEA predictions are giving yield at a later stage to what observed at Pinjar test site. At both loads, the Plaxis 3D FEA predicted effective lengths where zero lateral displacement occurs agree well with the observed depths from in-situ measurements.

Unfortunately, no displacement profiles were measured for the 450-mm piles (see section 4.3.3) and as a result relevant examination of the predicted deflection profiles for these piles cannot be able to conduct.

Figure 6-11 Comparison of FEM estimated pile deflections and measures for 340-mm diameter piles under lateral loads of (a) 80 kN, and (b) 120 kN
6.3.6. Predicted bending moment profiles

Figure 6-12 and Figure 6-13 present the Plaxis predicted bending moment distributions with whole depth for 340-mm and 450-mm piles respectively, using BE parameters. For the 340-mm piles, numerical peak moments $M_{\text{max}}$ are approximately 30% smaller than the best-fit estimates, yet the depths of zero moment predicted in Plaxis 3D are compatible with the best-fit estimates at the selected loads of 80 kN and 120 kN.

![Figure 6-12 Comparison of FEM estimated bending moments and fitted measures of 340-mm piles under lateral loads of: (a) 80kN, and (b) 120 kN](image)

The predicted $M_{\text{max}}$ values for the 450-mm piles agree well with the estimate best fitted at a low lateral load of 200 kN but the difference becomes greater at a higher load of 355 kN. The numerical $M_{\text{max}}$ is approximately 28% smaller than the average $M_{\text{max}}$ of stiffer pile and softer pile. This greater difference may reflect the differences between predicted and actual moment-curvature relationships.

In summary, the FEM predictions using BE material set generally match observed average lateral load-displacement responses. Despite the underestimation of peak moment $M_{\text{max}}$, the effective depths of zero moment are consistent with the observations. Determination of appropriate input parameters for the MC volume pile is crucially
important to the FEM predictions. Reasonable agreement between the numerical responses and observations demonstrated that the soil properties theoretically assessed for the CPT data can be generally applied into a finite element analysis of laterally loaded piles.

![Figure 6-13 Comparison of FEM estimated bending moments and fitted measures of 450-mm piles under lateral loads of: (a) 200kN, and (b) 350 kN](image)

6.4 Summary

In summary, the FEM using value sets LB and BE generally yields lateral load – displacement responses that are compatible with the experimental measures at small strains. Despite the underestimation of peak moment $M_{\text{max}}$ predicted, the numerical effective depths of zero moment are consistent with the observations. Determination of input parameters for the MC volume pile in Plaxis appears to be crucially important for the FEM to produce comparable lateral responses to the measures.

Applying the soil parameters that back analysed for $q_c$ data into three dimensional finite analysis of load test, it basically yields pile responses generally close to the average of measures. This observation implies a feasibility of numerical predictions for the lateral behaviors using the soil parameters of cavity expansion-based CPT analysis.
CHAPTER 7  NUMERICAL ANALYSIS OF CENTRIFUGE SCALE PILE TEST

7.1. Introduction

The CPT data from the centrifuge model test were interpreted following the same procedures as those for the Pinjar field test (using the CEM). The soil properties assessed in the CPT data analysis are then applied to numerical finite element predictions of the lateral load tests. Importantly, in absence of bending moments measured for the centrifuge model piles, three-dimensional finite element modelling was necessary and implemented to aid interpretation of the un-instrumented piles.

7.2. Back-analysis of CPT data for centrifuge soils

For the properties of soil samples used in centrifuge model tests, the MC model parameters (i.e. strength cohesion \( c' \) and friction angle \( \phi' \)) used to predict the \( p_{\text{lim}} \) and \( q_c \) were based principally upon the results of the laboratory tests. The values of the dilation angle \( \psi \) were estimated according to the \( \phi' \) and \( \psi \) relation for sand, \( \phi' = \phi'_{\text{crit}} + 0.8 \psi \) \( (\text{Bolton 1986}) \). The \( \phi'_{\text{crit}} \) values were taken as approximately 32\(^{\circ}\)-34\(^{\circ}\) for all soil samples at an effective mean stress of 50 kPa (see section 3.2.6) and the \( \psi \) value were estimated to be approximately 7\(^{\circ}\) if taking the peak friction angle \( \phi' \) of 37\(^{\circ}\). Table 7-1 summarizes the values of the MC parameters determined through the soil laboratory tests (see section 3.2.6).

<table>
<thead>
<tr>
<th>Soil sample parameters of MC model for ( q_c ) prediction of centrifuge soils</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Soil sample</strong></td>
</tr>
<tr>
<td>Unit weight, ( g_{\text{sat}} ) (kN/m(^3))</td>
</tr>
<tr>
<td>Cohesion, ( c' ) (kPa)</td>
</tr>
<tr>
<td>Angle of friction, ( \phi' ) ((^{\circ}))</td>
</tr>
<tr>
<td>Angle of dilation, ( \psi ) ((^{\circ}))</td>
</tr>
</tbody>
</table>

The operational soil modulus \( E_s \) was determined through a trial-and-error procedure
after fixing the $c'$, $\phi'$ and $\psi$ values given in Table 7-1, to determine a best fit to the measured $q_c$ values. A LB and an UB material set was also examined in this trial-and-error process. CEM back-figured BEs of operational modulus for all soil samples are summarized in Table 7-2.

Table 7-2 Summary of best-estimated (BE) operative modulus $E_s$ for all soil samples

<table>
<thead>
<tr>
<th>Soil sample</th>
<th>CS0</th>
<th>CS2.5</th>
<th>CS5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground-line stiffness, $E_0$ (MPa)</td>
<td>0.25</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Stiffness, $E_1$ (MPa), for depth $z \leq 3$ m</td>
<td>$0.25 + 3z$</td>
<td>$0.5 + 6z$</td>
<td>$0.5 + 17z$</td>
</tr>
<tr>
<td>Stiffness, $E_1$ (MPa), for depth $z \geq 3$ m</td>
<td>$0.25 + 3z$</td>
<td>18.5</td>
<td>51.5</td>
</tr>
<tr>
<td>Poisson’s ratio, $\nu$</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
</tbody>
</table>

The predicted $q_c$ and measured $q_c$ profiles are compared in Figure 7-1. A good agreement between the CEM predictions and the measurements can be seen generally for the whole depths of all soils, even though the predicted $q_c$ values are slightly less than the measured below the depth of 3 m for soils CS2.5 and CS5.
Figure 7-1 Comparison of CEM predictions and measures of CPT $q_c$ for model soils: (a) CS0; (b) CS2.5; and (c) CS5.
With the operational modulus estimated from the CPT back-analysis, the ratio of the BE operative modulus to the cone end resistance was calculated and plotted on Figure 7-2. The $E/q_c$ ratios are variable at shallow depth but tend to be a constant of about 4 below a (prototype) depth at 2 m.

Figure 7-2 Variation of $E/s/q_c$ ratio of best-estimate modulus to cone penetration resistance of centrifuge soil samples

7.3. Finite element modelling of centrifuge-scale pile tests

As the centrifuge-scale model piles were not instrumented with electronic resistance strain gauges, data for the strains on piles used to derive the bending moments data, internal force diagrams and displacements were subsequently unavailable. For these purposes, 3D FEM with program Plaxis 3D were conducted to supplement the centrifuge model tests.
7.3.1. Plaxis 3D approach for centrifuge-scale piles

General procedures in the Plaxis 3D approach have been discussed in section 6.3.1. Particular procedures associated with numerical modelling of the centrifuge model tests will be described in this section.

The size and fineness of the mesh have significant effects on the finite element modelling. A finer mesh of an appropriate 3D model with a sufficiently large soil domain yield more accurate results, however, computational time is greater. For successful FE modelling, the fineness and determined model boundary are used in the critical sections. A cubic model geometry, with approximate dimensions of $13 \, \text{m} \times 8.2 \, \text{m} \times 6 \, \text{m}$ in prototype scale $(26D \times 16.2D \times 1.2L_e)$, was set for all FEM soil domains of the centrifuge model tests. The boundary conditions were globally meshed with fine density. Local refinements were made to the soils adjacent to the single pile in a circular area and further refinement was made to the pile’s volumetric elements. In total, approximately 30,381 and 31,358 elements were generated for $L_e/D = 5$ piles; 40,380 and 43,777 elements were generated for $L_e/D = 10$ piles in Plaxis 3D models.

![Figure 7-3 Plaxis 3D finite-element mesh for pile in very weakly-cemented sands: model geometry (left) and plot of displacement vector for the soil-pile system (right)](image-url)
Figure 7-3 presents a typical Plaxis 3D finite element mesh for soil condition of weakly-cemented sand (CS2.5 and CS5) and plot of displacement vector of the pile in the soil.

Modelling of pile

Unlike the volume pile using the MC model for full-scale piles in the field test, a linear elastic pile can be adopted for the centrifuge model piles. To record the data on internal forces and soil-pile deformation computed by Plaxis 3D code, a structural beam was employed in the models of the rigid pile. The basis for the beam element is a 3-node line element, which is compatible with the side of a 6-node triangle or a 10-node volume element. However, beam elements are slightly different from 3-node line elements in the sense that they have six degrees of freedom per node instead of three in the global coordinate system.

The test pile was modelled as a volume pile possessing volumetric elements. The properties of the volume pile were assigned with a material data set for soil using the linear elastic material property parameters of the real physical pile. In elasticity, the internal force diagrams can be computed and extracted by employing a weightless beam that is inserted into the pile along its length and assigned with a reduced Young’s modulus approximately ten times smaller than the stiffness of the pile \( E_{\text{beam}} = E_{\text{pile}}/10^{10} \). This ensures that the pile response in loading is not affected. Upon completion of computation, the beam would return the force components. The real diagrams of internal forces can then be obtained by multiplying the force components of the beam by the factor of \( 10^{10} \). Table 7-3 lists the properties of prototype pile and the inserted virtual beam.

Modelling of soil

The material behavior of soil was modeled using the classical MC model with tension cut-off by applying appropriate values of tensile strength to the cemented soils. The values of the MC model soil parameters used were identical to those assessed in back-analysis of the centrifuge CPT data. For soil condition CS0, the parameter \( E_{\text{increment}} \)
of the elasticity modulus gradient was applied for the whole embedment depth. For soil conditions CS2.5 and CS5, a double-layered stratigraphy consisting of soil with a soil modulus proportional to depth for the upper layer \( z \leq 3 \text{ m} \), and soil with constant elasticity for the lower layer \( z > 3 \text{ m} \) was adopted by defining the parameters of gradient \( E_{\text{increment}} \) and constant \( E_{\text{reference}} \) see Table 7-4. Tensile strength was considered for the MC soils CS2.5 and CS5 at a value of 10 kPa and 28 kPa respectively. For the FEM of centrifuge model test, all soil domains were submerged with water on the surface.

Table 7-3 Properties of prototype pile and inserted beam

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Name</th>
<th>Volume pile</th>
<th>Parameter</th>
<th>Name</th>
<th>Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material type</td>
<td>Model Type</td>
<td>Linear elastic</td>
<td>Cross-section area A (m²)</td>
<td>0.1963</td>
<td></td>
</tr>
<tr>
<td>Drainage type</td>
<td>Type</td>
<td>Non-porous</td>
<td>Volumetric weight ( \gamma \text{ (kN/m}^3 )</td>
<td>78</td>
<td></td>
</tr>
<tr>
<td>Unit weight ( \gamma \text{ (kN/m}^3 )</td>
<td>78</td>
<td>Young’s modulus ( E \text{ (kN/m}^2 )</td>
<td>0.02</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Young’s modulus ( E \text{ (kN/m}^2 )</td>
<td>200×10⁶</td>
<td>Moment of inertia</td>
<td>0.003058</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Poisson’s ratio ( \nu )</td>
<td>0.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 7-4 Soil parameters of MC model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>CS0</th>
<th>CS2.5</th>
<th>CS5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model Type</td>
<td>MC Drained</td>
<td>MC Drained</td>
<td>MC Drained</td>
</tr>
<tr>
<td>( \gamma_{\text{water}} \text{ (kN/m}^3 )</td>
<td>16</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>( \gamma_{\text{sat}} \text{ (kN/m}^3 )</td>
<td>21</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>( E_0 \text{ (kN/m}^2 )</td>
<td>250</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td>( E' \text{ (kN/m}^2$/m)</td>
<td>3000</td>
<td>6000</td>
<td>17000</td>
</tr>
<tr>
<td>( E \text{ (kN/m}^2 ), z \geq 3 \text{ m}</td>
<td>-</td>
<td>18500</td>
<td>51500</td>
</tr>
<tr>
<td>( \nu )</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>( c'_{\text{ref}} \text{ (kN/m}^2 )</td>
<td>0.2</td>
<td>10</td>
<td>28</td>
</tr>
<tr>
<td>( \phi \text{ (°) }</td>
<td>38</td>
<td>36</td>
<td>37</td>
</tr>
<tr>
<td>( \psi \text{ (°) }</td>
<td>8</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>( K_0 )</td>
<td>0.38</td>
<td>0.41</td>
<td>0.40</td>
</tr>
</tbody>
</table>

In the model experimental test analyses, interfaces were assigned to the pile surface surrounding its circumference as well as to the pile toe where the high shear stress and strain gradient occurs due to the ‘kick-off’ phenomenon. It was necessary in the case of a rigid pile, which rotates about a point, to adopt interface element for modelling an accurate interaction between the soil and the pile behaviour.
Calculation phase

In the calculation mode of Plaxis, a number of calculation phases can be defined. First, the initial phase of the initial stress state must be defined for the whole model using the submerged unit weight for both the soil elements and the elements that later become the pile. A $K_0$-procedure was used, in which it is assumed that $K_0 = 1 - \sin \phi'$, to determine the initial horizontal effective stress. A sensitivity study showed that the selected $K_0$ value had virtually no impact on predictions (for values examined of up to unity). Following the initial phase, the pile is installed by replacing the soil elements of the volume pile with a set of pile material and interfaces are assigned to the pile surface with the adjacent strength of soil. Modelling of the lateral loading is performed through incremental lateral displacement prescribed for $x$-direction, free for $y$- and $z$-direction. Importantly, sufficiently small displacement advancements should be set for initial loads, aiming for the derivation of the initial stiffness of the soil afterwards if required.

One of salient features of the centrifuge model tests was that very large lateral loads were applied at the pile head for most of the model piles; consequently, considerably large lateral displacements occurred at the pile head. The Plaxis analyses were therefore also conducted using the large strain option available within the program.

With large deformation theory is included in the FEA program Plaxis 3D, this is done automatically by selecting ‘Updated mesh’ option as one of the deformation control parameter in calculation phase (Brinkgreve et al. 2011). It should be noted that an updated mesh analysis takes much more time and is less robust than a normal calculation. Hence, this option should be used only in special cases (Brinkgreve et al. 2011).

7.3.2. Sensitivity analysis

Prior to simulation of the lateral load tests on the centrifuge model piles, sensitivity analyses were exercised to investigate the influences of the boundary condition imposed on the model piles by the soil volume in the strongbox, and the interface strength relating to the less rough soil-pile interaction in the case of the steel model piles. The
effects of these factors cannot easily be evaluated in the centrifuge experimental tests. At this stage, conventional finite element analysis was used for these analyses.

**Effects of boundary condition and mesh density**

The effect of model geometry on the lateral response of the model pile was investigated by enlarging the soil domain in plan and depth, and keeping the soil and pile parameters constant. A geometry model of $25 \text{ m} \times 25 \text{ m} \times 10 \text{ m}$ (denoted as LD) sufficiently larger than the geometry model of the centrifuge test in prototype dimension (denoted as PD) was set to investigate the effect of boundary condition on lateral response. The large geometry model and the centrifuge prototype model were meshed at a medium-fine density and fine density respectively. The inputs of the soil and pile property parameters including the interface reduction factors (set $R_{\text{int}} = 1$) were the same for the four 3D FEMs. The results of the maximum lateral loads, $H$, and peak moments, $M_{\text{max}}$, at two different boundary conditions and two different mesh densities are summarized in Table 7-5 and Table 7-6.

<table>
<thead>
<tr>
<th>Mesh density</th>
<th>Model</th>
<th>H (kN)</th>
<th>M (kN.m)</th>
<th>Effect of boundary condition $(H_{\text{PD}}-H_{\text{LD}})/H_{\text{LD}}$ (%)</th>
<th>$(M_{\text{PD}}-M_{\text{LD}})/M_{\text{LD}}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MM</td>
<td>LD</td>
<td>734</td>
<td>1910</td>
<td>4.8</td>
<td>4.3</td>
</tr>
<tr>
<td></td>
<td>PD</td>
<td>769</td>
<td>1992</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FM</td>
<td>LD</td>
<td>715</td>
<td>1832</td>
<td>6.3</td>
<td>7.8</td>
</tr>
<tr>
<td></td>
<td>PD</td>
<td>760</td>
<td>1974</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 7-6 highlights the errors in the computed results using two boundary conditions (PD and LD) for fine-mesh (FM) and medium-fine density (MM), respectively. Table 7-6 highlights the errors in the computed results using two boundary conditions (PD and
LD) for fine-mesh (FM) and medium-fine density (MM) respectively. At finishing displacement of 4D at load eccentricity of 3D, the PD conditions yielded approximately 4.8% and 6.3% higher lateral loads, and 4.3% and 7.8% higher bending moments, than those using LD conditions, meshed with MM and FM respectively. The MM conditions yielded approximately 2.6% and 1.2% higher lateral loads, and 4.1% and 0.9% higher bending moments, than those using FM conditions, at model geometries LD and PD respectively. In general, the effects of mesh density were found to be less than the effects imposed by the changing boundary conditions.

The loads and moments predicted by the PD condition were slightly higher than those predicted by the LD condition. The prototype dimensions of strongboxes should be employed, so as to realistically reflect the test conditions in the centrifuge environment. Fine mesh as a global fineness factor was applied in the series of FEM to obtain more accurate numerical results.

*Effect of $R_{inter}$ and influence of large deformation*

The model piles were pre-installed into the soil sample. Therefore, the stress level on soils surrounding the piles can be assumed to be unaffected by the pile installation. Nonetheless, the smoothness of the steel piles would cause a certain reduction in soil strength. Hence, the value of the factor $R_{inter}$ was considered an important factor when modelling three-dimensional soil-pile interaction.

Starting with a $R_{inter}$ value of 0.67, as recommended by Brinkgreve et al. (2012), the $L_e/D =10$ pile in soil condition CS5 was selected to examine the influence of $R_{inter}$ value; the predicted responses were found to be consistently stiffer than the experimental responses for all piles. The reduction factor $R_{inter}$ was then reduced to 0.6, with which value the predicted response moderately approached towards the experimental responses. The effect of the choice of $R_{inter}$ on the response is shown on Figure 7-4a, i.e. slight increase in stiffness and ultimate capacity as $R_{inter}$ increase. However, no soil failure was indicated from the FE computation at a defined large displacement.
A further analysis with $R_{\text{inter}} = 0.6$ was examined using the updated mesh option in calculations. The general lateral response produced stiffer than that observed. Using a $R_{\text{inter}}$ value of 0.4 was found to yield close lateral load-displacement responses at
calculation with updated mesh to that observed, as shown in Figure 7-4b.

In comparison with lateral load-displacement responses measured at the pile head for \( L_e/D =10 \) pile in soil condition CS5, numerically predicted responses are found to be closely comparable with measured responses when using the \( R_{\text{inter}} \) value of 0.6 in conventional FEM and 0.4 in the computation with updated mesh control. Table 7-7 summarizes the two \( R_{\text{inter}} \) values employed for the interface elements. These values were applied in the modelling of all centrifuge piles tests.

Table 7-7 Back-figured \( R_{\text{inter}} \) values of strength reduction factor for interface elements

<table>
<thead>
<tr>
<th>Soil parameters for CS5</th>
<th>Without updated mesh control (w/o UM)</th>
<th>With updated mesh control (UM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( R_{\text{inter}} )</td>
<td>0.6</td>
<td>0.4</td>
</tr>
</tbody>
</table>

Figure 7-5 shows bending moment diagrams for \( L_e/D = 10 \) piles predicted without an updated mesh control using a \( R_{\text{inter}} \) value of 0.6 and with an updated mesh control \( R_{\text{inter}} \) value of 0.4. For each set of soil parameters, the computations yield similar bending moment distributions along the pile length. Table 7-8 details the peak moments and Table 7-9 lists the lateral loads predicted using two computations for each soil sample type. At pile head displacements of 0.6\( D \) for CS0, the \( M_{\text{max}} \) and \( H \) values computed with conventional FEM are approximately 1.5 % and 7.3 % higher than those computed with the updated mesh control. At displacement of 1.2\( D \) for CS2.5 and CS5, the \( M_{\text{max}} \) and \( H \) values computed with conventional FEM are negligibly smaller than those computed with the updated mesh control.

Table 7-8 Comparison of peak moments computed with and with no updated mesh control for \( L_e/D = 10 \) piles in centrifuge model soils

<table>
<thead>
<tr>
<th>Soil parameter</th>
<th>( y/D )</th>
<th>( M_{\text{max}} ) (kN.m)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>With updated mesh control (UM)</td>
<td>With no updated mesh control (w/o UM)</td>
</tr>
<tr>
<td>CS0</td>
<td>0.6</td>
<td>529*</td>
<td>537*</td>
</tr>
<tr>
<td>CS2.5</td>
<td>1.2</td>
<td>1323</td>
<td>1240</td>
</tr>
<tr>
<td>CS5</td>
<td>1.2</td>
<td>3008</td>
<td>3001</td>
</tr>
</tbody>
</table>
Table 7-9 Comparison of lateral loads computed with and with no updated mesh control for $L_e/D = 10$ piles in centrifuge model soils

<table>
<thead>
<tr>
<th>Soil parameter</th>
<th>$y_h/D$</th>
<th>$M_{max}$ (kN.m) With updated mesh control (UM)</th>
<th>$M_{max}$ With no updated mesh control (w/o UM)</th>
<th>Error (%) $\frac{(M_{UM}-M_{w/o\ UM})}{M_{UM}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS0</td>
<td>0.6</td>
<td>192$^a$</td>
<td>206$^a$</td>
<td>-7.3</td>
</tr>
<tr>
<td>CS2.5</td>
<td>1.2</td>
<td>490</td>
<td>483</td>
<td>1.4</td>
</tr>
<tr>
<td>CS5</td>
<td>1.2</td>
<td>1188</td>
<td>1149</td>
<td>3.3</td>
</tr>
</tbody>
</table>

(a) $y_h/D = 0.6$

(b) $y_h/D = 1.2$

---

![Graph](image-url)
Figure 7-5 Comparisons of numerical moment diagrams of \( L_e/D = 10 \) piles, computed with and without updated mesh control using different values of \( R_{\text{inter}} \) parameter for soils: (a) CS0, (b) CS2.5, and (c) CS5

7.3.3. Predictions for centrifuge model tests

The FEM predicted load-displacement relationships at the pile head and measured in prototype dimension for the centrifuge model test are compared for all piles in all soil conditions in Figure 7-6. All of these predictions used the CPT-derived parameters listed in Table 7-1 and Table 7-2.

Reasonable agreement is apparent between the predicted and observations. For the model soil CS0, the numerical \( H-y_h \) curves fit consistently well with the experimental data for both piles at the two embedment depths. A reasonable good match exists between the numerical and measured lateral responses for the longer pile in CS2.5, whereas the numerical lateral response is evidently softer than the measured response for shorter pile in CS2.5. This might be indicative of slight differences in the samples prepared in two strongboxes. For soil sample CS5, the predicted initial stiffness is reasonably compatible with that measured for both piles but the capacity for the shorter pile is significantly over predicted.
Chapter 7 Numerical analysis of centrifuge-scale pile test

(a) Experimental Plaxis, Rinter=0.6

CPT back analyzed:
\( c' = 0 \) kPa
\( \phi = 38^\circ \)
\( \psi = 8^\circ \)
\( E = 0.25 + 3z \) MPa
\( \nu = 0.3 \)

\( H_u = 56 \) kN

(b) Experimental Plaxis, Rinter=0.6

CPT back analyzed:
\( c' = 0 \) kPa
\( \phi = 38^\circ \)
\( \psi = 8^\circ \)
\( E = 0.25 + 3z \) MPa
\( \nu = 0.3 \)

\( H_u = 285 \) kN
Chapter 7 Numerical analysis of centrifuge-scale pile test

(c) $H_u = 150 \text{kN}, \frac{y_h}{D} = 0.6$

CPT back analyzed:
$\gamma' = 12 \text{kPa}$
$\phi = 36^\circ$
$\psi = 6^\circ$
$E = 0.50 + 6z \text{MPa}$
$\nu = 0.3$

(d) $H_u = 480 \text{kN}$

CPT back analyzed:
$\gamma' = 12 \text{kPa}$
$\phi = 36^\circ$
$\psi = 6^\circ$
$E = 0.50 + 6z \text{MPa}$
$\nu = 0.3$
Figure 7-6 Numerical load-displacement relations with measures for centrifuge pile tests: (a) CS0, \(L_e/D = 5\), (b) CS0, \(L_e/D = 10\), (c) CS2.5, \(L_e/D = 5\), (d) CS2.5, \(L_e/D = 10\), (e) CS5, \(L_e/D = 5\), and (f) CS5, \(L_e/D = 10\)
7.3.4. Numerical bending moments along pile length

Figure 7-7 presents the distributions of the FEM predicted bending moments $M$ with depth $z$, at identified ultimate lateral loads for piles in all soil samples. The bending moments for shorter piles are far lesser than those of longer piles. For the longer piles with $L_e/D=10$, the smallest peak moment $M_{\text{max}}$ of 734 kN.m was predicted at an ultimate load of 293 kN in uniform sand CS0. With the addition of cement content, the bending moments increase with cement content increased load capacity. The $M_{\text{max}}$ value of 3,000 kN.m for sample CS5 under a lateral load of 1,180 kN is more than twice the $M_{\text{max}}$ value of 1,240 kN.m for sample CS2.5 under an ultimate load of 483 kN.

Figure 7-7 Distributions of bending moment along pile length, $M$ - $z$ curves of model piles at non-dimensional embedment depths of 5 and 10

The depths $z_m$ of the peak moments are estimated to be approximately 1.9 m ($z_m \approx 0.38L_e$) in uniform sand CS0 and 1.83 m ($z_m \approx 0.37-0.38L_e$) for cemented sands CS2.5 and CS5. The relations of induced moments by horizontal loads, excluding the moment
(\(M_0 = H.e\)) at the soil surface from the maximum moment (\(M_{\text{max}}\)), and the applied horizontal loads at the pile head (\(H\)), (\(M_{\text{max}} - M_0\))/\(H\) ranges from approximately 0.64\(z_m\) to 0.68\(z_m\).

7.3.5. Numerical limit lateral pressures on piles

The numerically derived \(P_u\) (p/D) variations with depth for un-cemented sand and weakly cemented sands were derived by double differentiation of the moment profiles and are presented in Figure 7-8.

![Figure 7-8 Distribution of transverse pressure along embedded length of prototype piles](image)

The lateral stress is presumed zero at the soil surface and the resultant net pressure increases with depth up to approximately 43%, 41% and 40% of the pile embedment in soils CS0, CS5.2 and CS respectively. The pressure then decreases until it reduces to zero at a depth \(z_r\) about where the pile rotates (the \(z_r\) values are approximately 80%, 78% and 75% of the pile embedded depths in these soils). Below \(z_r\), the net soil pressure operates in the opposite direction and increases from zero at depth \(z_r\) to a maximum at
the tip of the pile. Above depth \( z_s \), the maximum lateral pressure on the front side of pile is estimated to be 500 kPa at a depth of approximately \( 0.43L_e \) for uniform sand, and 860 kPa and 2,080 kPa at a depth of approximately \( 0.41L_e \) to \( 0.40L_e \) for weakly cemented sands, respectively. Below depth \( z_s \), the maximum pressures acting on the pile at the pile toe are computed as 590 kPa, 1,410 kPa and 3,700 kPa.

The average ratio of \( z_s/L_e \) was predicted approximately 0.54 for CS0 and is slightly less than the reported 0.6 observed in laboratory tests on model piles in cohesionless soils (Prasad and Chari 1999). Table 7-10 summarizes the magnitudes of peak pressure above and below a depth of \( z_s \) for these soil conditions.

Table 7-10 Numerical limiting transverse pressures on the front and back pile faces for longer piles with \( L_e/D=10 \)

<table>
<thead>
<tr>
<th>Depth</th>
<th>Lateral pressure, ( P ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( z \approx 0.40L_e )</td>
<td>CS0 510</td>
</tr>
<tr>
<td>( z = L_e )</td>
<td>CS0 590</td>
</tr>
</tbody>
</table>

With the limit lateral pressure revealed for the upper depths of approximately \( 4D \) \((0.4L_e)\) for longer piles with \( L_e/D=10 \), the slopes of the limit pressure increasing were then calculated to be approximately 255 kPa/m, 430 kPa/m, and 1050 kPa/m for these centrifuge soils. The values of limit pressure \( P_u \) within the depths revealed can then be obtained.

7.3.6. FEM derived P-y curves of weakly-cemented sands

The soil-pile lateral movement \( y \) data along with the embedded length of the pile can directly be extracted from the inserted beam in the centerline of the volume pile. Relating the estimated net lateral pressures \( P \) to the corresponding \( y \) values at that depth, numerical \( P\cdot y \) curves were obtained for all centrifuge soil samples.

To develop a characteristic curve for the soil resistance with lateral displacement, the soil pressure \( P \) was normalized by the quantity \( P_u \), and \( y \) normalized by the piled diameter \( D \). The limit pressures discussed in previous section were determined for the
Figure 7-9 presents the normalized $P_y$ data for three types of soils CS0, CS2.5 and CS5. It is evident that, for a specific soil sample, the multiple dimensional $P_y$ curves for various depths were closely merged.
Chapter 7 Numerical analysis of centrifuge-scale pile test

Figure 7-9 Normalized P-y data for centrifuge soils: (a) CS0, (b) CS2.5, and (c) CS5

Non-linear regression analyses on the dimensionless P-y data yield the P-y relations for these soil samples, expressed as:

\[
\frac{P}{P_u} = \tanh \left[ 3.41 \left( \frac{y}{D} \right)^{0.75} \right] \quad \text{CS0} \quad (7.1)
\]

\[
\frac{P}{P_u} = \tanh \left[ 4.41 \left( \frac{y}{D} \right)^{0.75} \right] \quad \text{CS2.5} \quad (7.2)
\]

\[
\frac{P}{P_u} = \tanh \left[ 4.65 \left( \frac{y}{D} \right)^{0.75} \right] \quad \text{CS5} \quad (7.3)
\]

The stiffness of the soil P-y springs for CS5 is expected to become stiffer with the increased cementation degree compared with those for CS0 and CS2.5. Nonetheless, the P-y spring stiffness of CS5 appears to be slightly lower than expected and the \( R^2 \) value is also less satisfactory than those of the other two soil types.

For the purpose of comparing the P-y data of the weak cementation soils with those well
cementation soils at Pinjar, a generalized formulation is rational for representing all the centrifuge soils. Figure 7-10 shows the combined $P-y$ data. A similar tangent hyperbolic equation was found to mathematically best fit to these combined $P-y$ data. The general functional model of the $P-y$ curve for weakly cemented sands is written as

$$\frac{P}{P_u} = \tanh \left[ 3.84 \left( \frac{y}{D} \right)^{0.75} \right]$$  \hspace{1cm} (7.4)

Figure 7-10 also presents a comparison of the prediction based on Equation (7.4) and the FEM derived results.

Figure 7-10 Normalized lateral pressure vs. displacement with prediction of hyperbolic function of Equation (7.4)

7.4. Summary

Through back analysis of the CPT data of the model experiment, the soil parameters of a simple model with MC yield surface have been assessed for centrifuge model soils using the methodology of spherical cavity expansion solutions. Meanwhile, the values
of soil stiffness $E$ were also estimated to be approximately $4q_e$.

Soil parameters assessed in back analysis of CPT data have been effectively applied in FEM for simulating centrifuge-scale load tests, providing appropriate $R_{\text{inter}}$ values for soil interface elements. Reasonable lateral responses were yielded for the prototype piles at two different embedment depths in these soils. This consistency suggests the feasibility of using CPT data directly for analysing laterally loaded piles.

The numerically derived $P_u$ values are approximate to be $0.32q_e$ to $0.29q_e$ for uncedmented sand and very weakly cemented sands. Regression analyses on the dimensionless $P-y$ data from FEM yield a hyperbolic trigonometry function that can mathematically describe the soil springs; the stiffness of these soil springs slightly varies with the cementation degree. However, it’s noteworthy that the limiting pressures on numerical $P-y$ curves are developed at surprisingly great displacement.
CHAPTER 8   RESULTS AND DISCUSSION

8.1.   Introduction

The load transfer relationships $P-y$ curves for the calcareous sandstone at Pinjar have been derived from measured bending moments and deflections in CHAPTER 4 whilst $P-y$ relationships for the weakly cemented sands tested in the centrifuge have been estimated using finite element analyses from observed displacements and an assumed linear elastic Mohr-Coulomb material model in CHAPTER 7. These Pinjar observations may be regarded as being representative of lateral responses of strong $c-\phi$ soils while the centrifuge observations gave insights into the responses of very weak $c-\phi$ soils (and un-cemented sand with $c'=0$).

In this chapter, an examination of the two groups of $P-y$ curves will first be provided, taking into account the different levels of cementation. These curves are assessed for consistency with the tests reported in this study and then applied to a selected history case. The second section of this chapter evaluates the ability of existing approaches (discussed in CHAPTER 2) to predict the ultimate lateral pressure $P_u$ and the relationship between $P$ and $y$ for the Pinjar and centrifuge tests. Finally, based on the characteristics of back-calculated $P-y$ curves, and the operational soil moduli $E$ deduced from CPT $q_c$ data, a simple methodology to develop idealized $p-y$ model for the general $c-\phi$ soils is proposed. The suitability and accuracy of the proposed CPT $q_c$-based $p-y$ model is also discussed.

8.2.   Examination of the back-calculated $P-y$ curves from Pinjar and centrifuge tests

The ability of the fitted $P-y$ relationships to predict the lateral responses of the Pinjar and centrifuge test piles is assessed using the $P-y$ formulations given by Equation (4.5) (see section 4.5.3) and Equations (7.1) to (7.3) (see section 7.3.6). The formulations are summarized in Table 8-1.
These curves were entered into the program Oasys Alp (ALP 2010) which was then run to predict the lateral pile responses. The computation program Oasys-Alp is a simple, yet efficient solver to predict lateral pile responses, modelling the pile as a series of elastic beam elements and soil as a series of non-interactive discrete non-linear Winkler springs; the assumption of elastic beam elements is a clear deficiency for predicting the Pinjar tests where a non-linear pile structural response has been observed. The graphical and tabular analysis outputs from Alp contain predictions of the pressure, the pile deflection, the shear force and bending moment induced in the pile. Alp-predicted lateral $H-y_h$ responses at the pile head are compared with the observed responses.

Table 8-1 Formulations back-calculated from Pinjar field tests and centrifuge scale tests

<table>
<thead>
<tr>
<th>Soil</th>
<th>Formulation</th>
<th>Equation No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Well-cemented sands and weak rocks</td>
<td>$\frac{P}{P_a} = \tanh \left[ 60 \left( \frac{y}{D} \right)^{0.75} \right]$</td>
<td>Eq.(4.5)</td>
</tr>
<tr>
<td>Pinjar material</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CS0</td>
<td>$\frac{P}{P_a} = \tanh \left[ 3.41 \left( \frac{y}{D} \right)^{0.75} \right]$</td>
<td>Eq. (7.1)</td>
</tr>
<tr>
<td>Very weakly cemented sands and the uncemented sand</td>
<td>$\frac{P}{P_a} = \tanh \left[ 4.41 \left( \frac{y}{D} \right)^{0.75} \right]$</td>
<td>Eq. (7.2)</td>
</tr>
<tr>
<td>CS2.5</td>
<td>$\frac{P}{P_a} = \tanh \left[ 4.65 \left( \frac{y}{D} \right)^{0.75} \right]$</td>
<td>Eq. (7.3)</td>
</tr>
<tr>
<td>CS5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

8.2.1. Examination of back-calculated P-y curves against the present tests

To perform the analyses with the program Oasys-Alp, the full-scale test piles need to be modelled as elastic with a flexural rigidity ($EI$) of 7200 kN.m² for $D=340$ mm and 31000 kN.m² for $D=450$ mm. For two diameters, the values of the constants in Equation (4.5) of 60 and 0.75 were used and the back-calculated $P_u/q_c$ ratio of about 0.15 for shallow depths was assumed for whole depth. The average CPT $q_c$ profile was adopted for the calculation.

Figure 8-1a and Figure 8-1b show the predicted and the measured lateral load-deflection responses for the 340-mm piles and 450-mm piles. The predicted $H-y_h$ response of the 340-mm pile appears to be softer than that of 450-mm pile under the same lateral loads. This reflects the importance of the piles’ section property $EI$. The comparison between
measurements and predictions (noting that measurements are only plotted when there is full elastic behaviour) is reasonable with difference being attributable to highly variable \( q_c \) values at the site and a very high initial stiffness seen during the experiments on the 340 mm piles.

![Figure 8-1 Predicted and measured \( H-y_h \) responses of the full-scale piles of (a) 340-mm, and (b) 450-mm diameters](image)

Figure 8-1 Predicted and measured \( H-y_h \) responses of the full-scale piles of (a) 340-mm, and (b) 450-mm diameters
Figure 8-2 presents and compares the measured and predicted normalized deflection profiles for the 340 mm diameter piles under a lateral load of 120 kN. Reasonable agreement is observed although it is evident that Alp predicted a slightly more rapid decay in lateral displacement below ground level.

The average CPT $q_c$ profile in each type soil was used in the Alp analyses for the centrifuge model soils CS0, CS2.5 and CS5. The ultimate lateral pressures, $P_u$, were approximated by assuming the average proportional relationship with $q_c$ data indicted in the experiments i.e. $P_u/q_c = 0.3$.

Figure 8-3 presents examples of the predicted lateral load-displacement responses at the pile head for centrifuge model soils CS0 and CS5. In line with the experiments, the calculated $H-y_h$ responses for piles embedded at $L_e/D$ ratios of 5 are generally softer than those with $L_e/D=10$. For the un-cemented soil CS0, the prediction accurately estimates the ultimate lateral load of the pile at $L_e/D = 10$ but underestimates the lateral capacity of the pile with $L_e/D = 5$. The lateral capacities of both piles were predicted satisfactorily in sample CS5.
Figure 8-3 Predicted $H$-$y_h$ responses compared with the measured for centrifuge soils (a) CS0 and (b) CS5, using the back-calculated $P_u$ and $P_y$ curves.

Figure 8-4 shows a typical bending moment diagram predicted using Alp and compares it with the Plaxis prediction for a prototype pile embedded in soil CS5. Both methods
predict comparable bending moments (as may be expected, given that the $P-y$ curves used in Alp were derived from these moments). However the Alp prediction of lateral movement is higher than that given by the FEM (at the high load level considered) and this possibly reflects the effects of base shear which is not included in the Alp analysis in addition to some conservatism in the assessment of Equation (7.5).

Figure 8-4 Predicted bending moment diagram and deflection diagram compared with numerical estimates for centrifuge soil CS5 (a) $M-z$ curves, and (b) $y-z$ curves
8.2.2. Application of Equation 7.1 to Blessington sand (Li et al. 2014)

It is instructive to examine the suitability of the uncemented sand component of Equation (7.1) for a natural overconsolidated sand deposit in Blessington, Ireland (Li et al. 2014). The deposit has been heavily over-consolidated due to effects of glaciation and recent excavation (the site is at the base of large quarry). The sand is extremely dense (in-situ relative density of close to 100%) with a unit weight of 20 kN/m$^3$. No free water was observed within the embedded depths of the test piles. The critical friction angle $\phi_{\text{crit}}$ was about 37° and peak friction angles were between 54° at a 1 m depth and 40° at a depth of about 5 m. Multiple CPTs were conducted and the averaged $q_c$ values was found to be relatively consistent; see Figure 8-5.

Load tests on piles PS2 and PS5 were chosen for the comparison with the $P$-$y$ model. Both piles, with an outer diameter of 340 mm and a wall thickness of 14 mm, were driven open-ended to embedded depths of 2.2 m and 5 m respectively. The horizontal loads were applied at 0.4 m and 1.32 m above the ground surface on piles PS2 and PS5. Pile PS2 was classified as a short rigid pile and PS5 as a long flexible pile. The complete load-displacement data of the two piles are (reproduced here and) shown in Figure 8-6, in which only static load results are shown and the cyclic loads at small cycle-numbers are ignored.

To compare the computed lateral responses with the measurements, the derived $P$-$y$ model was used to generate $P$-$y$ data based on the average measured $q_c$ values, as shown in Figure 8-5; the $P$-$y$ relationship for the centrifuge soil CS0 ($c' = 0$ kPa) [Equation (7.1)] and $P_u/q_c = 0.32$ were employed.

Figure 8-6 compares the predicted $H$-$y_o$ responses just above ground level with the measured responses of PS2 and PS5. Good agreement is evident demonstrating the applicability of the hyperbolic tangent $P$-$y$ relationship coupled with the proportional relationship between $P_u$ and $q_c$ for the Blessington sand.
Figure 8-5 Measured CPT-$q_c$ profiles at test site in Blessington, Ireland (Li et al. 2014)

Figure 8-6 Comparison of predicted $H$-$y_h$ responses with the measured for test piles P2 and P5, load test in Blessington (after Li et al. 2014)
8.3. Evaluation of existing models to predict $P_u$ and $P-y$ curves

This section examines the ability of existing models to predict the lateral pressure $P_u$ and load-transfer $P-y$ curves measured at Pinjar and inferred from the centrifuge tests. Table 8-2 lists the existing methods, which have been described in sections 2.4, 2.6 and 2.7. Table 8-3 lists the existing formulations considered for the comparison of $P-y$ curves (described earlier in sections 2.6 and 2.7). When constructing the $P-y$ curves for $c-\varphi$ soil, the estimation of the ultimate lateral resistance ($p_u$) employs the $p_{u-c\varphi}$ term, which combines the cohesive component and frictional components (Evans and Duncan 1982; Ismael 1990; Reese and Van Impe 2001). The soil resistance $p$ force per unit length in the considered models is accordingly transformed into the lateral pressure $P$ throughout the evaluation process.

Table 8-2 Existing models used for comparing the ultimate lateral pressure, $P_u$

<table>
<thead>
<tr>
<th>Existing models</th>
<th>Centrifuge model soils</th>
<th>Pinjar field material</th>
</tr>
</thead>
<tbody>
<tr>
<td>API sand (O’Neill and Murchison 1983)</td>
<td>✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
<tr>
<td>c-\varphi soil (Brinch Hansen 1961)</td>
<td>✓ ✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
<tr>
<td>c-\varphi soil (Reese and Van Impe 2001)</td>
<td>✓ ✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
<tr>
<td>Weak rock (Reese 1997)</td>
<td>✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
<tr>
<td>CPT $q_c$-based, sand (Lee et al. 2010)</td>
<td>✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
</tbody>
</table>

* if $c'=0$ kPa, the c-\varphi soil (Reese and Van Impe 2001) reduces to sandy soil (Reese et al. 1974).

Table 8-3 Existing models used for comparing $P-y$ curves

<table>
<thead>
<tr>
<th>Existing models</th>
<th>Centrifuge model soils</th>
<th>Pinjar field material</th>
</tr>
</thead>
<tbody>
<tr>
<td>API sand (O’Neill and Murchison 1983) with $p_{u-c\varphi}$ (Reese and Van Impe 2001)</td>
<td>✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
<tr>
<td>API soft clay (Matlock 1970) with $p_{u-c\varphi}$ (Reese and Van Impe 2001)</td>
<td>✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
<tr>
<td>c-\varphi soil (Reese and Van Impe 2001)</td>
<td>✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
<tr>
<td>Stiff clay with no free water (Welch and Reese 1972) with $p_{u-c\varphi}$ (Reese and Van Impe 2001)</td>
<td>✓ ✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
<tr>
<td>Weak rock (Reese 1997)</td>
<td>✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
<tr>
<td>Rock mass (Liang et al. 2009)</td>
<td>✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
<tr>
<td>CPT $q_c$-based sand (Suryasentana and Lehane 2014)</td>
<td>✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
<tr>
<td>CPT $q_c$-based carbonate sand (Novello 1999)</td>
<td>✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
<tr>
<td>CPT $q_c$-based calcareous sand (Dyson and Randolph 2001)</td>
<td>✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
</tr>
</tbody>
</table>
Soil and weak rock parameters used for comparison

Table 8-4 lists the soil parameters of stiff clay model for Pinjar in-situ material. To make an equivalent comparison, the $\epsilon_{50}$ value is assigned a value of 0.0005, which is the same as the $k_{im}$ value in Reese weak rock model. The weak rock properties of the Pinjar field material are summarized in Table 8-5. The unconfined compressive strength, $q_{ucs}$, is estimated from the $c$ and $\phi$ values assuming the Mohr-Coulomb failure criterion. The initial modulus of the subgrade reaction $k_i$ of $c$-$\phi$ soil is determined according to suggestion by Reese and Van Impe (2001). In addition, the main parameters required for rock mass p-y criterion proposed by Liang et al. (2009) are listed in Table 8-6. It includes the modulus of intact rock mass $E_i$, unconfined compressive strength $q_{ucs}$, and rock mass constants of GSI and $m_i$. The value of $E_i$ was taken as the small-strain modulus. The value of GSI and $m_i$ were obtained according to Marinos and Hoek (2000).

Table 8-4 Soil parameters for stiff clay and c-$\phi$ soil models of Pinjar field material

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit weight</th>
<th>Cohesion</th>
<th>Internal friction angle</th>
<th>Strain</th>
<th>Initial stiffness</th>
<th>Pile diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>c-$\phi$ Soil</td>
<td>$\gamma$ (kN/m$^3$)</td>
<td>$c'$ (kPa)</td>
<td>$\phi$ ($^\circ$)</td>
<td>$\epsilon_{50}$</td>
<td>$k_i$ (kN/m$^3$)</td>
<td>D (m)</td>
</tr>
<tr>
<td>Pinjar</td>
<td>15.86</td>
<td>160</td>
<td>40</td>
<td>0.0005</td>
<td>260000$^a$</td>
<td>0.34</td>
</tr>
</tbody>
</table>

$^a k_i = k_{oc} + k_{oc} = 210$ (MN/m$^3$) + 50 (MN/m$^3$) = 260 (MN/m$^3$) (Reese and Van Impe 2001).

Table 8-5 Parameters of Reese’s weak rock model for Pinjar field material

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Compressive strength</th>
<th>Reduction factor</th>
<th>Constant</th>
<th>Initial modulus</th>
<th>Pile diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>weak rock</td>
<td>$q_{ucs}$ (kPa)</td>
<td>$a_c$</td>
<td>$k_{mr}$</td>
<td>$E$ (kN/m$^2$)</td>
<td>D (m)</td>
</tr>
<tr>
<td>Pinjar material</td>
<td>686$^a$</td>
<td>1</td>
<td>0.0005</td>
<td>400000</td>
<td>0.34</td>
</tr>
</tbody>
</table>

$^a$ Equivalent $q_{ucs} = 2c' \tan(45 + \phi/2)$, in which $c' = 160$ kPa and $\phi = 40^\circ$.

Table 8-6 Parameters of Liang et al. rock mass model for Pinjar weak rock

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unconfined compressive strength</th>
<th>Rock mass constant</th>
<th>Rock mass constant</th>
<th>Modulus of intact rock</th>
<th>Pile diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock</td>
<td>$q_u$ (kPa)</td>
<td>$m_i$</td>
<td>GSI</td>
<td>$E_i$ (kN/m$^2$)</td>
<td>D (m)</td>
</tr>
<tr>
<td>Pinjar material</td>
<td>686</td>
<td>12</td>
<td>40</td>
<td>350000$^a$</td>
<td>0.34</td>
</tr>
</tbody>
</table>

$^a$ Determined as small-strain modulus from the SCPT measurement.
Table 8-7 lists the soil properties required for the estimation of $P_u$ and calculation of the $P-y$ curves for the centrifuge samples. These values are principally determined from the results of the soil laboratory tests and are consistent with those of the CPT back-analysis and numerical analysis discussed in previous chapters. The $\varepsilon_{50}$ values for soils CS2.5 and CS5 are taken approximately as 0.004 according to soil laboratory tests. The initial modulus of subgrade reaction $k_i$ is determined simply as a summation of the $k_c$ and $k_\phi$ values based on the recommendation for computing the $P-y$ curves of $c-\phi$ soil (Reese and Van Impe 2001). The $k_c$ and $k_\phi$ values of clay and sand were suggested by Reese and Sullivan (1980) and Reese et al. (1974). However, the magnitude of parameter $k_i$ has been found to have little effect on the predicted behavior of lateral load-deflection relationships (Ismael 1990).

Table 8-7 Soil parameters of centrifuge soils for sand, clay, and silt models

<table>
<thead>
<tr>
<th>Soil</th>
<th>Unit weight $\gamma$ (kN/m$^3$)</th>
<th>Cohesion $c'$ (kPa)</th>
<th>Internal friction angle $\phi$ ($^\circ$)</th>
<th>Strain $\varepsilon_{50}$</th>
<th>Initial stiffness $k_i$ (kN/m$^3$)</th>
<th>UCS $q_{uc}$ (kPa)</th>
<th>Pile diameter $D$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS0</td>
<td>11.0</td>
<td>0.02</td>
<td>38</td>
<td>-</td>
<td>28500</td>
<td>-</td>
<td>0.5</td>
</tr>
<tr>
<td>CS2.5</td>
<td>10.6</td>
<td>12</td>
<td>36</td>
<td>0.004$^a$</td>
<td>31200</td>
<td>50</td>
<td>0.5</td>
</tr>
<tr>
<td>CS5</td>
<td>10.8</td>
<td>28</td>
<td>37</td>
<td>0.004$^a$</td>
<td>36600</td>
<td>125</td>
<td>0.5</td>
</tr>
</tbody>
</table>

$^a$ Determined at $\sigma'_1 = 50$ kPa for drained standard triaxial compressive tests.

8.3.2. Prediction of $P_u$ using existing methods

Figure 8-7 compares the measured $P_u$ values with predictions for the centrifuge model soils. For un-cemented sand CS0 within prototype depths of less than 3 m, all estimated $P_u$ values by the methods considered are similar and agree reasonably well with the measured values. For depths greater than 3 m, however, estimations using the CPT $q_c$-based sand model by Lee et al. (2010) sand gives the lowest $P_u$ values at great depth ($z > 3$ m) among these estimates and to some extent differs from the measured values while estimations using the $c-\phi$ soil model developed by Brinch Hansen (1961) are the closest values. Limit pressures estimated using the API sand model by O'Neill and Murchison (1983) and another $c-\phi$ soil model by Reese and Van Impe (2001) are closely similar to each other and progressively higher than the measured values at great depth. The difference between these estimations becomes greater with depth.
For very weakly cemented sand CS2.5 (very weak c-φ soil), $P_u$ values estimated by Brinch-Hansen c-φ soil are reasonably compatible with the measured $P_u$ values for whole depth. The $P_u$ estimates using the remaining methods, including the API sand and Reese and Van Impe c-φ soil, Lee et al. $q_c$-based sand, and Reese weak rock, are generally lower than the observations.

For soil CS5, the $P_u$ values using the considered methods are estimated in similar patterns to those with CS2.5. Again, the Brinch-Hansen c-φ soil gives reasonable estimates of $P_u$ values. The API sand model and Reese and Van Impe c-φ soil consistently yield considerable lower $P_u$ values, the same is true with Lee et al. $q_c$-based sand.

For both cemented soils used in the centrifuge, CS2.5 and CS5, the Reese weak rock model, consistently produces the lowest $P_u$ values among all the methods, and is evidently shown to be not suitable for weakly cemented sands. Interestingly, for weakly cemented sands, the accuracy of estimates by the Lee et al. $q_c$-based sand appears to improve with the increase of cohesion strength.

![Graph](image-url)
Figure 8-7 Comparison of $P_u$ profiles estimated by existing methods with derived relations for centrifuge model soil conditions: (a) CS0, (b) CS2.5, and (c) CS5.
Based on the foregoing comparisons, it can be concluded that the Brinch Hansen (1961) c-φ soil provides reasonable predictions for $P_u$ in sand and weakly cemented sands. The Lee et al. (2010) CPT $q_c$-based sand model under-predicts the $P_u$ values for cemented sands. The API sand model and the c-φ soil model of Reese and Van Impe (2001) are clearly not suitable to sands with a $c'$ component of strength. The Reese (1997) weak rock model is not applicable to weakly cemented sands. The $P_u$ comparison implies that the c-φ soil models are not applicable to the Pinjar field material (i.e. for hard c-φ soils).

Figure 8-8 Comparison of derived $P_u$ values with predictions by existing methods for Pinjar field material

Figure 8-8 compares $P_u$ values estimated by two selected c-φ soil models, CPT $q_c$-based sand and Reese weak rock with $P_u$ values measured at Pinjar. The $P_u$ values for shallow depth of Pinjar field material were derived on basis of measures; those for great depth unrevealed were simply calculated at a similar $q_c$-$P_u$ relation to that derived for the shallow depth. The soil resistance at depth is taken to be constant simply viewing the well cemented material as a cohesive soil or weak rock (Matlock 1970; Reese 1997;
As demonstrated, Brinch-Hansen $c$-$\phi$ soil estimates the highest $P_u$ values among these for the whole depth. The $P_u$ value tends to increase progressively with mean stress. The $P_u$ predictions of Reese & Van Impe $c$-$\phi$ soil also tends to increase with the stress, however, it yields the lowest $P_u$ values at shallow depths and then relatively higher values at great depth. The difference between the predicted $P_u$ values of the two $c$-$\phi$ soil models becomes greater with depth.

As can be seen on Figure 8-8, the $P_u$ values predicted by Reese weak rock model and Lee et al. CPT $q_c$-based sand model are more comparable with the Pinjar $P_u$ values than other models. The Reese weak rock model and the derived CPT $q_c$-based model have demonstrated the capability to estimate lateral resistance $P_u$. It should be noted that the $P_u$ values of Reese weak rock model are still marginally smaller than the measured $P_u$ values whilst the $P_u$ values of Lee et al. CPT $q_c$-based sand model are slightly larger than the measured $P_u$ values. Meanwhile, it can be seen that the estimated $P_u$ values by the Lee et al. CPT $q_c$-based sand slightly increase with the effective mean stress.

8.3.3. Comparison of $P$-$y$ curves with existing formulations

Figure 8-9 presents estimated and predicted $P$-$y$ curves at a depth of $3D$ for the centrifuge cemented soils. Using Reese & Van Impe $c$-$\phi$ soil model, API sand and the API soft clay with $p_{u-c\phi}$ model, the predicted $P$-$y$ curves are significantly different from the estimated $P$-$y$ curves in shape and magnitude. The derived $P$-$y$ curves are consistently initially softer, yet, in all cases, ultimately stronger than the predicted $P$-$y$ curves. This observation is similar to that observed in a comparison of the lateral response of calcareous sand with the API sand curves presented by Dyson and Randolph (2001).

Of the three $P$-$y$ models considered (Matlock 1970; O'Neill and Murchison 1983; Reese and Van Impe 2001), API sand $p_{u-c\phi}$ yields a relatively stiffer initial slope and lower capacity than the $c$-$\phi$ soil and API $p_{u-c\phi}$ soft clay.
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(a) Lateral pressure $P$, (kPa) vs. Lateral displacement, $y$ (mm)

- c-φ soil, $c=0$ (Reese & Van Impe 2001)
- API Sand (O'Neill & Murchison 1983)
- CS0

(b) Lateral pressure $P$, (kPa) vs. Lateral displacement, $y$ (mm)

- c-φ soil (Reese & Van Impe 2001)
- API Sand with $p_u$-cφ
- API Soft clay with $p_u$-cφ
- CS2.5
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Figure 8-9 Comparison of derived P-y curves with considered P-y curves for centrifuge soil samples: (a) CS0, (b) CS2.5, and (c) CS5

Figure 8-10 shows predictions using the Reese weak rock model and the derived P-y curves for a depth of 3D for centrifuge soils CS2.5 and CS5. In comparison, the Reese weak rock P-y curves constructed here have a stiffer initial slope but significantly lower lateral resistance capacity than the derived P-y curves. The $P_u$ values applied in the normalization (in Figure 8-10) are listed in Table 8-8.

Table 8-8 $P_u$ values for a depth of 3D for soils CS2.5 and CS5

<table>
<thead>
<tr>
<th>Soil</th>
<th>$P_u$(kPa)</th>
<th>$P_u$(kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Reese weak rock</td>
<td>Eq. (7.2), Eq. (7.3)</td>
</tr>
<tr>
<td>CS2.5</td>
<td>256</td>
<td>767</td>
</tr>
<tr>
<td>CS5</td>
<td>597</td>
<td>1918</td>
</tr>
</tbody>
</table>

The difference between the predicted and derived curves may be partially attributed to the particular UCS value employed. However, as discussed, the use of UCS to estimate $P_u$ for weakly cemented soils does not show much success.
Figure 8-10 Comparison of the present $P$-$y$ curves with calculated curves using Reese weak rock for soils CS2.5 and CS5: (a) dimensional $P$-$y$ curves, and (b) dimensionless $P$-$y$ curves for a depth of 3D
Figure 8-11 presents the predicted \( P-y \) curves at a depth of \( 3D \), using CPT \( q_c \)-based methods for soil samples CS0, CS2.5 and CS5. Unlike the \( P-y \) curves calculated by the API soil models and \( c-\varphi \) soil model, in which the parameters are all based predominantly on the undrained shear strength \( c' \) and/or friction angle \( \varphi \), the calculated curves of existing \( q_c \)-based \( P-y \) models are reasonably comparable to those derived. In particular, the initial stiffness of the \( P-y \) curves calculated with sand model developed by Suryasentana and Lehane (2014) show greater agreement than the other two models, Novello’s and Dyson and Randolph’s, before the soil reaches its limiting pressures for weakly cemented sands (CS2.5 and CS5). It still yields a much stiffer initial \( P-y \) slope than the derived slope for un-cemented sand at a depth of \( 3D \). The initial slopes of \( P-y \) curves of sand models by Novello (1999) and Dyson and Randolph (2001) generally agree well with those derived for the un-cemented sand CS0. However, they are much softer than those derived for the cemented materials CS2.5 and CS5. The general agreement of these CPT based methods is surprising given that these methods were derived for sands with no \( c \) component of strength.
Figure 8-11 Comparison of derived P-y curves with existing $q_c$-based $P-y$ curves for centrifuge model soils: (a) CS0, (b) CS2.5, and (c) CS5

Predicted $P-y$ curves for the strong $c$-$\phi$ soil in Pinjar are compared in Figure 8-12a with the mean measured curve at a depth of about $2.2D$ for the 340mm diameter pile. Figure 8-12b presents normalised $P-y$ relationships to show the characteristic shape of these
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$P-y$ curves. As can be seen in Figure 8-12a, the $c-\phi$ stiff clay with $p_u-c\phi$ yields comparable initial stiffness but significantly lower ultimate resistance; the derived $P-y$ curve has the highest lateral resistance at a depth of $2.2D$ and the ultimate resistance estimated by Reese weak rock model is relatively close to the measured curve. The group of CPT $q_c$-based $P-y$ curves is shown to have softer initial slopes and relatively low lateral resistance with displacement. The Liang et al.’ rock mass $P-y$ model yields extremely low values of resistance, which may be attributed to the low $q_{u cs}$ values assigned to the rock mass constants GSI and $m_i$ for the case of Pinjar material.

For the characteristic $P-y$ curves in Figure 8-12b, i.e. $P/P_u$ versus $y/D$ relationships, good agreement exists between the $P-y$ curves calculated using Reese weak rock and stiff clay with $p_u-c\phi$, and typical measured $P-y$ curves. The remaining $P-y$ curve predictions are generally softer than the measured one. The Reese weak rock model and the stiff clay $P-y$ model with $p_u-c\phi$ are essentially equivalent functions with a “one-fourth” parabola; they only differ in the $p_u$ values determined.

Based on the foregoing assessment of $P-y$ curves, only Reese weak rock model predicts a curve comparable to the measured one. It is noteworthy that this same weak rock model provided very poor predictions for the weakly cemented centrifuge soils.
Figure 8-12 Comparison of $P$-$y$ models in (a) dimensional unit, and (b) non-dimensional unit for Pinjar field material
8.3.4. Lateral responses predicted by selected models

The conventional $P-y$ models and the non-$q_c$-based $P-y$ models were implemented in Oasys Alp to predict lateral load-deflection responses for centrifuge soil samples.

An estimate of the ultimate lateral loads ($H_u$) for the centrifuge model piles is first made using the Brinch-Hansen $c$-$\phi$ soil model. Table 8-9 lists the estimated load capacities of piles having normalized depths ($L_e/D$) of 10 and 5; corresponding errors of estimations are also indicated for each pile. This $c$-$\phi$ soil model was found to be rather sensitive to soil strengths.

Table 8-9 Estimated lateral capacities by the Brinch-Hansen $c$-$\phi$ model for centrifuge model piles in prototype dimension

<table>
<thead>
<tr>
<th>Soil sample</th>
<th>Piles, $L_e/D = 10$</th>
<th></th>
<th></th>
<th>Piles $L_e/D = 5$</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Estimated $H_u$</td>
<td>Error, ($H_{u,est}$-$H_u$)/$H_u$</td>
<td></td>
<td>Estimated $H_u$</td>
<td>Error, ($H_{u,est}$-$H_u$)/$H_u$</td>
</tr>
<tr>
<td>CS0</td>
<td>236</td>
<td>-16</td>
<td></td>
<td>37</td>
<td>-26</td>
</tr>
<tr>
<td>CS2.5</td>
<td>593</td>
<td>24</td>
<td></td>
<td>140</td>
<td>-7</td>
</tr>
<tr>
<td>CS5</td>
<td>1300</td>
<td>10</td>
<td></td>
<td>287</td>
<td>29</td>
</tr>
</tbody>
</table>

Figure 8-13 compares the $H$-$y_h$ responses predicted by conventional $P-y$ models for the centrifuge model pile tests with those measured in CS0 and CS5. As can be seen in Figure 8-13, the lateral load capacity predicted by the conventional $P-y$ curves is somewhat lower than that measured for un-cemented sand CS0 (Figure 8-13a) but significantly lower for cemented sand CS5 (Figure 8-13b).
Figure 8-13 Predicted $H-y_h$ responses predicted by conventional models compared with measured for piles in centrifuge soils (a) CS0, and (b) CS5
Figure 8-14 Computed lateral responses with the measured values for 340-mm pile: (a) bending moment diagram at $H = 120$ kN, and (b) deflection diagram at $H = 120$ kN
Figure 8-14a includes the computed bending moment diagram and deflection diagram of 340-mm pile at a lateral load of 120 kN. The derived peak moments are greater than the value calculated by the Reese’s weak rock model, which is less than the value calculated by the $P_y$ curves of stiff clay with $p_{u-cp}$. Similar observations can be found for the depths of maximum bending moment and zero bending moment as that for peak moments. As can be seen from the deflection diagram in Figure 8-14b, the largest ground line deflection and deepest effective depth of the pile reflect the softest lateral response predicted by the stiff clay with $p_{u-cp}$; in contrast, the weak rock model produces a diagram with the smallest magnitude of pile deflection.

8.3.5. Comments on the comparisons

The Brinch-Hansen $c-\phi$ soil model provides reasonable estimates of the ultimate lateral stress ($P_u$) in the weakly cemented (centrifuge) soils, but greatly over-predicts $P_u$ in the stronger $c-\phi$ soil of Pinjar. The Reese weak rock model and Lee et al. CPT $q_c$-based sand provide the relatively accurate estimates of $P_u$ for Pinjar soils and weak rocks.

The CPT $q_c$-based sand model by Lee et al. estimates ultimate lateral pressures that are in good agreement with the back-calculated values for Pinjar soils with no free water, highlighting the potential of the parameter $q_c$ for describing $P_u$. Though it under-estimates ultimate lateral pressures for the submerged centrifuge soils.

For un-cemented siliceous sand CS0, the API sand model estimates a slightly lower limiting resistance $P_u$, but gives a marginally stiffer initial slope for the $P_y$ curve than the $c-\phi$ soil model proposed by Reese and Van Impe. The lateral load capacity is under-predicted by this $c-\phi$ model. For cemented sands CS2.5 and CS5, the API soft clay model with $p_{u-cp}$ prediction gives a softer initial $P_y$ curve slope than that of the other two models.

From evaluation of the conventional $P_y$ criteria and existing CPT $q_c$-based $P_y$ criteria, the CPT $q_c$-based $P_y$ criteria appear suitable for weak $c-\phi$ soils. It is noted that the existing CPT $q_c$-based $P_y$ criteria adopt a limiting ultimate lateral resistance, while the measured data show that a limit does exist.
The Reese $p-y$ formulation of weak rock is assessed to be a reasonable approximation for the strong cemented soils and weak rocks at Pinjar site to predict lateral response of test piles.

8.4. Correlations of $E_s$, $P_u$ with $q_c$

Although the Pinjar and centrifuge soils are quite different types of cemented materials, this section examines trends combining both sets of measurements. It should be noted that although the Pinjar soil is unsaturated ($S_r=0.2$), the UCS strengths inferred from triaxial tests on fully saturated samples of this material were closely comparable to direct measures of UCS on unsaturated samples. It would therefore appear that the possible presence of suction within the unsaturated in-situ Pinjar soil has little or effect on the shear strength. Furthermore, there were clearly no inclined low strength discontinuities at Pinjar and hence the comparison of this soil with the relatively homogeneous nature of the saturated centrifuge soils is justified.

Back-analysis of the CPT cone data presented in sections 6.2 and 7.2 indicates that the ratio of the operational soil modulus $E_s$ to the cone penetration resistance $q_c$, $E_s/q_c$, at the Pinjar field test was about 12 while that derived from the centrifuge laboratory soils was 3 to 4. The values of $E_s$ (take an example of a depth of 3 m) are plotted against corresponding $q_c$ values in Figure 8-15. The $E_s/q_c$ ratio is seen to generally increase with $q_c$ (and essentially also with the $c$ component of strength).
A regression analysis of the $E_s$ and $q_c$ data of the same depths yields the following simple $q_c$-$E_s$ correlation, which is also plotted in Figure 8-15

$$E_s = (0.3 \pm 0.05) (q_c/p_a)^{1.6} p_a$$ (8.1)

in which $p_a$ is the reference pressure, taken to equal atmospheric pressure (100 kPa or 0.1 MPa). This correlation indicates that the operational modulus increases very strongly with the cone resistance. Further data on materials with levels of cementation in between the centrifuge samples and the weak rock at Pinjar would help refine Equation (8.1).

A relationship between the ultimate lateral pressure and the CPT $q_c$ value was also investigated for all cemented soils in this study, bearing in mind that this relationship would potentially show a dependence on the relative depth ($z/D$). Various correlations were trialed and the following best fit relationship was obtained, which was independent
of \( z/D \).

\[
P_u = (0.9 \pm 0.3) \left( \frac{q_c}{P_u} \right)^{0.66} P_u
\]  

(8.2)

The accuracy of this relationship can be assessed on Figure 8-16a, which plots the variation of \( P_u/q_c^{0.66} \) (units for \( P_u \) and \( q_c \) in MPa) with depth \( z/D \) for the four soils investigated. It is seen that there is relatively high scatter in the \( P_u/q_c^{0.66} \) but this scatter shows no systematic relationship with the level of soil cementation or the \( z/D \) ratio.

Figure 8-16b presents the variation of \( P_u \) with \( q_c \) for a selected representative depth of 3 m. The slower rate of increase of \( P_u \) as \( q_c \) increases is apparent. Figure 8-16b also includes \( P_u \) values estimated using the \( P_u/q_c \) ratio of 0.41 proposed correlation proposed by Dapp et al. 2011 for weakly cemented loess soil (\( c' \) of 8 to 35 kPa). The measured \( P_u/q_c \) ratios are compared in Table 8-10 with proposals of Dapp et al. (2011) and Schmertmann (1978).

| Table 8-10 Values of \( P_u/q_c \) ratio obtained and those reported |
|-----------------|-------|-------|----------------|------------------|
|                 | CS0   | CS2.5 | CS5            | Reference            |
| \( P_u/q_c \)   | 0.34  | 0.32  | 0.30           | 0.15               | Observed correlations in this study |
| \( P_u/q_c \)   | -     | 0.41  | 0.41           | 0.41               | Reported by Dapp et al. (2011) |
| \( P_u/q_c \)   | 0.11a | 0.37b | 0.37b          | 0.55c              | Proposed by Schmertmann (1978) |

\( ^a \) For dense sand;  
\( ^b \) For silt;  
\( ^c \) For clay on average;
Figure 8-16 Observations of $q_c$-$P_u$ relation: (a) variation of $P_u/q_c$ with depth $z/D$; and (b) $q_c$-$P_u$ relation for shallow soils
8.5. Proposed p-y relationship

The back-calculated P-y curves in section 8.2 have demonstrated the capability of simulating the lateral responses for the test piles in the present study (and the history case of uncemented sand in Blessington). However, the material constants in Equations (4.5), (7.1) to (7.3) are difficult to ascertain for general cemented soil conditions. A simple CPT q_c-based linear elastic-plastic approach is proposed here based on the derived correlations of $E_s$, $P_u$ with $q_c$ data.

Figure 8-17 presents the proposed methodology for constructing p-y curve for c-φ soils; this simple elastic-plastic p-y curve can be estimated from two components: the soil spring stiffness ($E_{py}=p/D$) and the limiting resistance of the soil ($P_u$). The ultimate soil resistance is determined using Equation (8.2). The p-y curve slope $E_{py}$ can be approximated using correlations of modulus of subgrade reaction and soil modulus suggested by Poulos (1971) and Poulos and Davis (1980) and in which the p-y slope $E_{py}$ is approximately $0.8\pm0.05E_s$; and $E_s$ may be determined using Equation (8.1).

![Figure 8-17 Proposed methodology for constructing p-y curve for general c-φ soils](image)

Figure 8-17 Proposed methodology for constructing p-y curve for general c-φ soils
The validity of the proposed $p$-$y$ formulation was investigated using a program LAP (Doherty 2014) to predict the load-displacement responses for all the piles tested. The program LAP represents the pile as a series of beam elements and the soil as a series of non-linear, non-interacting $p$-$y$ springs located between each beam element but also allows pile yielding. The program is similar to many commercially available laterally loaded pile programs; its accuracy was verified by a parallel series of calculations for the Pinjar piles performed by Oasys ALP. The $p$-$y$ curves were computed using the methodology described in Figure 8-17, in which the $E_{py}$ and $P_u$ values were evaluated directly from the $q_c$ data. The value of $E_{py}/E_s$ ranges from 0.75 to 0.82.

8.5.1 Validation against tests in the present study

It is seen on Figure 8-18 that the predicted lateral responses for all the model piles with a normalized embedment $L_e/D$ of 10 in the centrifuge soils agree well with the experimental data. For the piles with the shorter embedment $L_e/D$ of 5, however, the initial stiffness is significantly under-predicted although the lateral capacities are close to the measured values. Figure 8-18 indicates that the proposed $p$-$y$ curves based on the proposed methodology can generally well predict the lateral capacity of centrifuge model piles in test soils under lateral loading, which indicates the accuracy of proposed $p$-$y$ curves for predicting lateral pile capacity. However, at the same values of correlation factors, the soil-pile displacements predicted for soils CS2.5 and CS0 show less perfect agreement with the measures than for soil CS5.
Chapter 8 Results and discussion

(a) Experimental, $L_e/D=10$

(b) Measured $L_e/D = 10$
Figure 8-18 Comparison of test results and predictions using proposed p-y curves for centrifuge model piles: (a) CS0; (b) CS2.5; and (c) CS5 (in prototype dimension)

Figure 8-19 compares the measured and predicted lateral H-\(y_h\) responses at Pinjar. The average CPT \(q_c\) value was used for illustrative purposes (although clearly the stiffer piles of Pinjar were associated with high \(q_c\) values while the softer piles were associated with low \(q_c\) values; see Figure 3-21 and Figure 3-22. The estimated pile deflections are generally comparable to the measured values apart from the stiffest pile, 450A. In addition, the formulation clearly cannot capture the very high initial stiffness of the 340 mm diameter piles.

Good agreement between the foregoing predictions and the measured on Figure 8-18 and Figure 8-19 confirm the suitability of the simplified approach for both strong and weak c-\(\phi\) soils, noting that it is only likely to provide an approximation of the lateral pile stiffness because of the assumption of a linear pre-yield soil modulus.
Figure 8-19 Comparison of test results and predictions using proposed P-y curves for Pinjar full-scale piles: (a) 340 mm diameter; and (b) 450 mm diameter.
8.5.2 Validation against an independent centrifuge test

The validity of combining the data from the centrifuge piles and the Pinjar piles, leading to Equations (8.1) and (8.2), is examined here using a separate case history.

Erbrich (2004) presents the results from a lateral load test on a centrifuge pile in cemented soil. The soil comprised a mixture of two different carbonate sands with 10.5% gypsum and was expected to have similar characteristics to calcarenites found in offshore Australia. The CPT $q_c$ values at prototype scale increase from about 5 MPa at shallow depth to 12.5 MPa at 20 m depth (Figure 8-20a); these $q_c$ values fall between those in the UWA centrifuge tests (Figure 3-4) and in the Pinjar tests described above (Figure 3-21 and Figure 3-22). The test pile at prototype scale was 1.8 m in diameter with a wall thickness of 138 mm and, 20 m long with a yield moment of 80 MN.m and the lateral load was applied at an eccentricity of 2 m.

As for the case shown on Figure 8-20b, the pile response was predicted using LAP with p-y curves generated along the length of the pile using the $q_c$ data and Equations (8.1) and (8.2). The calculated response is plotted on Figure 8-20b, where it is seen to provide a good match to the observed lateral pile stiffness and capacity.
Chapter 8 Results and discussion

Figure 8-20 (a) CPT $q_c$ measures for centrifuge model test sample 1 (Erbrich, 2004); (b) LAP prediction and measured pile head load-displacement relationship of centrifuge model test 1, monotonic (after Erbrich, 2004)
8.6. Summary

The derived P-y curves in Pinjar field test and centrifuge model tests have been summarized and examined to be capable of predicting the lateral deformation of the full scale piles of pre-yielding and giving lateral capacities of model piles with satisfaction. The P-y relationships also replicate bending moment data in general errors ±13% for full-scale piles and small-scale model test piles.

The derived P-y relationships has also been shown to be applicable for other similar soils; however, the constants in the equation governing the overall slope of the P-y curve heavily rely on the specific soil properties, subjecting to certain adjustment.

From evaluation of conventional P-y criteria and existing CPT-q_c based P-y criteria, the CPT-q_c based P-y criteria is considered to be suitable for c-φ soils, the predicted lateral capacities closely approach to the measured values. Although CPT-q_c based P-y curves give less stiff initial slope yet stronger ultimate resistance capacity response than conventional P-y curves, the mathematical procedure to calculate P-y data is simply straightforward.

The correlations of operational soil modulus and cone resistance, as well as the ultimate soil resistance and cone resistance are obtained by combining data of Pinjar field tests and centrifuge model tests. These q_c-E_s and q_c-P_u correlations are comparable with some of empirical correlations reported.

A simple bi-linear CPT-based p-y approximation of lateral response is proposed for the analysis of laterally loaded piles in cemented materials. Validity of the proposed p-y approximation is examined with the present tests and an independent test. This formulation is shown to provide good predictions for lateral pile response in a variety of cemented deposits.
CHAPTER 9  SUMMARY AND FUTURE WORK

9.1 Thesis overview

This thesis describes an investigation into the performance of single piles embedded in cemented sands and weak rocks under lateral loading. This entailed gaining insights into:

- the geotechnical properties of cemented sands with various degrees of cementation;
- the lateral response of centrifuge-scale piles in variably cemented sands;
- the lateral response of full-scale D&G piles in weak rock (well-cemented sand).

These experimental results were then examined through:

- Comparison of the \( P-y \) relationships established in the field with existing theories and formulations;
- Comparison of the ultimate pressures assessed from the field and centrifuge scale piles with existing theories and formulations;
- Finite Element analyses of field and centrifuge tests assuming a simple elastic-perfectly plastic model with Mohr-Coulomb yield surface, where the elastic modulus was interpreted from the measured CPT \( q_c \) values using spherical cavity expansion theory;
- Use of all existing information to develop a simple methodology for the derivation of \( P-y \) curves for general \( c-\phi \) soils, using the in-situ CPT data.

9.2 Summary of major findings

The significant findings of this thesis are summarized as follows.
Triaxial tests on cemented sands confirm that the strength and stiffness increase with an increase in the degree of cementation degree as well as confining stress, whilst the strain at peak strength decreases with the degree of cementation. Very weakly cemented sands show a transitional response from brittle failure to ductile failure as the mean stress increases; the well-cemented sands and weak rocks show brittle failure behaviour with well-defined peak states under all confining stresses. Regression analyses on the normalised stress-strain curves of cemented soils show the characteristic stress-strain relationship up to peak stress is well-described using a hyperbolic tangent form.

For very weakly-cemented sands artificially prepared at cement contents of 2.5% and 5%, the effective cohesion ($c$) at peak strengths are revealed to be approximately 12 kPa and 30 kPa, respectively, with peak friction angles ranging $37^\circ$–$39^\circ$. At large strains, both cemented samples (and the un-cemented sample) approach a critical state as the bonding between the particles is progressively destroyed, with the determined critical friction angles ranging $32^\circ$–$34^\circ$. For the natural well-cemented sands encountered at Pinjar, the cohesion ($c$) was derived as approximately 270 kPa and the peak friction angle was $43^\circ$. At large strain, the weak rock sample also approaches a critical state and the critical friction angle was determined as $39^\circ$ under a low confinement of 50 kPa.

The cone tip resistance ($q_c$) is strongly affected by the degree of cementation. At high levels of cementation, the more brittle nature of the response is such that $q_c$ does not increase in direct proportion with the cohesion ($c'$).

Bender element tests on artificially cemented sands and SCPTs on naturally cemented sands suggest that the small strain shear modulus ($G_0$) increases with the level of cementation as well as with the confining stresses; the effect of confining stress is not significant for very well cemented materials. Average $G_0$ values of 230 MPa and 400 MPa are observed for samples with 2.5% and 5% cement respectively, while the $G_0$ value at Pinjar at the same
stress level is approximately 3400 MPa.

- The full-scale pile test at Pinjar confirmed that an increase in pile diameter significantly increases the lateral capacity as well as the initial stiffness.

- The relationships obtained at Pinjar indicate that the Reese (1997) weak rock method provides reasonable estimations of the lateral response but that the Brinch Hansen (1961) \(c-\phi\) soil model greatly over-predicts the stiffness and capacity in this material.

- The crack patterns observed near the soil surface in both centrifuge-scale model piles and full-scale piles suggest the shear resistance at the sides of the pile are fully mobilised ahead of the passive resistance at the front of the pile.

- The centrifuge-scale rigid piles tests show that the cementation of the soils as well as the embedment of the piles significantly enhances the lateral stiffness and capacity. It is shown that the lateral capacity of short piles (i.e. where the pile’s moment capacity is not exceeded) can be uniquely related to the gradient of the CPT \(q_c\) profile in an analogous way to the Broms (1964) formulations for capacities in clays.

- In contrast to the observations at Pinjar, the Brinch Hansen (1961) \(c-\phi\) soil model provided reasonable estimates of the ultimate lateral stresses in the artificially cemented centrifuge soils whereas the Reese weak rock model was seen to be inappropriate.

- The CPT \(q_c\) profiles can be approximated using the Yu and Houlsby (1991) spherical cavity expansion theory (applicable to a linear elastic Mohr Coulomb theory) and the simple relationship relating limit pressure to \(q_c\) proposed by Randolph et al. (1994). It has been shown that the operational elastic modulus corresponding to the measured CPT \(q_c\) values can be used to obtain a reasonable estimate of lateral pile response at typical working loads; 3D Finite element analyses of both the field and centrifuge scale tests verified this observation.
Back-analyses of CPT data using this methodology led to the derivation of a relationship with the CPT $q_c$ value of the operational modulus of cemented soils and weak rocks.

Ultimate lateral pressures on piles in cemented soils also correlate with cone tip resistance. Regression analyses of the assembled $P_u$ and $q_c$ data from the field and centrifuge tests led to a simple direct formulation with no apparent dependence on the relative depth ($z/D$).

A simple methodology to develop $p-y$ curves directly incorporating the in-situ cone penetration resistance $q_c$ data for general $c-\varphi$ soils is proposed. It is shown to provide a good match to the experimental data obtained in this thesis as well as independent test data of cemented soils having intermediate $q_c$ values.

### 9.3 Recommendations for future research work

This thesis has provided insights into the lateral response of single piles in cemented sands with various levels of cementation. Some areas requiring further research are as follows.

A literature review of piles subjected to lateral loads in cemented soils/weak rocks showed that this is an area with an extreme shortage of experimental data. This thesis addresses the shortage but clearly there is a significantly more testing required. The future construction of offshore wind farms in the chalk in southern England is one example of why such research is required.

The CPT back-analysed soil properties have been used input to a simple laterally loaded pile analysis. Further field and/or centrifuge tests are required to validate this approach. These tests should examine the influence of embedment, pile diameter, cyclic loading and load eccentricity.

Future research should also focus on obtaining better estimates of the initial
lateral stiffness of piles in cemented soils, as this stiffness is often controlling the design for wind turbines founded on monopiles. The $G_0$ data inferred from SCPTs and bender elements would provide the basic geotechnical input.

- No strain gauging was used on the centrifuge scale model piles employed in this thesis. While the absence of gauging facilitated the derivation of ultimate lateral stresses (as the piles used were rigid), future research should consider using gauging to enable derivation of $p-y$ curves from measured moment distributions.
APPENDIX A  Test results of Pinjar field experiment

A-1  Observations of Pinjar field test

Figure A-1 to Figure A-13 present the experimental results obtained from Pinjar field test, including loading with time, strain variation with load increasing, and back-calculated pile stiffness for the two diameters of 340 mm and 450 mm.

Figure A-14 to Figure A-17 show the photos of the test piles, test setup and observed soil/pile failure patterns at ground as well.

Figure A-19 to Figure B-1 list the stress-strain responses measured from laboratory UCS tests on the chunk samples of Pinjar field material.
Appendix A. Test results of Pinjar field experiment

Figure A-1 Lateral load vs. time of pile calibration tests on pair of 340 mm piles, load eccentricity $e = 0.9$ m

Figure A-2 Lateral load vs. time of pile load tests on pair of 340 mm piles, load eccentricity $e = 0.3$ m
Figure A-3 Strain variation with time for pile calibration tests of (a) 340A; and (b) 340B
Figure A-4 Strain variation with time for pile calibration tests of (a) 340A; and (b) 340B
Figure A-5 Moment with curvature data of 340 mm piles (b) diameter of 340 mm having grout cover; (b) diameter of 219 mm having no grout cover.
Appendix A. Test results of Pinjar field experiment

Figure A-6 Moment with strain data of 340 mm piles (b) diameter of 340 mm having grout cover; (b) diameter of 219 mm removing grout cover
Appendix A. Test results of Pinjar field experiment

Figure A-7 Relationship between flexural stiffness and moment of 340 mm piles (b) diameter of 340 mm having grout cover; (b) diameter of 219 mm removing grout cover.
Appendix A. Test results of Pinjar field experiment

Figure A-8 Lateral load vs. time of pile calibration tests on pair of 450 mm piles, load eccentricity $e = 0.9$ m

Figure A-9 Lateral load vs. time of pile load tests on pair of 450 mm piles, load eccentricity $e = 0.2$ m
Figure A-10 Strain variation with time for pile calibration test on pair of: (a) 450A; and (b) 450B
Appendix A. Test results of Pinjar field experiment

Figure A-11 Strain variation with time for pile calibration tests of (a) 450A; and (b) 450B
Appendix A. Test results of Pinjar field experiment

Figure A-12 Strain variation with time for pile calibration tests of (a) 450A; and (b) 450B

\[
\begin{align*}
\kappa &- M, \text{ with grout cover } D = 450 \text{ mm} \\
\text{Regression:} & \\
y & = 27348x \\
R^2 & = 0.9858 \\
\text{Regression:} & \\
y & = 29720x \\
R^2 & = 0.9839 \\
\end{align*}
\]
Figure A-13 Strain variation with time for pile calibration tests of (a) 450A; and (b) 450B.
Figure A-14 Steel pipes used for D&G piles: (a) view of pipe head; and (b) view of pipe foot
Figure A-15 340 mm piles: (a) setup at commencement of lateral load test; and (b) pile displacement upon completion of test
Figure A-16 450 mm piles: (a) setup at commencement of lateral load test; and (b) pile displacement upon completion of test
Figure A-17 Observed soil/pile cracks at ground: (a) 450A; and (b) 450B
Appendix A. Test results of Pinjar field experiment

Figure A-18 UCS tests on chunk well-cemented sand samples: (a) - (h) stress-strain responses
Appendix A. Test results of Pinjar field experiment

Figure A-19 UCS tests on chunk well-cemented sand samples: (a) - (h) stress-strain responses
Appendix A. Test results of Pinjar field experiment

A-2 Calculation of pile capacity

Table A-1 lists the pile property including pile elasticity and pile plasticity.

Table A-1 Calculation of pile capacity

<table>
<thead>
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<th>Mechanical properties of grout and steel pipe</th>
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<tr>
<td>Modulus $E_{\text{grout}} = 5.5$ GPa</td>
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<td>Modulus $E_{\text{steel}} = 200$ GPa</td>
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</table>

<table>
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<th>Flexural rigidity</th>
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<th>$D_{\text{core}}$ (mm)</th>
<th>$D_{\text{steel}}$ (mm)</th>
<th>$t$ (mm)</th>
<th>$I_{\text{cover}}$ (m$^4$)</th>
<th>$I_{\text{core}}$ (m$^4$)</th>
<th>$I_{\text{steel}}$ (m$^4$)</th>
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<th>$EI_{\text{core}}$ (kNm$^2$)</th>
<th>$EI_{\text{steel}}$ (kNm$^2$)</th>
<th>$EI$ (kNm$^2$)</th>
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<td>7218</td>
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<td>0.00068</td>
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<td>21253</td>
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<th>$Z_c$ (m$^3$)</th>
<th>$Z_{\text{steel}}$ (m$^3$)</th>
<th>$M_{y,\text{cover}}$ (kN.m)</th>
<th>$M_{y,\text{core}}$ (kN.m)</th>
<th>$M_{y,\text{steel}}$ (kN.m)</th>
<th>$M_y$ (kN.m)</th>
<th>$M_{p,\text{cover}}$ (kN.m)</th>
<th>$M_{\text{core}}$ (kN.m)</th>
<th>$M_{u,\text{steel}}$ (kN.m)</th>
<th>$M_p$ (kN.m)</th>
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<td>77.2</td>
<td>90</td>
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<td>10.9</td>
<td>7.9</td>
<td>209.2</td>
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<td>13.4</td>
<td>271.1</td>
<td>300</td>
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APPENDIX B  Photos of centrifuge-scale model test

Figure B-1 Centrifuge-model tests: (a) a photo of in-situ miniature CPT test; and (b) a photo of lateral load test
APPENDIX C  Back-analysis of CPT data

C-1  Cavity expansion-based CPT analysis

Closed-form solution for determination of the limiting pressure $p_{\text{lim}}$

Closed form solutions proposed by Yu and Houlsby (1991) were used to evaluate limit pressure for spherical cavity expansion. Procedures used for calculating the limit pressure are given as the following:

(a)  Choose soil input parameters $E$ (soil Young’s modulus), $\nu$ (Poisson’s ratio), $c’$ (cohesion intercept), $\phi$ (internal friction angle), $\psi$ (the dilatancy angle) and (in-situ effective stress) $p_o$, and parameter $m$ ($m = 2$) indicating the spherical analysis. Parameter $m$ is used to indicate cylindrical analysis ($m = 1$) or spherical analysis ($m = 2$). The initial radius of the cavity is referred as $a_0$, whilst the current radius during cavity expansion referred as $a$.

(b)  Calculate the derived parameters:

$$G = \frac{E}{2(1+\nu)} \quad \text{(C -1)}$$

$$M = \frac{E}{1-\nu^2(2-m)} \quad \text{(C -2)}$$

$$\gamma = \frac{2c\cos\phi}{1-\sin\phi} \quad \text{(C -3)}$$

$$\alpha = \frac{1+\sin\phi}{1-\sin\phi} \quad \text{(C -4)}$$

$$\beta = \frac{1+\sin\psi}{1-\sin\psi} \quad \text{(C -5)}$$

$$\gamma = \frac{\alpha(\beta + m)}{m(\alpha - 1)\beta} \quad \text{(C -6)}$$
\[ \delta = \frac{Y + (\alpha - 1)p_0}{2(m + \alpha)G} \]  
\[ \eta = \exp \left\{ \frac{(\beta + m)(1 - 2\nu)[Y + (\alpha - 1)p_0][1 + (2 - m)\nu]}{E(\alpha - 1)\beta} \right\} \]  
\[ \xi = \frac{[1 - \nu^2(2 - m)][1 + m]\delta}{(1 + \nu)(\alpha - \beta)} \left[ 2\beta + \nu - \frac{m\nu(\alpha + \beta)}{1 + \nu(2 - m)} \right] \]  
\( C -7 \)  
\( C -8 \)  
\( C -9 \)  
\( C -10 \)  
\( C -11 \)  
\( C -12 \)  
\( C -13 \)

(c) For pressure \( p \) less than the pressure \( p_1 \), \( p_1 = 2mG\delta + p_0 \), required to initial plasticity, calculate the current cavity radius from a small-strain elastic expansion \( (a - a_0)/a_0 = (p - p_0)/2mG \).

(d) For a given value of \( p \) (greater than \( p_1 \) and less than the limit pressure \( p_{\text{lim}} \)), calculate the cavity pressure ratio \( R \), which is defined by the following equation:

\[ R = \frac{(m + \alpha)[Y + (\alpha - 1)p_0]}{\alpha(1 + m)[Y + (\alpha - 1)p_0]} \]  
\( C -10 \)

(e) The explicit expression for the pressure – expansion relationship is given by the following equation:

\[ \frac{a}{a_0} = \left\{ \frac{R^\gamma}{(1 - \delta)^{(\beta + m)\gamma} - (\gamma / \eta)\Lambda_1(R, \xi)} \right\}^{\frac{\beta(\beta + m)}{(\beta + m)\gamma}} \]  
\( C -11 \)

where

\[ \Lambda_1(x, y) = \sum_{n=0}^{\infty} A_n^1 \]  
\( C -12 \)

in which

\[ A_n^1 = \begin{cases} 
\frac{y^n}{n!} \ln x & \text{if } n = \gamma \\
\frac{y^n}{n!} \left[ x^{n-\gamma} - 1 \right] & \text{otherwise} 
\end{cases} \]  
\( C -13 \)
Evaluate sufficient terms in the finite series to obtain an accurate value of $\Lambda_i$—a few terms will usually be sufficient.

(f) Evaluate $a/a_0$ from the pressure–expansion relationship and if necessary the pressuremeter displacement $u = a - a_0$ or the loop strain at the cavity surface $\varepsilon = \ln(a/a_0)$.

The procedure from (d) to (f) can be repeated to construct the complete cavity pressure–expansion relationship. By putting $(a/a_0) \to \infty$ in Equation (C-11), the limit cavity pressure may be obtained.

*Relate the limiting pressure $p_{\text{lim}}$ to cone tip resistance $q_c$* (Randolph et al. 1994)

The correlation between the end–bearing pressure of driven piles and the limit pressure of a spherical expansion proposed by Randolph et al. (1994), was used to evaluate the cone tip resistance. The relationship between the limit pressure, $p_{\text{lim}}$, and cone tip resistance, $q_c$, was expressed as the following:

$$q_c = q_b = p_{\text{lim}}(1 + \tan \phi' \tan \alpha)$$

where $\alpha$ equals to $60^\circ$ for a standard cone penetrometer and $\phi$ is the internal friction angle of the soil.

Computation of the $p_{\text{lim}}$ and $q_c$ in aforementioned procedure can be implemented through a written program in either spreadsheet or Matlab environment. Both approaches would produce the similar results.

**C-2 Data of the CEM $q_c$ predictions**

Table C-1 through to Table C-3 tabulate computed $p_{\text{lim}}$ and $q_c$ predictions, varying soil stiffness of lower bound, best estimate, and upper bound, for CPT in centrifuge soil samples CS0, CS2.5 and CS5 respectively. Table C-4 lists $p_{\text{lim}}$ and $q_c$ predictions of CPT for the Pinjar field test.
### Table C-1 CEM estimated $p_{lim}$ and $q_c$ of CPT in soil sample CS0

<table>
<thead>
<tr>
<th>Soil sample</th>
<th>Lower bound</th>
<th>Best-estimate</th>
<th>Upper bound</th>
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<td>$q_c$ (kPa)</td>
<td>$q_c$ (MPa)</td>
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### Table C-2 CEM estimated $p_{lim}$ and $q_c$ of CPT in soil sample CS2.5

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### Table C-3 CEM estimated $p_{lim}$ and $q_c$ of CPT in soil sample CS5

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### Table C-4 CEM estimated $p_{lim}$ and $q_c$ values for of CPT for field test

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<th>Best estimate (BE)</th>
<th>Upper bound (UB)</th>
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APPENDIX D  Data processing

D-1  Curve fitting of bending moment data

A composite product equation of an \( n \)-th order polynomial multiplied by an exponential function was used to fit the moments \( M_z \) measured at the strain gauges for Pinjar field test, as well as FEM estimated moment data for centrifuge-scale model tests. The algebraic equation of this production expression is given as:

\[
M_z = \left( 1 - \frac{1}{e^{a_i z} + 1} \right) \left( \sum_{i=1}^{n} a_i z^{i-1} \right)
\]  \hspace{1cm} (D-1)

in which \( a_i \) are the curve fitting coefficients, and \( z \) is the depth. The order \( n \) of the polynomial function component was typically 6 to 7, depending on the quantity of discrete data points. Curve fitting procedure using this equation had first been validated using a propped cantilever, and then successfully applied in practice by analyzing the experimental data of bending moment and pile deflection of single piles subjected to lateral and combined axial and lateral loads (Phillips 2002). Very good agreement between the experimental data and fitted profiles of bending moment and deflection slopes of the test piles were obtained. It suggested that the equation, subjecting to appropriate boundary constrains, provided excellent results. In the present study, curve fitting using this equation was mostly employed to fit the data of bending moments.

The polynomial function has frequently been used in curve fitting for bending moment data (Ashford and Juirmarongrit 2005; Dou and Byrne 1996; Doyle et al. 2004; Ismael 1990; Kong and Zhang 2007; Matlock and Ripperger 1956; Yang 2006). The exponential model is used to successfully model the decay phenomena of moment, shear force and/or deflection over the embedded depth of pile typically occurred for long flexible piles (Phillips 2002). Equation (D-1) has also proved to be applicable for curve fitting of moment data of short rigid piles in the present study with centrifuge model test.

The shear force in the pile is the first differential of bending moment. Differentiating
once the Equation (D-1) gives:

\[
SF = \frac{dM}{dz} = \left( \frac{a_i e^{a_i z}}{(e^{a_i z} + 1)^2} \right) \left( \sum_{i=1}^{n} a_i z^{i-1} \right) + \left( 1 - \frac{1}{e^{a_i z} + 1} \right) \left( \sum_{i=2}^{n} (i-1) a_i z^{i-2} \right)
\] (D-2)

The soil reaction is the second differential of bending moment. Differentiating twice the Equation (D-1) gives:

\[
p = \frac{d(SF)}{dz} = \frac{d^2 M}{dz^2} = \left( \frac{a_0^2 e^{a_0 z}}{(e^{a_0 z} + 1)^2} \right) \left( \sum_{i=1}^{n} a_i z^{i-1} \right) + \left( 1 - \frac{1}{e^{a_0 z} + 1} \right) \left( \sum_{i=3}^{n} (i-1)(i-2) a_i z^{i-3} \right)
\] (D-3)

Different boundary constraints were assumed at pile head or soil surface. For free-head piles, at soil surface, \( z = 0 \), the following boundary constrains may be known:

1. At soil surface, \( z = 0 \), the shear force, \( SF \), the first derivative of \( M_z \), is equal to the applied lateral load, \( SF = H \);
2. The bending moment, \( M_o \), at soil surface, \( z = 0 \), can be calculated as the applied load, \( H \), multiplied by the eccentricity, \( e \), of load, \( M_o = He \);
3. At soil surface where \( z = 0 \), the soil reaction, \( p \), will be generally negligible small and is assumed to be zero, \( p = 0 \).

Applying these boundary constraints into the equations, it will fix the following coefficients:

\[
a_1 = 2M_o
\]

\[
a_2 = 2H - \frac{a_0 a_1}{2} = 2H - M_o a_0
\]

\[
a_3 = -\frac{a_0 a_2}{2} = \frac{M_o}{2} a_0^2 - Ha_0
\]

The remaining coefficients in Equation (D-1) were determined by iteration process until a best-fit curve through the points of measured result data was obtained. A simple least squares method was used to evaluate the goodness of curve fitting. The procedure was
then repeated for any selected incremental lateral loads applied to piles to obtain the set of coefficients of each best-fit curve for its corresponding bending moment distributions. Thus, a continuous bending moment profile with depth was obtained.

Figure D-1 shows typical fitted bending moment profiles to the data of measures and computed shear force, and soil reaction profiles. Figure D-2 shows typical fitted bending moment profiles to the data of FEM estimations and computed shear force, and soil reaction profiles.

The lateral displacement, $y$, and rotation, $\theta$, at any depth can be computed by integrating the bending moment profile, according to the equations:

$$\theta_z = \int \frac{M}{EI} dz \quad (D-4)$$

$$y_z = \int \left( \int \frac{M}{EI} dz \right) dz \quad (D-5)$$

in which $EI$ is the pile flexural rigidity, and $z$ is the depth. In case of the $D&G$ piles tested in field experiment, displacements could not be estimated with accuracy from double integration using Equations (D-4) and (D-5) due to the non-linear pile flexural rigidity coupled with the imprecise nature of the $M$ profiles. In case of the rigid piles for centrifuge model experiments, the soil displacements could be estimated if the rotation degree at pile head is known by measurement, the measured rotation at soil surface is a boundary condition required for integrating the moment data.

For curve fitting of moment data of rigid piles, alternative techniques of curve fitting, such as a single polynomial function and piece-wise polynomial, were practiced to compare the difference between fitted profiles. It was found that the fitted profiles at these various methods were comparable each other. A single polynomial function seemed to be more convenient for processes of differentiation and integration among these methods. Exponent function would complicate the integrating process even though it can work well for differentiation. The piece-wise polynomial method would require additional techniques for splicing together the segments of profiles, and sufficient data points have to be available to implement this method.
Appendix D. Data processing

Figure D-1 Typical curve fitting results of a long flexible pile

Figure D-2 Typical curve fitting results of a short rigid pile
Appendix D. Data processing

D-2 Least squares method of curve fitting

A simple least-squares method was used to evaluate the errors between the fitted moments using Equation (D-1) and measured moments $M_i$. The best fit was achieved by minimizing the sum of squares of errors (difference), $SSE$, satisfying the requirement:

$$\min SSE = \min \sum_{i=1}^{m} [M_i - M(z, a)]^2 = \sum_{i=1}^{m} [M_i - M(z_i, \hat{a})]^2$$  \hspace{1cm} (D-6)

in which $a_i$ is the coefficients to be determined, and $\hat{a}_i$ is the determined coefficients with optimization.

The squared-value, $R^2$, the coefficient of determination, is a number that indicates how well data fit a specified model. In the present study, the $R^2$ value was defined as:

$$R^2 = 1 - \frac{SSE}{SST} = 1 - \frac{\sum_{i=1}^{m} [M_i - M(z_i, \hat{a})]^2}{\sum_{i=1}^{m} (M_i - \bar{M})^2}$$  \hspace{1cm} (D-7)

in which $SST$ is the sum of squares of the deviations about the average value, and $SST = \sum_{i=1}^{m} (M_i - \bar{M})^2$. The coefficient of determination, $R^2$ value, ranges from 0 to 1, with a value closer to 1 indicating that a greater proportion of variance is accounted for by the specified model. For example, an $R^2$ value of 0.90 means that the fit explains 90% of the total variation in the data about the average. In data processing of the bending moments for the present study, the $R^2$ values were generally over 0.99, suggesting the goodness of the model employed.
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