Experimental investigation on the behaviour of laterally loaded piles in soft clay, sand and residual soils

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(BE, BSc, MSc)

This thesis is presented for the degree of Doctor of Philosophy

School of Civil, Environmental and Mining Engineering
(Geotechnical Engineering)

2017
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Date: 3/7/2017
Thesis Abstract

The $P-y$ curve method is the current industry standard for predicting the behaviour of laterally loaded piles and is favoured for its ease of use. However, there are a number of shortcomings with the current recommendations. One, which has been identified in monopile designs for offshore wind turbines, is how the current methods were developed based on limited field tests on long slender piles and its application for shorter rigid to semi-rigid piles are therefore questioned. This stands for rigid lateral piles in clay and sand under static and cyclic loading. Another issue with current standards is the absence of design methods for lateral piles in residual soils which have been proven to behave differently to typical sedimentary soil deposits of sand and clay. The motivation for this research is to add a number of new experimental tests of short piles to the literature and to provide further insights on the behaviour of laterally loaded piles in soft clay, sands and residual soils.

This research firstly looks into soft clays where the ultimate net soil resistive pressures have long been of concern with various authors finding the industry recommendations to be conservative. An initial 3D numerical study was conducted with a linear elastic perfectly plastic soil model which verified the potential of a CPT based $P-y$ curve. Drum centrifuge tests carried out by others to investigate the behaviour of a circular, square and H section pile were analysed and T-bar based $P-y$ curves were derived with corresponding shape factors which show that square and H piles improves the lateral capacity of a circular pile of the same width. 3D numerical analyses were employed to verify the trends observed in the centrifuge. The importance of using the correct undrained strength measurements is also discussed as different modes of shearing results in different shear strengths. Cyclic load tests in the centrifuge also revealed the API (2011) recommendations to reduce the lateral pile capacity of cyclically loaded piles in clay to be conservative as the pile capacity was mostly unchanged due to cycling.
In sands, there already exists a number of improved methods for predicting lateral piles under static loads and therefore studies in this research looks at furthering the current investigations on cyclic loading. A number of published centrifuge test data for cyclic loading in dense sands are available and these tests have indicated that pile rotation accumulation is highly dependent on the cyclic load characteristics and that sequences of increasing one-way cyclic load packages can be predicted reasonably well by a strain superposition method. Additional centrifuge tests in medium dense sand were conducted and conclude that pile rotation accumulation is more significant compared to dense sand, cyclic sequences with packages of two-way loads reduce pile rotation, and post-cyclic pile capacity is dependent on soil density and residual pile rotation.

Field tests were conducted in a residual soil site using a pair of small scale instrumented steel pipes to derive CPT based $P$-$y$ curves. The difficulty in characterising the residual soil strength using standard laboratory tests is presented and suggests that the CPT method is more appropriate as in-situ properties such as matric suctions and natural saturation levels are captured. The derived CPT method was able to predict lateral pile behaviour of existing case histories and found that the applicability of the method is dependent on the CPT based soil behaviour type classifications.

This research has taken further steps to progress in the field of lateral pile behaviour by providing new observations in cyclic lateral pile behaviour in sands, new $P$-$y$ curve correlations for lateral piles in soft clay and a first in CPT based $P$-$y$ curves for residual soils with proven success to predict a variety of case histories.
Acknowledgements

It is acknowledged that this research was supported by an Australian Government Research Training Program (RTP) Scholarship (formerly the Australian Postgraduate Award), The Australian Research Council (ARC), and Arup Pty Ltd.

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Thank you to all the UWA academics that had assisted me with the experiments, Dr Jit Kheng Lim, Dr Fengju Guo, Marcel Chee, Jessica de Paula and Le Viet Doan. Special thanks to visitor Dr Wen-Gang Qi for assisting in the centrifuge testing when I was not available. To all the SCEME staff Dr James Doherty, Prof. Yuxia Hu, Prof. Liang Cheng, and Prof. Anas Ghadouani for the support and positivity over the years.

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You have all contributed in a way and now, I have Phinished!!
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(Submitted to Géotechnique, currently under review)

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**Location in thesis:**
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<td>18-07-2017</td>
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<td></td>
<td>19-07-2017</td>
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**Details of the work:**
CPT q, derived P-y curves for laterally loaded piles in residual soil
(Submitted to the Journal of Geotechnical and Geoenvironmental Engineering)

**Location in thesis:**
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Nomenclature

Abbreviations

3D FEA  3 Dimensional Finite Element Analysis
API    American Petroleum Institute
CPT    Cone Penetration Testing
DTU    Technical University of Denmark
SCPT   Seismic Cone Penetration Test
SPT    Standard Penetration Test
UWA    University of Western Australia

Notations

$B$     pile width
$c'$    cohesion
$c_v$   coefficient of consolidation
$D$     pile diameter
$D_{50}$ grain size at 50% passing
$D_{eq}$ equal area equivalent diameter
$D_r$   relative density
$e$     load eccentricity from soil surface
$EI$    pile flexural rigidity
$E_p$   Young’s modulus of pile
$E_u$   undrained Young’s modulus of soil
$F_r$   CPT friction ratio
$G$     shear modulus
<table>
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<th>Definition</th>
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<td>$G_0$</td>
<td>small strain shear modulus</td>
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<tr>
<td>$G_s$</td>
<td>specific gravity</td>
</tr>
<tr>
<td>$H$</td>
<td>applied lateral load</td>
</tr>
<tr>
<td>$H_{1^\circ}$</td>
<td>reference horizontal load at $1^\circ$ pile rotation</td>
</tr>
<tr>
<td>$H_{\text{max}}$</td>
<td>maximum cyclic horizontal load</td>
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<tr>
<td>$H_{\text{min}}$</td>
<td>minimum cyclic horizontal load</td>
</tr>
<tr>
<td>$H_{\text{ref}}$</td>
<td>reference horizontal load</td>
</tr>
<tr>
<td>$H_{\text{ult}}$</td>
<td>ultimate lateral pile capacity</td>
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<tr>
<td>$I$</td>
<td>second moment of area of pile</td>
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<td>$I_r$</td>
<td>rigidity index</td>
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<tr>
<td>$J$</td>
<td>dimensionless empirical constant</td>
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<td>$k$</td>
<td>rate of increase of undrained shear strength with depth</td>
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<tr>
<td>$k_{T\text{bar}}$</td>
<td>rate of increase of $S_{u,T\text{bar}}$ with depth</td>
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<tr>
<td>$K_r$</td>
<td>pile rotational stiffness</td>
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<tr>
<td>$K_{r,N}$</td>
<td>pile rotational stiffness at N cycles</td>
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<td>$K_{r,\text{stab}}$</td>
<td>stabilised pile rotational stiffness</td>
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<td>$L$</td>
<td>pile embedment length</td>
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<td>$LL$</td>
<td>liquid limit</td>
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<td>$LS$</td>
<td>linear shrinkage</td>
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<td>$M$</td>
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<tr>
<td>$M_{\text{max,N}}$</td>
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<td>$N_{SPT}$</td>
<td>SPT blowcount</td>
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<td>$N_k$</td>
<td>undrained strength cone factor</td>
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<td>$N_{kt}$</td>
<td>undrained strength net cone factor</td>
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<td>lateral pile capacity factor $= P_u/s_u$</td>
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<td>lateral pile capacity factor $= P_u/s_{u,CU}$</td>
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<td>$N_{pq}$</td>
<td>CPT lateral pile capacity factor</td>
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<tr>
<td>OCR</td>
<td>over-consolidation ratio</td>
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<td>$p$</td>
<td>net lateral soil resistance per unit length</td>
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<td>$P$</td>
<td>net lateral soil resistive pressure</td>
</tr>
<tr>
<td>$P_1$</td>
<td>net soil resistive pressure at first peak load</td>
</tr>
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<td>PI</td>
<td>plasticity index</td>
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<tr>
<td>PL</td>
<td>plastic limit</td>
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<tr>
<td>$P_{max}$</td>
<td>maximum measured net lateral soil resistive pressure</td>
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<tr>
<td>$P_N$</td>
<td>net soil resistive pressure at peak load of $N^{th}$ cycle</td>
</tr>
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<td>$P_u$</td>
<td>ultimate net lateral soil resistive pressure</td>
</tr>
<tr>
<td>$P_{ultc}$</td>
<td>ultimate net lateral soil resistive pressure for cohesive component</td>
</tr>
<tr>
<td>$P_{ult\phi}$</td>
<td>ultimate net lateral soil resistive pressure for frictional component</td>
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<td>$q_c$</td>
<td>cone penetration resistance</td>
</tr>
<tr>
<td>$q_f$</td>
<td>ultimate deviator stress</td>
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Nomenclature

$q_{\text{max}}$  
maximum applied deviator stress

$q_{\text{net}}$  
net cone resistance

$q_{\text{UCS}}$  
unconfined strength

$r^2$  
correlation coefficient

$s$  
matric suction

$s_{\text{ae}}$  
suction at air entry

$S_r$  
saturation

$s_u$  
undrained shear strength

$s_{u,CU}$  
consolidated isotropic undrained triaxial shear strength

$s_{u,FE}$  
undrained shear strength specified in finite element analysis

$s_{u,op}$  
operational undrained shear strength

$s_{u,Tbar}$  
T-bar undrained shear strength

$s_{u,UU}$  
unconsolidated undrained triaxial shear strength

$u_2$  
pore pressure at cone shoulder

$u_a$  
pore air pressure

$u_w$  
pore water pressure

$y$  
lateral pile displacement

$y_N$  
lateral pile displacement at N cycles

$y_{\text{ref}}$  
reference pile displacement

$z$  
depth below soil surface
### Nomenclature

#### Symbols

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<td>accumulation coefficient</td>
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<tr>
<td>$\alpha_c$</td>
<td>cone roughness factor</td>
</tr>
<tr>
<td>$\alpha_p$</td>
<td>pile roughness factor</td>
</tr>
<tr>
<td>$\alpha_r$</td>
<td>accumulation coefficient, defined in terms of rotation</td>
</tr>
<tr>
<td>$\alpha_{ref}$</td>
<td>strength reduction factor</td>
</tr>
<tr>
<td>$\alpha_{triax}$</td>
<td>accumulation coefficient defined in terms of triaxial axial strain</td>
</tr>
<tr>
<td>$\alpha_y$</td>
<td>accumulation coefficient defined in terms of lateral displacement</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>anisotropy factor for cavity expansion</td>
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<td>$\varepsilon_{50}$</td>
<td>strain at 50% of unconfined compressive strength test</td>
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<td>$\varepsilon_a$</td>
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<td>$\varepsilon_c$</td>
<td>strain from unconfined compressive strength test</td>
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<td>$\phi'$</td>
<td>friction angle</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>bulk unit weight</td>
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<tr>
<td>$\theta$</td>
<td>pile rotation at sand surface</td>
</tr>
<tr>
<td>$\theta_N$</td>
<td>pile rotation at peak of N cycle</td>
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<td>reference pile rotation</td>
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<tr>
<td>$\sigma'_v$</td>
<td>vertical effective stress</td>
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<td>$\sigma'_{vL}$</td>
<td>vertical effective stress at pile toe</td>
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<td>$\sigma_{vc}$</td>
<td>consolidation stress</td>
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Chapter 1. Introduction

1.1 Research motivation

The most common method for predicting lateral pile behaviour is the use of load-transfer analyses where the pile is represented as a series of beams with non-linear springs to represent the soil resistance. The soil springs are named $P$-$y$ curves where $P$ is the net lateral pressure and $y$ is the lateral displacement. Besides use of 3D Finite Element method, other approaches for predicting lateral pile behaviour include Poulos (1971) which assumes the soil as an elastic continuum, Davidson (1982) and Lam & Martin (1986) which is an extension of the $P$-$y$ method with additional reaction curves to account for vertical shear stresses around the pile and base shear at the pile toe. However, these are not so commonly used in industry compared to the $P$-$y$ method. The $P$-$y$ method is popular owing to the ease of its low computational effort. Commercial software automatically generates $P$-$y$ curves as recommended in industry standards such as the API (2011) and other well-known methods summarised by Reese & Van Impe (2011).

The $P$-$y$ curves recommended in current design standards and commonly selected in commercial software were derived empirically from field tests carried out many years ago in Matlock (1970) for soft clays; Reese & Welch (1975) for stiff clays; Reese, Cox & Koop, (1974) and Murchison & O’Neill (1984) for sand; and Reese (1997) for weak rock. The piles from these tests had embedment length to diameter ratios ($L/D$) in excess of 10 and exhibited flexible pile behaviour. The methods for clays and sands also include recommendations for predicting lateral cyclic behaviour which involve a reduction in the ultimate net soil resistance ($P_u$).

Recent increases in demand for more accurate predictions of lateral pile behaviour due to increased construction of wind turbines on monopile foundations, nearshore oil and gas infrastructure, and extension of design life for existing offshore structures, have renewed
interest in refining the lateral pile design methods. New studies have revealed inconsistencies in current $P$-$y$ recommendations for static loading such as: (i) under prediction of capacity and stiffness in soft clay (Jeanjean, 2009); (ii) difficulties in selecting stiffness and strength parameters in sands (Suryasentana & Lehane, 2014, 2016); (iii) current recommendations for sedimentary soils are not suitable for predicting lateral pile behaviour in residual soils (Simpson & Brown, 2003; Choi et al., 2013). Predictions for lateral pile behaviour under cyclic laterally loading also need improvement as there are concerns in the wind farm industry that the change in foundation stiffness may change the excitation frequency of the structure and cause resonance with the rotor. Effects of cyclic loading have been found by a large number of recent studies in clay and sand (such as: Rao & Rao, 1993; Long & Vanneste, 1994; Zhang et al., 2011; Leblanc, Houlsby & Byrne, 2010), to be dependent on the cyclic load characteristics and the number of load cycles which are not considered in the standard methods.

Although history has proven the current recommended $P$-$y$ curves to be successful in providing safe pile foundations in practice, the potential to save on costs and materials will present future projects to be more sustainable, in particular for offshore renewable energy sources where pile diameters are exceeding 7m.

### 1.2 Research aim

The overarching goal of this research is to carry out laboratory-based, field based and numerical investigations to extend the existing experimental database, and provide new insights where there are gaps in the literature regarding lateral pile behaviour and $P$-$y$ curves in soft clay, sand and residual soils. The main research aims are discussed in the following points:

1. **Investigate the potential of $P$-$y$ curves derived from in-situ tests for soft clays**

   Recent studies on Cone Penetration Test (CPT) derived $P$-$y$ curves in sands by
Novello (1999), Dyson & Randolph (2001), and Suryasentana & Lehane (2014, 2016), have shown that the end resistance measured in a CPT reflects the soil strength and stiffness behaviour of a laterally loaded pile better than laboratory tests. This method may also be valid for soft clays.

(ii) **Investigate the advantages of different pile shapes in soft clays**

As circular pile diameters are increasing with the demand to withstand larger environmental loads, there may be benefits in utilising square and H section piles which have a greater flexural rigidity ($EI$) per volume of material.

(iii) **Validate existing recommendations for cyclic lateral piles in soft clay**

The cyclic behaviour and post-cyclic lateral capacity of piles in clay have been shown to be dependent on the cyclic load characteristics and the number of cycles by Rao & Rao (1993) and Zhang *et al.* (2011). However, the API (2011) method does not take this into account and it is of interest to ascertain the suitability of API (2011).

(iv) **Investigate the effect of relative density on piles under constant amplitude cycling**

For piles in sand, new methods have been developed which provide better predictions for pile response under static lateral loads but further work is still required to predict cyclic response. Long & Vanneste (1994) collated case history data for field tests on various piles ($L/D > 3$, bored piles and driven piles) and highlighted the importance of sand density, cyclic load characteristics and installation method. A number of centrifuge tests are available in the literature for piles in very dense sands (Verdure, Garnier & Levacher, 2003; Rosquoët *et al.*, 2007; Klinkvort & Hededal, 2013) which also indicate the accumulation of pile head rotations to be dependent on cyclic load characteristics and number of cycles. Other tests performed at 1g in less dense sand are in agreement with these
observations (Leblanc, Houlsby & Byrne, 2010; Albiker et al., 2017). Centrifuge tests of rigid piles in medium dense sand are not yet available and will be required to confirm the effect of relative density on cyclic pile behaviour.

(v) **Investigate the effects of cyclic load sequences on final pile rotation**

Effects due to the order of cyclic load sequences were found to have minimal effect on the final pile rotation by Lin & Liao (1999) using a collation of historical field tests, new field tests by Li et al. (2015), and 1g tests by Peralta & Achmus (2010) as well as LeBlanc, Byrne & Houlsby (2010). These tests applied various sequences of constant amplitude one-way cyclic loads and also found the method of superposition by Lin & Liao (1999) to provide a reasonable prediction of final pile rotation. Further tests are required to determine the effects of two-way loads which also occur in the natural environment.

(vi) **Predicting lateral pile behaviour in residual soils**

There are currently no recommendations for the design of lateral piles in residual soils and others who have carried out field tests, such as Simpson & Brown (2003), Cho et al. (2007) and Choi et al. (2013), have found existing $P-y$ curve methods for sedimentary soils to be unsuitable. A CPT based $P-y$ method may be able to predict the lateral pile behaviour better as it has for lateral piles in sand.

### 1.3 Thesis outline

This thesis has been divided into three chapters for the three soil mediums investigated, namely soft clay, sand, and residual soil. The thesis is prepared as a series of papers, where individual papers are presented as sub-sections in each chapter. Some repetition exists throughout the thesis regarding research motivation and $P-y$ curve derivation methods which were necessary for the preparation of the submitted papers. To avoid more repetition, a general literature review has been omitted in the thesis as the background
literature relevant for each topic is discussed in each paper. An outline of each Chapter and Appendix is provided below.

Chapter 2 is a collection of 3 papers on the study of lateral piles in soft clay. Chapter 2.1 was published in the Proceedings of the 3rd International Symposium on Cone Penetration Testing (CPT14), Chapter 2.2 is currently under review for publication in Géotechnique, and Chapter 2.3 was published in the Proceedings of the 36th International Conference on Ocean, Offshore and Arctic Engineering (OMAE2017). In Chapter 2.1, the potential of a CPT $q_c$ based $P$-$y$ curve is investigated in a series of 3D finite element analyses using a linear elastic perfectly plastic soil model. Chapter 2.2 investigates the effect of pile shape and pile end condition in drum centrifuge tests at the University of Western Australia. 3D finite element analyses were carried out to support the centrifuge findings and to provide insights into the soil mechanisms occurring around different pile shapes. In comparing the tests with existing recommended methods, the importance of applying the correct operational undrained shear strength is revealed. Finally the recommended methods in the API (2011) for cyclically loaded lateral piles in clay are compared with the results from drum centrifuge tests in Chapter 2.3 and the cyclic behaviour of piles in sand is introduced by comparing accumulated pile head rotations in both soil types.

Chapter 3 is a collection of 2 papers on the study of cyclic loading on lateral piles in sand. Chapter 3.1 was published in the 3rd International Symposium on Frontiers in Offshore Geotechnics (ISFOG2015) and Chapter 3.2 has been submitted to Géotechnique. Chapter 3.1 investigates the variables which have the most significant effect on cyclic lateral pile behaviour by comparing pile studies in the literature with preliminary tests conducted in the drum centrifuge at the University of Western Australia. Chapter 3.2 extends this investigation with tests conducted at different densities in the beam centrifuge at the Technical University of Denmark and in addition, investigates the post-cyclic lateral capacity and the effect of cyclic load sequence.
Chapter 4 contains one paper submitted to the Journal of Geotechnical and Geoenvironmental Engineering. The paper in Chapter 4.1 is based on the field tests conducted at a residual soil site in Western Australia. This paper explores the behaviour of a particular unsaturated residual soil through a typical comprehensive series of laboratory and in-situ characterisation tests. The suitability of using the soil test results in conjunction with existing $P$-$y$ curves to predict the pile test results are investigated. The paper concludes with a newly proposed CPT $q_c$ derived $P$-$y$ method which is tested with existing case histories in the literature.

Chapter 5 highlights the major findings from this research and puts forward recommendations for future studies and experimental design in the centrifuge and field tests of laterally loaded piles.

Appendix A, B and C includes extra photos from the UWA drum centrifuge, DTU beam centrifuge, and Bullsbrook field tests respectively. These have been included to provide better visualisation of the work carried out since the data presented in this thesis is largely based on the experimental tests.

Appendix D provides some notes for consideration when preparing future experiments.

Appendix E outlines the Matlab applications developed for interpreting the test data from instrumented piles for static and cyclic loading. The apps SPile.m (for static load tests) and CPile.m (for cyclic load tests) were created with a user interface which allows a selection of different methods for deriving $P$-$y$ curves as suggested by Yang and Liang (2014). The programs allows inspection of the intermediate steps of curve fitting to check for any remnant issues during differentiation and integration of the strain gauge data. The SortCyclicData.m program was created to assign cycle numbers to the continuously recorded data and to allow ease of removing abnormal data points.

Appendix F is a data inventory submitted to UWA for future research.
Chapter 2.  Lateral piles in clay

2.1 Numerically derived CPT based P-y curves

This paper presents the results from numerical analyses which explore the possibility of deriving \( p-y \) springs for laterally loaded piles in clay using direct correlations with the CPT \( q_{\text{net}} \) value. Three dimensional finite element analyses were performed using a linear elastic perfectly plastic soil model to predict the response of single piles in clay subjected to lateral loads. The corresponding CPT \( q_{\text{net}} \) profile for each lateral load test simulation was determined via an expression that was derived using the same soil model in large displacement finite element analyses. A wide range of three dimensional finite element computations of lateral pile response was performed and used to derive CPT based \( p-y \) springs for piles in linear elastic perfectly plastic soil. The new \( p-y \) formulation is shown to be capable of matching the response measured in lateral load tests on centrifuge scale piles in soft clay.

2.1.1 Background

The industry standard method for estimation of the response to lateral load of piles in clay is to use the load transfer method with the soil \( p-y \) non-linear springs recommended by American Petroleum Institute (API, 2011), which were developed by Matlock (1970). The API \( p-y \) curves are a function of the clay’s undrained shear strength \( (s_u) \) and rigidity index \( (I_r) \), and also depend on the relative depth \( (z) \) of any given soil layer. The selection of a representative \( s_u \) profile is difficult given the wide scatter in measured \( s_u \) values observed in practice due, for example, to sampling disturbance effects (especially in soft clays). A direct CPT-based \( p-y \) formulation is therefore clearly preferable to an indirect approach involving \( s_u \).

This paper presents a numerical derivation for \( p-y \) curves applicable to clay assuming a linear elastic perfectly plastic (LEPP) soil model. The \( p-y \) curves are deduced from 3D
finite element analyses (FEA) for 6 different cases and corresponding numerically derived CPT $q_{net}$ (net cone resistance) data are determined using an existing FE solution for cone penetration in a similar LEPP soil. The $p$-$y$ and $q_{net}$ data are compared and used to develop a CPT based $p$-$y$ formulation as a function of $q_{net}$ and $I_r$.

A parallel centrifuge investigation of laterally loaded pile behaviour in soft clay was also performed at the University of Western Australian (UWA) and is presented in a companion paper by Guo & Lehane (2014). The formulations derived here are compared with the lateral pile response of the centrifuge piles to assess their potential for application to laterally loaded pile analysis.

### 2.1.2 Numerical analyses

The numerical analyses for this study were conducted using the 3D FEA program Plaxis 3D (Plaxis3D 2011). Six models were analysed with each having the same pile length to diameter ratio ($L/D$) of 11.8 and with a lateral load applied to the pile at 1.5 pile diameters above the ground surface. The pile comprised volumetric elastic elements (with a Young’s modulus of 30 GPa) and therefore the predicted lateral pile displacements did not include any component due to the pile itself yielding. The pile-soil interface was assumed to be fully rough for the present study; other analyses conducted by the authors show that having an interface roughness halfway between being fully rough and smooth reduces the ultimate lateral stresses developed on piles by about 10%. A linear elastic beam element with negligible stiffness ($E = 10^{11}$ GPa) was embedded at the centre of the volumetric pile to allow for ease of extraction of the pile bending moments and deflections.

The soil was represented as a LEPP isotropic material with a linear shear modulus ($G$) profile, linear $s_u$ profile, Poisson’s ratio ($\nu$) of 0.5 and bulk unit weight ($\gamma$) of 15.6 kN/m$^3$. Clays with strengths corresponding to overconsolidation ratios ($OCR$) of 1 and 2 were examined and the analyses permitted suction values of up to 100 kPa. The full range of
parameters investigated is presented in Table 2.1-1. It is seen that the pile diameters and rigidity indices \( I_r = G/s_u \) considered were varied by about one order of magnitude. A typical deformed mesh and displacement vector plot from the Plaxis 3D analyses is presented in Figure 2.1-1, showing the soil block, pile mesh and soil mechanism at failure.

Table 2.1-1. Input variables for Plaxis 3D analyses

<table>
<thead>
<tr>
<th>Model</th>
<th>Pile diameter, D</th>
<th>Net cone resistance, ( q_{net} )</th>
<th>Soil stiffness ratio, ( E/q_{net} )</th>
<th>Over-consolidation ratio, OCR</th>
<th>Undrained shear strength, ( s_u )</th>
<th>Rigidity index, ( I_r = G/s_u )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m</td>
<td>kPa</td>
<td></td>
<td></td>
<td>kPa</td>
<td></td>
</tr>
<tr>
<td>FE1</td>
<td>0.88</td>
<td>13.7z</td>
<td>17.5</td>
<td>1</td>
<td>1.2z</td>
<td>67</td>
</tr>
<tr>
<td>FE2</td>
<td>0.88</td>
<td>23.9z</td>
<td>17.5</td>
<td>2</td>
<td>2.1z</td>
<td>67</td>
</tr>
<tr>
<td>FE3</td>
<td>0.88</td>
<td>11.0z</td>
<td>5.4</td>
<td>1</td>
<td>1.2z</td>
<td>17</td>
</tr>
<tr>
<td>FE4</td>
<td>0.88</td>
<td>15.8z</td>
<td>38.0</td>
<td>1</td>
<td>1.2z</td>
<td>200</td>
</tr>
<tr>
<td>FE5</td>
<td>0.44</td>
<td>13.7z</td>
<td>17.5</td>
<td>1</td>
<td>1.2z</td>
<td>67</td>
</tr>
<tr>
<td>FE6</td>
<td>3.00</td>
<td>13.7z</td>
<td>17.5</td>
<td>1</td>
<td>1.2z</td>
<td>67</td>
</tr>
</tbody>
</table>

Figure 2.1-1. Plaxis 3D deformed mesh and displacement vector plot of a laterally loaded pile

The following relationship between the net CPT \( q_{net} \) and the clay \( s_u \) was adopted. This was developed by Lu et al. (2004) based on numerical analyses using the re-meshing and
interpolation FE technique in a LEPP soil i.e. the same soil model used for the laterally loaded pile analysis in this paper.

\[ N_{kt} \approx 3.4 + 1.6 \ln I_r - 1.9\Delta + 1.3\alpha_c \quad \text{with} \quad s_u = \frac{q_{net}}{N_{kt}} \quad (2.1-1a) \]

\[ N_{kt} \approx 4.7 + 1.6 \ln I_r \quad \quad \quad \text{with} \quad \Delta = 0 \quad \text{and} \quad \alpha_c = 1 \quad (2.1-1b) \]

The \( \Delta \) value in this equation is referred to as the anisotropy factor, which for the current analysis is taken as zero as the in-situ earth pressure coefficient \((K_o)\) adopted in the pile analyses was unity. A cone roughness factor \((\alpha_c)\) of unity is assumed when applying Equation 2.1-1 as this leads to predicted \( N_{kt} \) values comparable to those indicated in practice (i.e. between 11 and 14 for \( I_r \) between 50 and 300); adoption of this \( \alpha_c \) is consistent with the adoption of a fully rough interface between the pile and soil. The \( q_{net} \) profiles adopted implicitly in the FE analyses are provided in Table 2.1-1. These vary linearly with depth, matching the linear variation of \( s_u \) with depth. The values of the corresponding soil stiffness ratio \((E/q_{net})\) are also given in Table 2.1-1 and the relationship between this ratio and the rigidity index is given in Equation 2.1-2. The \( p-y \) formulation employed by API (2011) uses the strain at a half of the maximum stress recorded in an unconfined compressive strength test \((\varepsilon_c)\) and the relationship between this parameter and \( I_r \) is also given in Equation 2.1-2.

\[ I_r = \frac{1}{3} \left( \frac{E}{q_{net}} \right) N_{kt} = \frac{1}{3} \varepsilon_c \quad (2.1-2) \]

### 2.1.3 Derivation of P-y curves

The lateral pile displacements were taken directly from the FE output at each level of the applied load while the net soil resistance per unit length of pile \( (p) \) was determined by double differentiation of the pile bending moment profiles for each load. The MATLAB (MATLAB 2013) smoothing spline tool was used to assist differentiation and obtain \( p \) at normalized depths \((z/D)\) between 0.5 and 7; the net pressure \((P)\) is equal to \( p/D \). The soil
movement patterns tended to have a larger vertical component at \( z/D \) values greater than 7 (as they are close to the point of fixity) and therefore \( p-y \) data at these depths were not developed.

The maximum ultimate pressures \( (P_u) \) required normalized displacements \( (y/D) \) of up to 2 to develop. This pressure, which is obtained by normalizing the ultimate \( p \) value by the pile diameter, is related in the following equation, to the \( q_{net} \) value via a bearing factor, \( N_{pq} \):

\[
N_{pq} = \frac{P_u}{q_{net}}
\]

(2.1-3)

The calculated \( N_{pq} \) factors for each of the six FE models are presented in Figure 2.1-2, where they are seen to vary with depth at \( z/D \) ratios less than about 3.

\[ \text{Figure 2.1-2. Ultimate soil resistance factor, } N_{pq}, \text{ derived in finite element analyses.} \]

In addition to this depth dependence, computed \( N_{pq} \) values were also found to vary with \( I_r \). The \( N_{pq} \) predictions do not exhibit a dependence on the strength profile or on the pile diameter (e.g. compare predictions of FE1 with FE2, FE5 and FE6), and regression analyses of the computed data \( (z/D \geq 1) \) showed that the following expression provided a
best fit to the set of six FE cases (with a coefficient of determination \(r^2\) of 0.97). Additional FE analyses on a range of \(I_r\) values would help to refine this expression.

\[
N_{pq} = \frac{P_u}{q_{net}} = \left( \frac{3}{4.7 + 1.6 \ln I_r} \right) + [1.5 - 0.14 \ln I_r] \tanh \left[ 0.65 \frac{Z}{D} \right] \tag{2.1-4}
\]

The initial term in Equation 2.1-4, which essentially gives the value of \(N_{pq}\) at the ground surface, is based on the research of (Broms, 1964) and leads to what appears from Figure 2.1-3 to be a conservative ground surface \(N_{pq}\) value (of \(\sim 0.25\)). It is noted that the estimation of \(N_{pq}\) at \(z/D < 1\) was highly sensitive to small numerical errors and therefore this (conservative) assumption was adopted.

The \(N_{pq}\) values predicted using Equation 2.1-4 are compared on Figure 2.1-3 with those given by the API (2011) recommendations for soft clays. It is evident that API (2011) recommends a lateral resistance that is typically about 20% less than that predicted. The deduced \(N_{pq}\) value at \(z/D > 3\) (of about unity) for \(I_r = 200\) is comparable to the fully rough ultimate factor assessed by Randolph & Houlsby (1984) for the flow around mechanism.

Figure 2.1-3. Comparison of \(N_{pq}\) given by Equation 2.1-4 with API (2011) recommendations.
The relationship between $y/D$ with the net pressures normalized by the computed ultimate pressures ($P/P_u$) is presented on Figure 2.1-4 for the three rigidity indices considered. It is evident that contrary to API (2011) recommendations, these curves are not unique for all $z/D$ values. There is a clear tendency for the initial slope of these curves to reduce as $z/D$ increases with curves at $z/D = 3$ (where failure occurs by soil flowing around the pile) being essentially independent of $z/D$; curves at lower $z/D$ ratios reflect patterns associated with wedge and cavity expansion type mechanisms. The rigidity index also has a clear effect on the form of the normalized curves but the similarity of FE1, FE2, FE5 and FE6 (all with $I_r = 67$) indicates the normalized curves are not a function of the pile diameter or the soil strength profile.

Regression analyses of all the FE predictions led to the development of two equations, depending on the relative depth.

$$
\frac{P}{P_u} = \tanh \left[ (0.26I_r + 3.98) \left( \frac{y}{D} \right)^{0.85} \left( \frac{z}{D} \right)^{-0.5} \right] \quad \text{for} \quad 0 < \frac{z}{D} < 3 \quad (2.1-5a)
$$

$$
\frac{P}{P_u} = \tanh \left[ (0.15I_r + 2.3) \left( \frac{y}{D} \right)^{0.85} \right] \quad \text{for} \quad \frac{z}{D} \geq 3 \quad (2.1-5b)
$$

Equations (2.1-5a) and (2.1-5b) provide a fit to the FE data with respective $r^2$ values of 0.98 and 0.99. Further analyses will help to refine these equations. The equations are compared on Figure 2.1-5 with the API (2011) recommendations for soft clay with rigidity indices of 200 and 50 (used for illustrative purposes). It is evident that the API curve (applicable for all $z/D$ values) is stiffer at low $y/D$ values. Overall, however, Equations (2.1-5a) and (2.1-5b) predicts $P/P_u$ values at any given $y/D$ value that are on average within 15% of the API recommendation- despite the fact that these equations were derived for a LEPP soil.
Figure 2.1-4. $P/P_u$ versus $y/D$ curves derived from finite element analyses and Equation 2.1-5
2.1.4 Verification of P-y curves

The accuracy of Equations 2.1-4 and 2.1-5 was checked by using these equations directly in a standard load transfer program and then comparing the predicted lateral load displacement response with that predicted directly in the FE analyses. The program ALP (Oasys ALP 2013) was employed, which (as with many commercially available laterally loaded pile programs) models the pile as a series of beam elements and the soil as a series of non-interacting p-y (non-linear) springs located at nodes between the beam elements. The p-y springs were derived using Equations 2.1-4 and 2.1-5 and input into ALP at a spacing of D/2 along each pile.

The lateral load (H) with pile head displacement predictions obtained for the 6 FE predictions (see Table 2.1-1) are compared on Figure 2.1-6 with the corresponding ALP predictions. Evidently, very good agreement is observed. Such agreement verifies the numerical procedures used for the derivation of Equations 2.1-4 and 2.1-5 and also demonstrates that the load-transfer (beam-spring) approach is capable of reproducing results of the full 3D FEA models, even for diameters as large as 3 m.

Figure 2.1-5. Comparison of API (2011) recommendations with normalized p-y curves given by Equation 2.1-5.
Chapter 2. Lateral piles in clay

*Figure 2.1-6* also shows corresponding predictions obtained using the API (2011) recommendations for soft clay (noting that the rigidity index is related to the API $\varepsilon_c$ parameter via *Equation 2.1-2*). The API (2011) predictions provide a reasonable match to the other sets of predictions up to about half of the ultimate capacities, but then deviate considerably because of the lower API (2011) $N_{pq}$ values (see *Figure 2.1-3*).

Direct application of *Equations 2.1-4* and 2.1-5 is problematic in practice as the rigidity index ($I_r$) is difficult to quantify. Back analyses of existing lateral load test case history data using these equations is the best way of assessing a representative $I_r$ value (which is analogous to the selection of $\varepsilon_c$ when employing API (2011)). One example of such a back analysis is provided on *Figure 2.1-7*, which plots the lateral load-displacement response of two centrifuge scale piles reported by Guo & Lehane (2014). These piles were jacked into kaolin at OCRs of 1 and 2 and load tested after full equalization. The predicted responses of these piles obtained using ALP and *Equations 2.1-4* and 2.1-5 with $I_r = 150$ are seen on *Figure 2.1-7* to provide a good match to the measured responses and are a clear improvement on the predictions obtained using API (2011) with the same $I_r$ value. Future investigations will examine how a suitable $I_r$ value can be derived from shear wave velocities and $q_{net}$ measurements obtained directly in seismic cone penetration tests.
Figure 2.1-6. Comparison of load displacement results between finite element analyses, derived P-y curves, and the API (2011) recommendations.
2.1.5 Conclusions

This paper presents a numerical derivation of $P-y$ curves that are suited to the prediction of the response to lateral load of piles in clays that can be represented by a linear elastic modulus and undrained shear strength, $s_u$. The $P-y$ curves can be derived directly from CPT $q_{net}$ profiles but require an assumption to be made regarding the value of the $I_r$ of the soil. The formulations for the $P-y$ curves when implemented in a standard load transfer (beam-spring) computer model are shown to lead to virtually the same pile response predictions as full 3D finite element analyses. Preliminary back analyses of lateral load test data measured in kaolin at $OCR = 1$ and 2 show that Equations 2.1-4 and 2.1-5 lead to good predictions of the observed response assuming $I_r = 150$. 

Figure 2.1-7. Back analysis of Guo & Lehane (2014) centrifuge results (prototype units) with derived $P-y$ relationships.
2.2  Effects of pile shape and pile end condition

A series of lateral tests conducted in a centrifuge on displacement piles in kaolin is used to examine the effects of pile shape, pile end-condition and clay over-consolidation on the lateral load transfer $P-y$ curves. These experimental tests are supported by finite element analyses which examine the responses of the circular, square and H pile sections used in the centrifuge tests. The experimental and numerical results reveal an important effect of pile shape on $P-y$ curves, but no discernible effect of the pile end condition. These results also indicate comparable dependencies of net ultimate pressures on normalised depth and undrained shear strength. The findings enable development of a new formulation for $P-y$ curves in soft clay, which incorporates the effects of pile shape and uses the triaxial compression consolidated undrained shear strength as the reference undrained strength.

2.2.1 Background

Piled foundations are commonly required to withstand lateral loads from wind, wave, earthquake and collision forces. The sizing of piles, when lateral loads are dominant, is controlled by the allowable lateral movement or rotation at the pile head at the serviceability limit state and by the lateral geotechnical and structural capacity of the pile at the ultimate limit state. The standard method for predicting the lateral performance of single piles involves discretisation of the pile into beam elements, each with corresponding non-linear, non-interactive, springs representing the variation of the net soil resistance with lateral displacement ($y$). These non-linear springs are referred to as $p-y$ or $P-y$ curves, where $p$ is the net soil resistive force per unit length of pile and $P$ is the net soil pressure.

The $P-y$ curves recommended in literature have been derived from instrumented lateral pile tests. The net soil resistance is calculated directly from the pile bending moments
measured in these tests (using the Euler–Bernoulli beam equation) and therefore incorporates the depth dependence of the soil failure mechanisms described by Randolph & Houlsby (1984), and others. Active and passive wedge failures are expected at shallow depths but, with increasing depth, there is a transition to a flow-around mechanism in the absence of the influence of the free surface. A gap forms on the active side near the pile head when a non-zero undrained shear strength exists at the soil surface. If the pile has a high bending capacity, the pile fails by rotating about a point at depth with net soil resistance on the pile below this rotation point acting in the same direction as the applied load to provide lateral and moment equilibrium. A component of base shear may also be significant for large diameter piles where rotation causes large movements at the pile toe.

A pile’s flexural rigidity clearly has a significant effect on its overall performance as quantified, for example, by dimensionless stiffness coefficients proposed by Poulos & Hull (1989). However, Suryasentana & Lehane (2016), Fan & Long (2005), and others, have shown that the $P$-$y$ curves derived from bending moment profiles are independent of the pile rigidity. Suryasentana & Lehane (2016), for example, derive identical $P$-$y$ curves for piles with $EI$ values varying by over 4 orders magnitude in both loose and dense sand. This finding supports the general success of application of $P$-$y$ approaches for laterally loaded piles, which are embedded in recommendations such as API (2011).

**Ultimate lateral pressures**

The American Petroleum Institute recommendations for predicting lateral pile response (API, 2011) are commonly used by industry for both offshore and onshore piles. The recommended method for soft clay is based almost entirely on a research programme described by Matlock (1970) involving a series of tests on an instrumented 0.324 m diameter, 12.8 m long steel circular pile. $P_n$ at discrete depths is calculated as:
Chapter 2. Lateral piles in soft clay

\[ P_u = N_p s_u = \min \left( \left( 3 s_{u, UU} + \sigma'_v + J \frac{s_{u, UU} z}{D} \right), 9 s_{u, UU} \right) \]

(2.2-1)

where \( s_{u, UU} \) is the clay’s undrained strength measured in an unconsolidated undrained (UU) triaxial compression test, \( N_p \) is a dimensionless lateral capacity factor, \( D \) is the pile diameter, \( \sigma'_v \) is the vertical effective stress, and \( J \) is a dimensionless empirical constant assumed by Matlock (1970) to equal 0.5 for offshore soft clays similar to Gulf of Mexico clay. The depth dependence of \( N_p \) embodied in Equation 2.2-1 reflects the evolution from the shallow wedge type mechanism to the deep flow-around mechanism, with \( N_p \) increasing to a maximum value of 9 at depth.

Analytical and numerical studies have shown that \( N_p \) for the deep flow-around mechanism is larger than the value of 9 adopted in API (2011). For example, using separate analytical approaches, Randolph & Houlsby (1984), Murff & Hamilton (1993) and Martin & Randolph (2006) derived a range for \( N_p \) of 9 to 10 for smooth piles and 10 to 12 for rough piles. 3D finite element analyses by Templeton (2009), Jeanjean (2009) and Tzivakos & Kavvadas (2014) reported \( N_p \) values for a rough pile at \( z > 3D \) of between approximately 12 and 15.

Recent instrumented lateral pile experiments in (nominally) normally consolidated clay described by Jeanjean (2009) indicate that, withstanding some evident non-uniformities in the sample, \( N_p \) values increase from about 10 at \( z/D = 1.5 \) to about 16 at \( z/D = 10 \); the operational \( s_u \) value used by Jeanjean (2009) for calculation of \( N_p \) was that corresponding to an average undrained shear strength gradient typical of a normally consolidated clay of 1.2 to 1.25 kPa/m, which equated to a CPT cone factor (\( N_k \)) of 13 in the experiments.

**Pile Shape**

Theoretical support for \( N_p \) factors has been provided for wished-in-place circular piles using the method of characteristics and wedge analyses (e.g. Randolph & Houlsby (1984), Murff & Hamilton (1993) and Martin & Randolph (2006)). Such mechanisms have also
been observed in 3D finite element simulations of laterally loaded piles with a circular cross-section (Tzivakos & Kavvadas, 2014 and Truong & Lehane, 2014). There is not, however, any comparable research performed on the effect of pile shape on these mechanisms and on the resulting $P$-$y$ response. Choi et al. (2014) examined the differences between rectangular and square piles in an elastic soil, but noted no significant effect of shape. In contrast, Suryasentana & Lehane (2016) employed 3D Finite Element non-linear analyses for piles in sand to conclude that square piles were more efficient than circular piles at resisting lateral load. In particular, they found that, for a given pile flexural rigidity and a fully rough interface, the lateral response of a square pile was approximately equal to that of a circular pile with the same circumference.

**Operational undrained shear strength**

The relatively wide range of reported $N_p$ values is partly attributable to the operational undrained strength ($s_{u, op}$) assumed for its derivation and arises because of the well-known effects of sampling disturbance, shearing mode, anisotropy and shearing rate on any specific $s_u$ measurement. API (2011) recommends use of the $s_{u,UU}$ on vertically oriented specimens but the correspondence between this measure of undrained strength and that of $s_{u, op}$ operating within the soil mechanisms in the vicinity of a laterally loaded pile is unclear and likely to vary from site to site. Anderson et al. (2003) used the API (2011) recommendations to predict lateral pile behaviour in seven case histories employing $s_u$ values assessed from a range of different in-situ tests. They found that Equation 2.2-1 gave best predictions when the operational undrained strength was derived from the net cone resistance ($q_{net}$) of the cone penetration test (CPT) using a cone factor of 15. However, the soil adjacent to most of the piles considered did not reach ultimate conditions and the values of $P_u$ in each test were uncertain. In addition to in-situ measurements of $N_p$, the relationship between $s_{u, op}$ and the single isotropic $s_u$ value used
in analytical and finite element (FE) analyses, discussed above, is also not clear and requires investigation.

**Installation effects**

There is little information available to assess the influence of the pile installation method on pile lateral response. No systematic differences due to different installation methods have been identified in soft clays and, based on observations described in Gabr et al. (1994) and elsewhere, this may be presumed to be because the lateral response is controlled by soil extending some distance away from the zone of large installation-induced disturbance. Reese & Van Impe (2011) show how the process of bored pile installation in both sand and clay causes little change to the CPT $q_c$ values in its vicinity while Tomlinson & Woodward (2014) state that there is unlikely to be any long term difference between the lateral performance of a driven and bored pile in cohesive soils. It is clear, however, that further research in this area is required.

Based on the limitations outlined above, this paper describes the results from a centrifuge testing programme designed to address the issues of pile shape, operational strength and installation disturbance in normally consolidated and lightly over-consolidated kaolin. 3D FE analyses are presented to examine the influence of shape for wished-in-place piles in normally consolidated clay. The research presented leads to a new set of recommendations for $P$-$y$ curves for piles in soft clay.

### 2.2.2 Centrifuge tests

**Test equipment**

The centrifuge tests were conducted in the University of Western Australia (UWA), 1.2 m diameter drum centrifuge; a full description of this centrifuge is provided in Stewart et al. (1998). The first set of lateral pile tests was conducted at 80g with a sample over-consolidation ratio ($OCR$) of 1 while the second was performed at 40g with $OCR$ of 2.
Chapter 2. Lateral piles in clay

The schedule of pile tests is summarised in Table 2.2-1 and involved tests on circular, square and H section piles. It is noted that the label B denotes the width of the square and H piles and the diameter of the circular piles. Corresponding end pieces were manufactured for each of the hollow sections to enable investigation of effects of soil displacement induced during closed-ended and open-ended pile installation.

Table 2.2-1. Lateral pile testing schedule

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Shape</th>
<th>Toe</th>
<th>B (a)</th>
<th>L</th>
<th>t</th>
<th>e</th>
<th>g level</th>
<th>El (prototype)</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Circular</td>
<td>Closed</td>
<td>11</td>
<td>135</td>
<td>1</td>
<td>17</td>
<td>80</td>
<td>1.00 × 10^6</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>Circular</td>
<td>Open</td>
<td>11</td>
<td>132</td>
<td>1</td>
<td>17</td>
<td>80</td>
<td>1.00 × 10^6</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>Square</td>
<td>Closed</td>
<td>12</td>
<td>135</td>
<td>1</td>
<td>19</td>
<td>80</td>
<td>2.86 × 10^6</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>Square</td>
<td>Open</td>
<td>12</td>
<td>133</td>
<td>1</td>
<td>19</td>
<td>80</td>
<td>2.86 × 10^6</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>H section</td>
<td>n/a</td>
<td>10</td>
<td>135</td>
<td>1</td>
<td>19</td>
<td>80</td>
<td>1.81 × 10^6</td>
<td>1</td>
</tr>
<tr>
<td>6</td>
<td>Circular</td>
<td>Closed</td>
<td>11</td>
<td>134</td>
<td>1</td>
<td>17</td>
<td>40</td>
<td>63 × 10^3</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>Circular</td>
<td>Open</td>
<td>11</td>
<td>132</td>
<td>1</td>
<td>17</td>
<td>40</td>
<td>63 × 10^3</td>
<td>2</td>
</tr>
<tr>
<td>8</td>
<td>Square</td>
<td>Closed</td>
<td>12</td>
<td>134</td>
<td>1</td>
<td>17</td>
<td>40</td>
<td>179 × 10^3</td>
<td>2</td>
</tr>
<tr>
<td>9</td>
<td>Square</td>
<td>Open</td>
<td>12</td>
<td>132</td>
<td>1</td>
<td>18</td>
<td>40</td>
<td>179 × 10^3</td>
<td>2</td>
</tr>
<tr>
<td>10</td>
<td>H section</td>
<td>n/a</td>
<td>10</td>
<td>131</td>
<td>1, 1.8(b)</td>
<td>20</td>
<td>40</td>
<td>113 × 10^3</td>
<td>2</td>
</tr>
</tbody>
</table>

(a) Dimension includes epoxy coating
(b) Flange & web thickness = 1 mm & 1.8 mm respectively

The piles were instrumented with eight pairs of strain gauges arranged with half bridge configurations. Photos of the test piles showing their cross-sections and the locations of the strain gauges are provided in Figure 2.2-1. The strain gauges were protected and water proofed by a 0.5 mm thick coating of epoxy; this thickness of epoxy is included in the test pile dimensions summarised in Table 2.2-1. End pieces fitted to the base of the hollow section piles were used for closed-ended pile installation and each increased the overall pile length by 2 mm. Each of the instrumented piles was calibrated on a laboratory bench in a cantilever mode by hanging weights from the pile toe while the head was fully clamped. The corresponding voltage readings in the strain gauges were recorded to determine calibration factors to convert voltage output to bending moment.
Lateral loading was applied using a loading arm (see Figure 2.2-1), attached and controlled by the actuator in the drum centrifuge. The loading arm had a rounded end to facilitate application of a point load to the pile head. This loading configuration is shown in Figure 2.2-2.

Figure 2.2-1. Centrifuge test piles and loading arm

Figure 2.2-2. Loading configuration (right) and UWA drum centrifuge (left)
**Clay properties and preparation**

The kaolin clay used is commercially available and its properties are summarised in Lehane *et al.* (2009). The clay has 100% fines content with 70% clay fraction, liquid limit of 61% and plasticity index of 34%. The undrained strength ratio measured in both simple shear and isotropic triaxial compression tests on normally consolidated kaolin, $(s_u/\sigma'_v)_{nc}$, varied between 0.25 and 0.3 while the equivalent ratio in triaxial extension was approximately 0.15. The undrained strength ratios are consistent with the following relationship proposed by Ladd *et al.* (1977):

$$\frac{s_u}{\sigma'_v} = \left(\frac{s_u}{\sigma'_v}\right)_{NC}^{OCR^{0.8}} \quad (2.2-2)$$

The drum sample was prepared by mixing a slurry at 2 times the liquid limit in a vacuum chamber. This slurry was then fed through a tube into the spinning drum channel where it was allowed to consolidate under a centrifuge acceleration of 80g for 4 days. The channel was topped up a number of times over this period and had a height of about 170 mm when fully consolidated. A small thickness of clay was scraped off the top of the sample to ensure a level surface. Once the tests at 80g were completed, the centrifuge acceleration was reduced to 40g and the sample allowed to swell for a period of 16 hours before the first test at $OCR = 2$ commenced.

T-bar tests were conducted during the testing programme at 80g and 40g. The T-bar undrained strengths $(s_{u,Tbar})$ were determined by applying a T-bar bearing factor of 10.5 to the penetration resistance after correction for effects of lateral stress on the load cell output, buoyancy and shallow penetration (Zhao *et al.*, 2016; White *et al.*, 2010). The $s_{u,Tbar}$ profiles are presented using the scaled prototype depth in *Figure 2.2-3*. These show some spatial variation with T-bar undrained strength ratios generally varying from the mean $(s_{u,Tbar}/\sigma'_v = 0.23)$ with a coefficient of variation of 9% and given by the following equation:
\[
\frac{S_{u,Tbar}}{\sigma'_v} = (0.23 \pm 0.02)OCR^{0.8}
\]  

(2.2-3)

The strength variation indicated in this equation was accounted for in the data processing by correlating each pile test result with the closest T-bar test. Figure 2.2-3 also shows that extrapolation of the \( S_{u,Tbar} \) profiles to the surface gives small non-zero values; this is partly due to minor over-consolidation induced by a slight drop in water level below the clay surface during sample preparation. The average rate of increase of \( S_{u,Tbar} \) with depth \((k)\) for a submerged unit weight of 6 kN/m\(^3\) is approximately 1.4 kPa/m and 2.4 kPa/m at \( OCR = 1 \) and 2 respectively.

![Graph showing undrained shear strength measured from T-bar at prototype scale](image)

*Figure 2.2-3. Undrained shear strength measured from T-bar at prototype scale*

**Test procedure**

The actuator in the drum centrifuge was used for pile installation and loading. Each pile was jacked into the sample at 1g at a rate of 2 mm/s using the actuator. After pile
installation, the lateral loading arm was installed onto the actuator and moved in position for loading. The sample was then ramped up to the designated g-level and allowed at least 4 hours of re-consolidation before carrying out the lateral load test. During ramp up, water was slowly added to the channel so that all tests would be conducted under fully saturated conditions with free water on the clay surface. The lateral load was applied at a rate of 0.3 mm/s at the eccentricities above the soil surface (e) shown in Table 2.2-1; this rate of loading equates to approximately 1.6 pile diameters or widths of lateral movement per minute and led to fully undrained conditions.

2.2.3 Centrifuge tests results

Data processing

The bending moments \( M \) at each strain gauge level and at each lateral load level were derived via calibration factors with the strain gauge output. \( P \) and \( y \) values at any particular load level were then calculated from the following equations:

\[
p = P \times D = -\frac{d^2 M}{dz^2} \quad \text{(2.2-4)}
\]

\[
y = \int M/EI \, dz \quad \text{(2.2-5)}
\]

where \( EI \) is the flexural rigidity of the pile. The yield stress in the aluminium piles was not exceeded at the maximum load in any of the lateral pile tests and therefore the lateral pile failure observed was due to a geotechnical rather than pile structural failure. Various curve fitting methods were investigated to achieve a curve fit to the \( M \) values determined at the strain gauge levels; these included the cubic spline, 5\textsuperscript{th} order polynomial and 4\textsuperscript{th} order piecewise polynomial. The cubic spline method was not suitable as the relatively wide spacing of the strain gauges resulted in fluctuations after the second differentiation of \( M \) to calculate \( P \). The (high) 5\textsuperscript{th} order polynomial was also not acceptable as it over-predicted the base shear at the pile toe and resulted in unexpected inflection points at shallow depths. The 4\textsuperscript{th} order piecewise polynomial (using 3 pieces) gave the lowest error.
when the evaluated $P-y$ curves were used in a standard load transfer program to reproduce the measured pile head-load displacement curves.

Determination of $y$ using Equation 2.2-5 requires two known displacements. One of these displacements was measured at the actuator location (i.e. where the lateral load was applied) and the other was assumed to have a zero value where the net soil resistance switched direction i.e. at the inferred point of rotation. This approach was seen to be consistent with the observations from FE analyses discussed later.

**Ultimate net soil resistance ($P_u$)**

Because of some lateral variation in the T-bar strength profiles, the soil pressures developed at the lateral capacity of the tested piles ($P_{max}$) are normalised by $s_{u,Tbar}$ and compared at normalised depths ($z/L$) on Figure 2.2-4a; a duel Y-axis with the approximate $z/B$ value is also provided in this figure, noting that $L/B$ varied between 11 and 13.5 in the experiments. No data are shown at $z/L$ values less than 0.1 (equivalent to $z/B$ less than about 1) because of uncertainties associated with sensitivity to both small changes in $s_{u,Tbar}$ and to the curve fitting methods.

By inference from the progression of pile deflections during lateral loading shown for a typical case on Figure 2.2-4b (from Test 1) and the load transfer curves on Figure 2.2-5 (discussed below), it is clear that mobilised pressures occurring between $z/L$ of 0.7 and 0.9 are below the ultimate values ($P_u$) as they are close to the point of pile rotation where lateral movements are small. The relative depth of the point of rotation for all of the centrifuge piles was closely comparable to that seen on Figure 2.2-4b, which also indicates that the pile response was essentially rigid with $P_{max}/s_{u,Tbar}$ values close to the pile toe being similar to those at $z/L = 0.7$. The existence of friction at the base of the pile influences evaluated $P_{max}/s_{u,Tbar}$ ratios near the toe and these are therefore a less reliable measure of net ultimate lateral resistance.
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Figure 2.2-4. (a) Profiles of maximum normalised net lateral resistance for all tests (z/B axis assumes the mean B value of the test piles) (b) progression of pile deflection profiles in Test 1 (circular pile)

Figure 2.2-4a reveals a substantial effect of pile shape on the ratio of the ultimate pressure to T-bar strength \( \frac{P_u}{s_u Tbar} \) defined as \( N_{p,Tbar} \). The ratios for square and H piles are evidently about 25-40% higher than the ratios for circular piles but show no clear dependence on OCR or pile end condition. It should be noted that although plugging of the open-ended piles (which were installed at 1g) began after a pile penetration of about 3\( B \) (\( z/L < 0.25 \)), the close similarity of \( N_{p,Tbar} \) for open and closed-ended piles up to this penetration indicates that \( N_{p,Tbar} \) is essentially independent of whether the pile was coring or fully plugged at these shallow depths.

\( N_{p,Tbar} \) increases non-linearly with depth up to \( z/L = 0.4 \) (or \( z/B \sim 5 \)) and has approximate average values between \( z/B = 3 \) and \( z/B = 7 \) of 12.4, 15.5 and 17.1 for the circular, square
and H piles respectively. For comparative purposes, Figure 2.2-4a also shows the profile of \( N_p \), as deduced using Equation 2.2-1 following the API (2011) recommendations and equating \( s_{u_T} \) with \( s_{u,UU} \). It is clear that these \( N_p \) values are considerably lower than corresponding \( N_{p_T,UU} \) values and also show a less pronounced dependence on depth to that indicated in the centrifuge tests.

**P-y/B curves**

The influence of pile shape, pile end condition and OCR on the variations of \( P-y \) curves are examined in Figure 2.2-5 and Figure 2.2-6. The effects of pile shape are illustrated in Figure 2.2-5 for open-ended piles at \( OCR = 1 \) at two representative \( z/B \) values. This figure plots the variations of \( P/s_{u_T} \) against normalised lateral displacement \( (y/B) \) for the three different shapes examined. In keeping with the trends indicated on Figure 2.2-4, the ultimate pressures for the square and H-piles are 25-40% larger than those of the circular piles.

![Figure 2.2-5. Normalised load transfer curves for open ended piles at z/B of 3 and 5 in normally consolidated kaolin](image)

Experimental variability and the fact that piles had varying flexural rigidities (see Table 2.2-1) do not permit a general conclusion to be made regarding the stiffness of the
response. It can be inferred, however, that the stiffness increases as $P_u$ increases. For all the pile tests and for $z/B$ between 2 and 6, the mean values of $y/B$ required to develop $0.25P_u$, $0.5P_u$ and $0.75P_u$ were 0.003, 0.013 and 0.05 respectively; the associated coefficient of variation for these values was approximately 0.25.

The influence of OCR and pile end condition is illustrated for a typical case Figure 2.2-6 which plots $P$ values normalised by the respective $P_u$ values against $y/B$ for each pile shape at $z/B = 5$. While there are some differences between the curves for each pile type, it can be concluded that, within the experimental accuracies involved, there is not an observable systematic dependence of the $P/P_u$ vs. $y/B$ variations on the pile end-condition and clay OCR; the same conclusion was inferred for trends shown by $N_{p, Tbar}$. The installation of the piles at 1g and the plugging of the open-ended piles at $z/B > 3$ precludes firm conclusions to be drawn in relation to the end condition. Withstanding these issues, the general similarity of $P/P_u$ vs. $y/B$ variations at shallower depths for closed and open-ended piles supports the contention that the end condition effects are not significant.
Figure 2.2-6. Normalised P-y/B curves at z/B = 5 for (a) circular, (b) square and (c) H piles.
**Overall lateral pile response**

As seen on Table 2.2-1, the embedment of the piles \((L)\) ranged between 131 mm and 135 mm, the pile widths varied from 10 to 12 mm and the eccentricities \((e)\) ranged from 17 mm to 20 mm. These variations therefore need to be allowed for to provide a clear comparison of the lateral performance of the respective pile types. This comparison was achieved by calculating the response of a centrifuge pile with average dimensions of \(B = 11\) mm and \(L = 132\) mm with \(e = 16.5\) mm. The calculations were performed using a standard commercial beam-spring program for laterally loaded piles (Oasys ALP, 2014) and using the \(P-y\) non-linear curves determined at each \(z/B\) value for the respective piles as input to the program.

The calculated load-displacement responses of the piles are summarised on Figure 2.2-7. In keeping with published solutions for laterally loaded piles in soft clay (e.g. Fleming et al. 2009), the lateral load is normalised by \(kB^3\), where \(k\) is the average rate of increase of \(s_{u,Tbar}\) with depth at each pile location, with zero strength at ground surface assumed. Figure 2.2-7 reflects all of the trends observed in Figure 2.2-4 and Figure 2.2-5, namely (i) the lateral capacity of the square and H piles is 25-40% higher than for circular piles and (ii) there is no clear effect of pile end condition on the lateral performance of the piles.

It is evident from Figure 2.2-4 and Figure 2.2-7 that, in terms of pile material volume, H piles have the most efficient shape to resist lateral load. For circular and square piles with the same volume, the width of the square pile is 0.89 times the diameter of the circular pile. It may be inferred from Figure 2.2-4 and Figure 2.2-7 that, even with this reduced width dimension, square piles are more efficient at resisting lateral load than circular piles. This trend is investigated numerically in the following.
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2.2.4 3D Numerical analyses

3D finite element (FE), total stress analyses were performed using Plaxis 3D (Plaxis 3DAE 2015) to examine the effect of shape on lateral pile capacity and to compare calculations with the centrifuge observations at $OCR = 1$. As shown on Figure 2.2-8, half models with the line of symmetry in the direction of loading were created for each pile shape. Each model was 15 m wide, 15 m deep and 40 m long and comprised approximately one quarter of a million 10-noded tetrahedral elements. The meshes are very fine in the vicinity of the piles and coarsen with distance from the piles towards the model boundaries. The vertical boundary surfaces are fixed in the horizontal direction and the basal boundary is fully fixed. Refinements to the meshes and extensions of the model boundaries were carried out until further refinements did not change the solution.
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Figure 2.2-8. Plaxis 3D mesh half models (a) circular pile, (b) square pile, and (c) H pile
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The soil was modelled as an elastic, perfectly plastic material with a Tresca yield criterion and a specified shear strength \((s_{u,FE})\) increasing linearly with depth at a rate of \(k = 1.4\) kPa/m. The ratio of the Young’s modulus to shear strength \((E_u/s_{u,FE})\) was assumed constant and equal to 210; preliminary analyses showed that this ratio led to computed pile head displacements that were consistent with those measured in the experiments.

The piles were modelled as solid sections using linear elastic volume elements with no yielding permitted (recalling that the centrifuge piles did not display any evidence of yielding). A series of beam elements with very low flexural rigidity located along the centreline of the piles enabled easy extraction of pile bending moments and displacements from the FE output. The equivalent prototype dimensions to the mean centrifuge pile dimensions were employed throughout i.e. \(B = 0.88\) m, \(L = 10.56\) m and \(e = 1.32\) m which is equivalent to the mean centrifuge dimensions of \(B = 11\) mm and \(L = 132\) mm with \(e = 16.5\) mm scaled linearly by 80 (for tests conducted at 80g). These dimensions are identical to those used in the load transfer analyses presented in Figure 2.2-7. The FE analyses also investigated the influence of pile-soil interface roughness by varying the roughness parameter \(\alpha_p\), defined as the ratio of the maximum shear stress that can develop at the interface divided by \(s_u\); cases with \(\alpha_p = 1\) and \(\alpha_p = 0.5\) were considered. In addition, given the relatively efficient performance of the square centrifuge piles, a case where the lateral load was applied orthogonal to the diagonal of the square was also examined; for this case, the diagonal dimension was set equal to \(B = 0.88\) m. The full series of FE analyses performed is summarised in Table 2.2-2.

**FE Results**

The computed load displacement results are presented in Figure 2.2-9 in the same format as that employed for the centrifuge tests in Figure 2.2-7. The calculated \(P_u\) and specified \(s_{u,FE}\) profile are used to determine FE \(N_p\) values, termed \(N_{p,FE}\) factors \(= P_u/s_{u,FE}\). These
are plotted on Figure 2.2-10 in the same format as the $N_{p, Tbar}$ values on Figure 2.2-4a. It is evident that the relative effects of pile shape indicated in the FE analyses are in general agreement with trends shown by the centrifuge piles. The capacity of H piles is marginally greater than for square piles, but the capacities for both of these shapes are typically 30% higher than for circular piles.

Table 2.2-2. 3D FE analyses and ultimate load capacities

<table>
<thead>
<tr>
<th>Pile Properties</th>
<th>$a_p$</th>
<th>Hult kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>B (orthogonal to loading)</td>
<td>0.88 m</td>
<td></td>
</tr>
<tr>
<td>L</td>
<td>10.56 m</td>
<td></td>
</tr>
<tr>
<td>e</td>
<td>1.32 m</td>
<td></td>
</tr>
<tr>
<td>EI - Circular Pile</td>
<td>$1.00 \times 106$ kNm$^2$</td>
<td></td>
</tr>
<tr>
<td>EI - Square Pile</td>
<td>$2.86 \times 106$ kNm$^2$</td>
<td></td>
</tr>
<tr>
<td>EI - Rotated square</td>
<td>$2.86 \times 106$ kNm$^2$</td>
<td></td>
</tr>
<tr>
<td>EI - H Pile</td>
<td>$1.81 \times 106$ kNm$^2$</td>
<td></td>
</tr>
</tbody>
</table>

The calculated load displacement responses of the square and H piles are seen on Figure 2.2-9 to be stiffer than for the circular piles, despite each analysis having the same linear Young’s modulus profile ($E_u$). This observation reflects differences in soil deformation paths associated with each of the three pile shapes. The centrifuge experiments also showed a stiffer load displacement response for the square and H-piles but the effect was less pronounced than calculated using the linear elastic-perfectly plastic model.
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Figure 2.2-9. Normalised load-displacement curves from 3D FE analysis

Figure 2.2-10. FE calculations for the lateral pile capacity factor (and a comparison with API 2011)
Maximum $N_{p,FE}$ values, with fully rough conditions ($\alpha_p = 1$) at $z/B > 3$ are 12.5, 17.3 and 18.7 for the circular, square and H pile respectively. For the semi rough case with $\alpha_p = 0.5$, these $N_{p,FE}$ values reduce to corresponding values of 11.5, 17 and 18.7. Evidently, $N_{p,FE}$ for circular piles is most sensitive to the interface roughness and is least sensitive for H piles. The maximum $N_{p,FE}$ values for circular piles of 12.5 and 11.5 at $\alpha_p = 1$ and 0.5 are 5% larger than the respective values of 11.9 and 10.8 derived by Martin & Randolph (2006) using classical plasticity. Unlike the circular pile, the $N_{p,FE}$ values for the square and H piles tend to continue to show modest increases in magnitude up to $z/B = 6$; such a tendency was predicted in the solutions of Murff & Hamilton (1993) and was also indicated for all pile shapes by the centrifuge $N_{p,Tbar}$ values on Figure 2.2-4a.

Additional insights into the effect of shape can be drawn from the analysis of a square pile which is loaded normal to the diagonal and where the diagonal width is the same as that of the square pile with loading normal to one side. The $EI$ value for both piles was held constant (see Table 2.2-2). It is seen on Figure 2.2-9 that the response of the rotated pile (i.e. loaded normal to its diagonal) is almost identical to that of the fully rough circular pile. This response suggests that the soil failure modes for both these cases are very similar.

The difference between the flow around failure modes for each pile shape is illustrated on Figure 2.2-11, which presents computed displacement vectors on the horizontal plane at $z/B = 5$ at ultimate conditions. It can be seen that the circular section and the square section loaded on the diagonal encourage a rotational soil movement at short distances from the centreline. In contrast there is greater tendency for lateral translation across the full widths of the square and H piles and consequently a much larger radius for the rotational mechanism developed at ultimate conditions. Generation of this wider mechanism requires more energy and gives rise to the interpretation of higher net ultimate pressures and $N_p$ values. The slightly less constrained movement of clay between the
flanges of the H pile explains why its $N_p$ value is marginally greater than that of a square pile.

Figure 2.2-11. Displacement vectors at $z/B=5.7$
2.2.5 Operational undrained shear strength

It is of interest to compare the ultimate net pressures \((P_u)\) recorded in the centrifuge tests with those calculated in the FE analyses and with API (2011), which recommends a maximum \(N_p\) value \((P_u/s_{u,UU})\) of 9 for flow-around conditions (see Equation 2.2-1).

The profile of undrained shear strength specified in the FE analyses was \(s_{u,FE} = 1.4\) kPa/m, which is the same as the mean T-bar strength profile for \(OCR = 1\) clay. Computed values of \(P_u\) with \(\alpha_p\) between 0.5 and 1 for a circular pile are within 10% of those inferred from the centrifuge tests, although it is noted that these tests display a gradual increase in \(N_p\) with depth that is not observed in the FE analyses. This comparison suggests that the operational strength \((s_{u,op})\) in the vicinity of the circular pile (i.e. the strength that should be input to the FE analysis to predict the centrifuge pile response) is comparable to that of a T-bar.

Undrained triaxial test data for kaolin (e.g. Lehane et al. 2009) indicate that the undrained strength ratio of both isotropically and anisotropically normally consolidated samples of kaolin ranges from 0.25 to 0.3, with an average value of 0.275. This average ratio gives a strength gradient for consolidated undrained strength \((s_{u,CU})\) of 1.65 kPa/m which is 1.17 times greater than \(s_{u,Tbar}\). The ultimate pressure ratios for the centrifuge tests calculated using \(s_{u,CU}\) rather than \(s_{u,Tbar}\) (i.e. \(N_{p,CU} = P_u/s_{u,CU}\)) are presented in Figure 2.2-12, where they are seen, for circular piles, to give a mean stabilised \(N_p\) value at depth (i.e. corresponding to full flow-around) of 10.5 which is more in line with the design value of 9 recommended in API (2011) and used commonly in practice. However, as indicated in Equation 2.2-1, API (2011) employs the unconsolidated undrained triaxial strength \(s_{u,UU}\) as the reference strength. If the \(s_{u,UU}/s_{u,CU}\) ratio of 0.7 recommended by Chen & Kulhawy (1993) is adopted, the corresponding \(N_p\) value at depth shown in the centrifuge tests is 15, that is 66% higher than the recommended value.
The foregoing suggests that the operational strength in the vicinity of a lateral loaded pile in soft clay is comparable to the undrained strength measured in a T-bar. However, a slightly lower $N_p$ value should be used when relating net ultimate pressures to undrained strengths measured in triaxial compression ($s_{u,\text{CU}}$) on samples consolidated to their in-situ stress state.

### 2.2.6 Proposed P-y formulation

A $P$-$y$ formulation for soft clays is presented here using the value of $s_{u,\text{CU}}$ as the reference undrained strength. This strength is used because of the popularity of the triaxial compression test and the declining practice of relying on UU strengths due to well-known losses in effective stress that occur during tube sampling. A circular pile is used as the base case for the formulation for which the flow-around value of $N_{p,\text{CU}}$ at depth in the centrifuge tests averaged at about 10.5.

*Equation 2.2-6* was found by regression analysis of the centrifuge $P_u$ data bearing in mind the observation that the ratio of the ultimate net lateral soil pressure ($P_u$) to undrained shear strength in soft clays is independent of the OCR (for $OCR \leq 2$) and the installation mode (at least for $z/B < 3$), but varies with depth, pile shape and pile roughness. $P_u$ is taken to vary with normalised depth ($z/B$), rather than soil depth ($z$), which is in accordance with results from finite element analyses involving lateral tests on piles with different widths (e.g. as reported by Truong & Lehane (2014), and others).

$$\frac{P_u}{s_{u,\text{CU}}} = 10.5 \left[ 1 - 0.75 e^{-0.6z/B} \right] S_p$$

*Equation 2.2-6* is compared on *Figure 2.2-12* with the results from the centrifuge experiments for the 3 pile shapes and two OCRs investigated (noting that cracks do not occur on the active side at these OCRs). It is evident that *Equation 2.2-6* provides a reasonable fit to all centrifuge data and FE calculations. As the centrifuge $P_u$ data at very
shallow depths are prone to experimental error, the FE results were used to inform the regression analysis for \( z/B < 2 \) and gave \( N_p \) factors of 2.75 for the circular pile and 3.85 for a H-pile at the clay surface \( (z = 0) \). Slightly higher ultimate pressures than those given by Equation 2.2-6 may be expected for piles with a roughness greater than the aluminium centrifuge piles.

The full set of \( P/P_u - y/B \) curves obtained between \( z/B \) of 2 and 6 in the centrifuge tests are plotted on Figure 2.2-13. It has previously been seen that these normalised curves do not depend in any obvious way on the normalised depth \( (z/B) \), pile shape, pile end condition...
and clay. Although the spread of curves appears significant on Figure 2.2-13, the following best fit function to all curves has a correlation coefficient ($r^2$) of 0.96:

$$\frac{P}{P_u} = \tanh \left[ 5.45 \left( \frac{y}{B} \right)^{0.52} \right]$$

(2.2-7)

Figure 2.2-13 also presents the $P/P_u$-$y/B$ curve recommended by API (2011) for clay using $\varepsilon_{50}$ values of 0.005 and 0.02, where $\varepsilon_{50}$ is the strain to reach 50% of the mobilised strength in an undrained triaxial compression test. It is interesting to note that $\varepsilon_{50}$ values measured in triaxial tests on the UWA kaolin ranged from 0.001 to 0.004 with an average value of 0.002. Matlock (1970) suggests $\varepsilon_{50}$ of 0.02 for soft clays but the value of 0.005, which was recommended for stiff clays, provides the closest match to the experimental results.

![Figure 2.2-13. Normalised P-y curves](image-url)
Equation 2.2-6 and 2.2-7 are combined to give the following P-y relationship:

\[
P = 10.5 s_{u,\text{CU}} \tanh \left[ 5.45 \left( \frac{y}{B} \right)^{0.52} \right] \left[ 1 - 0.75 e^{-0.6z/B} \right] S_p \quad (2.2-8)
\]

The P-y/B responses calculated at z/B = 5 for an example case with B = 1 m are compared on Figure 2.2-14 with the P-y/B curve recommended by API (2011). It is seen that the API (2011) curve with \( \varepsilon_0 = 0.005 \) and \( N_p = 9 \) provides a reasonably conservative estimate of the curve for the circular pile, when the \( s_{u,\text{CU}} \) strength is used in place of \( s_{u,\text{UU}} \). Using the \( s_{u,\text{UU}} / s_{u,\text{CU}} \) ratio of 0.7, referred to above, the API (2011) recommendations obtained using \( s_{u,\text{UU}} \) are seen on Figure 2.2-14 to lead to a very conservative P-y/B response. The figure also highlights the significant benefits of using a square sectioned pile.

![Figure 2.2-14. P-y curve comparison example at depth of z/B=5](image-url)
2.2.7 Conclusion

(i) The results from a series of centrifuge lateral pile tests in kaolin supported by finite element analyses clearly illustrate the importance of pile shape on the lateral load transfer \((P-y)\) curves. Higher net pressures can develop for square and H-piles compared to circular piles and this arises because of the larger volume of clay per unit pile width involved in the failure mechanisms surrounding the square and H-pile sections.

(ii) Withstanding potential differences associated with pile installation at \(1g\), the centrifuge tests also showed that the end-condition for a displacement pile has no discernible effect on the subsequent \(P-y\) response at \(z/B < 3\); assessments of effects at deeper levels could not be made because open-ended piles plugged at these levels.

(iii) The centrifuge tests indicated approximately the same net ultimate pressures \((P_u)\) as FE analyses when the T-bar strengths are input to the analyses i.e. the operational strength is equal to \(s_{u,Tbar}\). This operational strength is, however, lower than usually inferred from consolidated triaxial compression tests \((s_{u,CU})\) and hence a lower ratio of \(P_u\) to undrained strength needs to be adopted when \(s_{u,CU}\) is used as the reference strength.

(iv) The centrifuge experiments did not reveal any systematic dependence of \(P/P_u\) vs \(y/B\) curves on \(z/B\), pile shape, pile end condition and clay \(OCR\), over the range of parameters investigated. This result combined with the observed dependence of net ultimate pressures on normalised depth, pile shape, undrained shear strength were used to develop a new \(P-y\) equation for piles in soft clay \((Equation 2.2-8)\). This equation is shown to be reasonably consistent with API (2011) recommendations for circular piles if \(\varepsilon_{50}\) is taken as 0.005 and \(s_{u,CU}\) is used in
place of $s_{uu}$. The shape factor included in Equation 2.2-8 reflects the importance of allowing for the effects of pile shape when evaluating $P-y$ curves.

### 2.2.8 Acknowledgements

The authors are grateful for the funding provided by the Australian Research Council. The authors would also like to acknowledge the valuable contributions of Dr. Fengju Guo to the centrifuge testing.
2.3 Effects of lateral cycling

The accumulation of lateral displacement of monopiles under lateral cycling over the life of a wind turbine is a major design consideration. This paper presents the results from centrifuge tests on instrumented piles installed in kaolin prepared to an over-consolidation ratio of 2; the tests involved both monotonic loading and two separate one-way cyclic loading histories. It is shown that the geotechnical ultimate limit state is relatively unaffected by cycling but that large permanent rotations occur during cycling, even at a relatively modest cyclic load ratios.

2.3.1 Background

The foundations for wind turbines usually comprise a single pile (or monopile) to resist applied lateral loads, moments and vertical loads. The strong recent growth of the offshore wind industry has provided added impetus to improve understanding of the response of these monopiles when subjected to lateral cyclic loading. This paper presents the results from a pilot experimental study that re-examines the recommendations of the American Petroleum Institute for estimation of the lateral response under cyclic loading of piles in soft clay (API 2011). These recommendations are essentially the same as those proposed almost fifty years ago by Matlock (1970). The paper also examines the accumulation of lateral pile displacement under the action of one-way cyclic loading; such accumulation is not covered in API (2011) but is of significant importance for offshore wind turbines as the safe operation of the turbine in service requires the total pile rotation over the lifetime of the turbine to be less than about 0.5 degrees (Golightly, 2014).

API (2011) recommendations for cyclic load transfer p-y curves in soft clay (i.e. load-displacement response at a given depth after cycling) assume that the ultimate pressures that can be generated are 72% of those that can develop under monotonic loading and that post-peak softening occurs at shallower depths. Typical API (2011) lateral pressure ($P$)
vs. lateral displacement curves for cyclic conditions are shown on *Figure 2.3-1* for the case replicating the experiments discussed in the next section of this paper. At large normalized depths \( (z/D) \), the ultimate pressures attain maximum values of \( 0.72 \times 9_{su} = 6.5_{su} \) and no softening is anticipated.

![Figure 2.3-1. P-y/D cyclic curves recommended by API (2011) for centrifuge experiments reported in this paper (assuming UU strengths equivalent to T-bar strength)](image)

More recent investigations of effects of lateral pile cycling in soft clays have been reported by Rao & Rao (1993), Jeanjean (2009) and Zhang *et al.* (2011). These have shown that, contrary to the reduction in lateral resistance to 72% of the monotonic capacity recommended by API (2011), the post-cycling capacity is actually higher than the monotonic capacity especially for piles subjected to lower level cycling. One-way cycling was seen to lead to an accumulation of pile displacements/rotations but the actual post-cyclic response was stiffer. The studies by Rao & Rao (1993) and Zhang *et al.* (2011) did not enable measurements of \( p-y \) data while that of Jeanjean (2009) did report \( p-y \) curves under cyclic loading but these tests were conducted under displacement controlled cycling rather than under load control conditions existing in practice.
The experiments described in this paper extend the existing database of information by reporting on a series of centrifuge experiments using an instrumented displacement pile to examine the lateral pile and $p$-$y$ response under load controlled cycling in lightly over-consolidated kaolin. The experimental results enable simple conclusions to be drawn related to the effects of lateral pile cycling in soft clay.

### 2.3.2 Centrifuge tests

The centrifuge tests were conducted in the 1.2 m diameter drum centrifuge at the University of Western Australia (UWA). The tests were performed at a centrifuge acceleration of 40g in kaolin that had been over-consolidated to an OCR value of 2 by allowing the kaolin to first consolidate from a slurry at 80g before ramping back to 40g.

The aluminium pile used in the tests was a hollow square section with a side width ($B$) of 12 mm, wall thickness of 1 mm and was fitted with an end cap to model a closed ended pile. The pile was equipped with eight pairs of strain gauges arranged with half bridge configurations at various levels to allow measurement of bending strains and hence bending moments. The equivalent prototype diameter ($D_{eq}$) of the test pile (assuming equivalence of areas) is 0.54 m.

The instrumented pile was pushed in by the actuator to an embedment of 5.36 m (prototype scale) at a number of locations in the drum sample and then subjected to lateral loading in the circumferential direction; this embedment equates to a normalized embedment ($L/D_{eq}$) of about 10. The pile tip was located at a distance of 2.9 m (prototype) away from the bottom of the drum channel. The centrifuge was stopped to allow the pile installation at 1g and the centrifuge acceleration was then increased back up to 40g before testing commenced after a 4 hour period of re-consolidation. Lateral loading was applied using an actuator with a loading arm that had a rounded end to facilitate application of a point load to the pile head.
Chapter 2. Lateral piles in clay

The tests reported in this paper are summarized in Table 2.3-1. The lateral loads were applied at an eccentricity above the surface of the kaolin (e) of about one equivalent diameter. The first test conducted involved monotonic loading of the pile to ultimate geotechnical failure (i.e. no yielding of the pile occurred). The cyclic tests (Nos. 2 and 3) comprised application of N number one-way cycles of lateral load at a fixed cyclic load ratio (ζb) followed by an immediate monotonic push to failure. The ζb value is defined as the ratio of the maximum lateral load applied in the cycles (Hmax) to the lateral load required to cause a pile head rotation of 1° in the monotonic test (Hult). It is noted that Hmax values for laterally loaded piles are usually defined at a given pile head rotation (e.g. LeBlanc, Houlsby & Byrne 2010) as the actual lateral capacity (Hult) often requires very significant lateral movement or is influenced by the structural capacity of the pile section.

The lateral loading rate employed during both monotonic and cyclic loading on the model pile was 0.3 mm/s; this rate is equivalent to 2.5% of the pile width per second, which was shown in Truong & Lehane (2016) for the same drum centrifuge models to lead to fully undrained conditions.

Table 2.3-1. Centrifuge test details (prototype scale)

<table>
<thead>
<tr>
<th>Test no.</th>
<th>B (m)</th>
<th>D_eq (m)</th>
<th>L (m)</th>
<th>e (m)</th>
<th>Hult (kN)</th>
<th>Hmax (kN)</th>
<th>Hult/H ± (°)</th>
<th>ζb =</th>
<th>N cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.48</td>
<td>0.54</td>
<td>5.4</td>
<td>0.68</td>
<td>54.6</td>
<td>-</td>
<td>48</td>
<td>-</td>
<td>Mono</td>
</tr>
<tr>
<td>2</td>
<td>0.48</td>
<td>0.54</td>
<td>5.4</td>
<td>0.56</td>
<td>60.0</td>
<td>26.4</td>
<td>-</td>
<td>0.55</td>
<td>139</td>
</tr>
<tr>
<td>3</td>
<td>0.48</td>
<td>0.54</td>
<td>5.4</td>
<td>0.70</td>
<td>53.7</td>
<td>42.4</td>
<td>-</td>
<td>0.88</td>
<td>24</td>
</tr>
</tbody>
</table>

The properties of kaolin clay employed are summarized in Lehane et al. (2009). The clay has a fines content of 100% with a 70% clay fraction, a liquid limit of 61% and plasticity index of 34%. The T-bar sensitivity (St) of this kaolin is approximately 3.5.

T-bar tests conducted in flight were used to assess the in-situ shear strength of the drum centrifuge sample. Undrained shear strengths (Su_Tbar) were determined using a T-bar factor of 10.5 with corrections for shallow effects, buoyancy and others as described in Zhao et al. (2016) and White et al. (2010). All Su_Tbar profiles obtained are summarized in
Figure 2.3-2, although showing some deviations from a linear increase with depth, \( s_{u,T_{bar}} \) can be reasonably characterized by the best fit line, \( s_{u,T_{bar}} = 2.3z \) (kPa). Figure 2.3-2 also indicates the presence of small non-zero shear strengths near the surface; this is thought to be partly due to minor over-consolidation induced by a slight drop in water level below the clay surface during the one week sample consolidation period.

![Figure 2.3-2. T-bar undrained strengths measured in drum centrifuge sample](image)

### 2.3.3 Experimental observations

The load-displacement \((H-y)\) responses measured at the pile head (at the original soil surface) in each cyclic test are compared with the monotonic response in Figure 2.3-3. The accumulation of displacement under one-way cycling is clear. It is also apparent that the rate of displacement accumulation is higher at the higher cyclic load ratio \((\zeta_b)\) and that the post-cyclic monotonic push to failure after lower level cycling \((\zeta_b = 0.55)\) gave a greater capacity.
Chapter 2. Lateral piles in clay

Figure 2.3-3. Load-displacement at pile head for (a) Test 2 and (b) Test 3

The load-normalized displacement ($H - y/B$) and applied moment-pile head rotation ($M_{app} - \theta$) responses observed in the monotonic test (Test 1) are compared on Figure 2.3-4 with those observed during the post cycling phases of Test Nos. 2 and 3 (see Table 2.3-1). It is evident that the monotonic and post–cyclic capacities are very similar when considering the applied horizontal load and bending moment together. A stiffer and slightly stronger response is in evidence for the Test 2 with lower level cycling while Test 3 exhibits a slightly lower stiffness and capacity. These trends are in keeping with those identified previously by Rao & Rao (1993), and others.
Chapter 2. Lateral piles in soft clay

Figure 2.3-4. (a) Lateral load vs. normalised displacement at ground surface and (b) applied moment vs. pile head rotation at ground surface

Figure 2.3-4b shows that the applied moment at a pile head rotation of 1° is 32.6 kNm, implying that $H_{1^\circ} = 32.6/0.68 = 48$ kN and this load occurs at $y/D_{eq} = 0.13$. The actual ultimate capacity ($H_{ult}$) of 54.6 kN is 13% higher but its development requires a pile rotation of about 3° and $y/D_{eq}$ ratio at the soil surface of 0.5.

One of the significant features of this experimental series is that the pile bending moment profile could be determined for any loading stage. Double differentiation of these bending moments at the ultimate conditions in the monotonic and post-cyclic pushes enabled determination of the net pressure profiles acting on the piles. These profiles are plotted
on Figure 2.3-5 where they are compared with the ultimate net pressures derived using API (2011). The drop in pressures between z/D of about 6.5 and 9 indicated on this figure arises because the pile movements close to the depth of rotation are insufficient to generate ultimate pressures; this depth of rotation was about 80% of the embedded length in all tests. It is seen on Figure 2.3-5 that the ultimate net stresses measured post-cycling are very similar and the same (if not very slightly larger) than the ultimate net pressures measured under monotonic loading. These trends are consistent with those inferred from measured pile head responses shown on Figure 2.3-4.

![Profiles of ultimate net pressures with depth](image)

**Figure 2.3-5. Profiles of ultimate net pressures with depth**

Although Figure 2.3-4 and 2.3-5 show little apparent effect of cycling, the action of one way cycling led to a progressive accumulation of pile displacement (as observed in Figure 2.3-3) and rotation with each cycle. The increase in rotation with each cycle reduced as cycling progressed in Test 2 (with $\zeta_b = 0.55$) but remained almost constant at $0.4^\circ$ per cycle in Test 3 (with $\zeta_b = 0.88$). The pile head rotation after N cycles ($\theta_N$) normalized by
the rotation after the first cycle ($\theta_1$) is plotted using double logarithmic scales on *Figure 2.3-6* for Tests 2 and 3. Test 3 with $\zeta_b = 0.88$ indicates a very significant level of movement and a pile head rotation after 24 cycles that is eight times the initial rotation after 1 cycle i.e. $8\theta_1$. Test 2, which has a cyclic load ratio that is more typical of storm event for a real pile ($\zeta_b = 0.55$), also shows a relatively high $\theta$ value (more than $3\theta_1$ at $N = 139$) but the rate of accumulation of rotation after $N = 100$ is very small (~0.0008°/cycle). The trends shown for the piles in kaolin on *Figure 2.3-6* are consistent with the dependence of accumulated shear strains on cyclic load ratio observed in 1-way cyclic direct simple shear (DSS) tests on normally consolidated kaolin (Zografou *et al.*, 2016). By inference from these DSS tests, continued high level cycling in Test 3 would lead to further increases in cyclically induced excess pore pressures and cyclic failure whereas the number of cycles to induce cyclic failure in Test 2 is likely to be extremely large. Nevertheless, the rotation experienced after 139 cycles in Test 2 is approximately 0.8° and hence the serviceability limit state of 0.5° typically imposed for a monopile is exceeded. The data from Test 2 show that 10 lateral load cycles with $\zeta_b = 0.55$ is sufficient to induce a SLS failure.

The relatively high levels of accumulated rotation developed in Tests 2 and 3 are compared on *Figure 2.3-6* with 1g test data reported by LeBlanc, Houlsby & Byrne (2010) for cyclic lateral pile tests on Leighton Buzzard sand. It is evident that, at comparable cyclic load ratios ($\zeta_b$), the pile rotations in loose sand (relative density, $D_r = 38\%$) are significantly lower than for the soft clay and are almost an order of magnitude smaller at the high cyclic load ratio ($\zeta_b = 0.81$ to 0.88).
The variation with cycling of the ratios of the net lateral pressure determined at the N\textsuperscript{th} cycle ($P_N$) to that after the first cycle ($P_1$) are plotted on Figure 2.3-7. These show that pressures at shallow depths ($z/D \leq 3$) reduce as cycling progresses. The maximum reduction in pressure observed is 30\% in Test 3 at $z/D = 2$ and this reduction is significantly less than the 75\% reduction at $z/D = 2$ adopted in API (2011), as indicated in Figure 2.3-1. Full remoulding of the clay due to cycling could be expected to reduce the lateral pressure by a factor of $1/S_t = 1/3.5 \approx 0.3$, but clearly this did not take place.

Increases in net pressures during (load-controlled) cycling of up to 20\% occur at depth (e.g. at $z/D = 6.2$ in Test 3) and these increases can be explained by the additional pile movement developed at these depths in response to the lower pressures and stiffness at shallower depths i.e. load shedding.
The bending moments induced in the piles after the first cycle and the 100th cycle in Test 2 and 20th cycle in Test 3 are plotted on Figure 2.3-8. It is apparent that the bending moment profile at \( N = 1 \) and \( N = 100 \) in Test 2 are almost identical, reflecting the comparable values of \( P_N/P_1 \) seen on Figure 2.3-7a (and noting that the actual pressures at very low \( z/D \) values are of low magnitude and hence have a relatively small influence on the bending moment profile). A slight difference between the \( M \) profiles at \( N = 1 \) and \( N = 20 \) is evident for Test 3, which has a maximum moment about 10% larger at \( N = 20 \).
occurring at a slightly deeper location; this arises because of the 10-20% drop in $P_n/P_1$ at low $z/D$ values (seen in Figure 2.3-7b).

![Diagram of maximum bending moment development](image)

**Figure 2.3-8. Development of maximum bending moment**
2.3.4 Conclusions

The centrifuge tests reported in this paper allow the following conclusions to be made:

(i) Lateral pile capacities developed after one-way cycling are closely comparable to monotonic capacities and relatively un-affected by high level cycling. The API (2011) recommendations for ultimate lateral cyclic capacity are conservative.

(ii) One-way lateral cycling of piles in soft clay causes an accumulation of pile rotation, the rate of which increases with the cyclic load ratio. Permanent rotations induced by cycling are considerable higher than those induced by cycling in loose sand at the same cyclic load ratio.

(iii) The centrifuge tests showed that SLS failures for monopiles in soft clay are likely to control design as only 10 one-way cycles with a cyclic load ratio of 0.55 were sufficient to induce a SLS failure.

(iv) The degradation of lateral pressures near the soil surface under high level cycling was observed to be less than 30% in all cases and less than recommended by API (2011).

(v) Pile bending moments were relatively insensitive to the effects of cycling.

2.3.5 Acknowledgments

The authors are grateful for the funding provided by the Australian Research Council. The authors would also like to acknowledge the valuable contributions of Dr. Fengju Guo to the centrifuge testing.
Chapter 3. Lateral piles in sand

3.1 Effects of lateral cycling

Current design standards for piles in sand are based on research conducted over 30 years ago with limited testing data and consideration of cyclic effects. With a global increase in the use of monopiles as foundations for offshore and near-shore structures to sustain long term cyclic environmental loads, there is now a need to improve understanding of the behaviour of piles under lateral cyclic loading. This paper addresses the shortage of test data by presenting results from a series of lateral cyclic loading tests conducted in medium dense sand in the centrifuge. These results are compared with data from recent publications to improve insights into the effects of cyclic load characteristics and soil density on the pile head displacement, pile bending moments, soil-pile stiffness and soil reaction ($p$-$y$ curves).

3.1.1 Background

Due to a recent global increase in demand for offshore and near-shore infrastructure, there is also a demand to improve the current design methodology for monopile foundations. These foundations are subjected to long term cyclic loading from wind and waves. Today, engineers are still facing challenges in predicting the behaviour of piles under lateral cyclic loads which is essential for serviceability limit state design.

Current practice for predicting lateral pile behaviour in sand uses the recommendations of the American Petroleum Institute (API, 2011). These recommendations are based on studies by Murchison & O’Neill (1984) from three decades ago and API (2011) suggests that their formulations should only be used “in the absence of more definitive information”. However, there is not, as yet, a well-defined method or pile testing regime to allow estimation of pile behaviour under cyclic loads, and therefore, designers usually default to the formulations in the API (2011). The API (2011) method uses non-linear
lateral soil resistance-displacement \((p-y)\) curves to represent the soil stiffness over the depth of the pile. These \(p-y\) curves are dependent on the soil depth, pile diameter, soil friction angle, soil unit weight, and pile embedment length. To predict cyclic effects, the API (2011) recommends simply reducing \(p\) by a factor of 0.9 to represent a 10% reduction in soil resistance due to cyclic loading. However, more recent studies have found that the cyclic behaviour has also a relatively strong dependence on other parameters such as number of cycles \((N)\), cyclic load characteristics, load eccentricity \((e)\), soil relative density \((D_r)\) and pile installation. Hence, applying a single reduction factor to the static \(p-y\) curve cannot be deemed sufficient to confidently assess the impact of cyclic loading on lateral pile response.

It is common to describe the cyclic load characteristics by a cyclic load ratio \((\zeta_c)\) and a load magnitude ratio \((\zeta_b)\) as presented in Equation 3.1-1 and 3.1-2:

\[
\zeta_c = \frac{H_{\text{min}}}{H_{\text{max}}} \tag{3.1-1}
\]

\[
\zeta_b = \frac{H_{\text{max}}}{H_{\text{us}}} \tag{3.1-2}
\]

where \(H_{\text{min}}\) is the minimum and \(H_{\text{max}}\) is the maximum applied load in the cycle, and \(H_{\text{us}}\) is the ultimate static capacity of the pile. \(\zeta_c\) can range between -1, which represents two-way cyclic loading, and 1, which represents static loading; one-way cyclic loading corresponds to a \(\zeta_c\) value of 0. The value of \(\zeta_b\) is a measure of the severity of the applied loading but its value depends on how \(H_{\text{us}}\) is defined. Some authors take \(H_{\text{us}}\) as the horizontal load at a specified displacement (often as a proportion of the pile diameter) and some define \(H_{\text{us}}\) as the intersection between the initial tangent stiffness and the tangent to the asymptote of the lateral load-displacement curve. There is currently no agreed method for selecting \(H_{\text{us}}\) in sand.

This paper initially reviews published experimental studies and summarises some of the trends observed. A description of a new centrifuge study is then presented and
subsequently the trends observed in this study are compared with the previous studies to improve insights into the effects of cyclic load characteristics and soil density on lateral pile response.

Long & Vanneste (1994) collated results from 34 full scale cyclic lateral pile tests in sand to enable assessment of soil strength/stiffness degradation parameters corresponding to a given cyclic history. The tests included a variety of pile types, methods of pile installation, sand $D_r$, pile diameters, pile lengths and cyclic loading characteristics. This study demonstrated that lateral pile behaviour is significantly dependent on the nature of applied cyclic loads, the sand $D_r$ value and pile installation method. Degradation of $p$-$y$ was found to be greatest for one-way cyclic loading, for looser sands, and for backfilled and drilled piles.

Further experimental investigations on the effect of cyclic load characteristics and $D_r$ were conducted in the centrifuge by Verdure et al. (2003), Rosquoët et al. (2007), and Klinkvort & Hededal (2013); at 1g in laboratory model pile tests by LeBlanc et al. (2010); and in pressure chamber tests by Baek et al. (2014). The trends observed in these studies are in general agreement and also consistent with the database of Long & Vanneste (1994).

Verdure et al. (2003) and Rosquoët et al. (2007) found that the rate of lateral pile displacement accumulation, with load cycles at positive $\zeta$, decreases with increasing $\zeta$. LeBlanc, Houlsby & Byrne (2010) and Klinkvort & Hededal (2013) extended this study to negative $\zeta$ values and showed maximum displacement rate accumulation at $\zeta$ of about -0.5 reducing as cycling becomes more two-way ($\zeta$ of -1). Klinkvort & Hededal (2013) showed reduction in pile head displacement for $\zeta$ of -1 in contrast to LeBlanc, Houlsby & Byrne (2010) which showed an increase. Klinkvort & Hededal (2013) suggests that the difference seen in the 1g tests by LeBlanc, Houlsby & Byrne (2010) may be due to the
difference in dilatant behaviour of sand between centrifuge and 1g testing. The rate of
displacement accumulation was also found to increase with $\varphi$ by LeBlanc, Houlsby &

Studies also showed that the maximum pile bending moments generally increase as $\zeta_c$
increases, with Verdure et al. (2003) and Rosquoët et al. (2007) observing the location of
maximum bending moment moving down the pile as cycling progresses. The increase in
bending moment was less than 20% for 50 load cycles and the moments increased in
approximate proportion to the logarithm of the number of cycles. Results reported by
Kirkwood & Haigh (2014) suggest that bending moments can increase even after 1000
cycles. Further work is still required to allow assessment of the potential build-up of
moments in a monopile over its design life.

The lateral pile stiffness measured over a given load cycle was observed to increase with
$\zeta_c$ and decrease with $\varphi$ in LeBlanc, Houlsby & Byrne (2010) and Klinkvort & Hededal
(2013). An increase in this stiffness with $\zeta_c$ was also recorded by Verdure et al. (2003)
and Rosquoët et al. (2007).

Baek et al. (2014) looked at the effect of $D_r$ on $p$-$y$ curves by conducting one-way cyclic
tests with various $\varphi$ values in sands with a wide range of densities. The authors found
that, under cyclic loading, $p$ increased in looser sands ($D_r = 40\%$) and reduced in dense
sand ($D_r > 70\%$). Reductions in $p$ were also seen in dense sands by Verdure et al. (2003)
and Rosquoët et al. (2007). The increase in $p$ is believed to be caused by the increase in
soil density from cycling and the reduction in $p$ is possibly due to the mobilised strength
reducing from the peak to the ultimate value.

The load eccentricity ($e$) is also a parameter of interest as higher $e$ gives rise to higher
applied moments on the piles. Klinkvort, Leth & Hededal (2010) conducted tests in the
centrifuge to look at the influence of $e$ and unsurprisingly found that piles experience larger displacements with an increasing moment arm.

### 3.1.2 Centrifuge tests

Centrifuge tests have recently been performed in the drum centrifuge at The University of Western Australia (UWA) to investigate the drained behaviour of laterally loaded piles under cyclic loading in saturated sands. Two static and four cyclic tests were carried out in loose to medium dense sand and the data from these tests are presented and discussed in this paper.

Fine to medium grained sub-angular silica sand (commercially available sand typically used in the centrifuges at UWA) was prepared in strong boxes with an internal length of 260 mm, width of 160 mm and depth of 160 mm. Each box was lined with geofabric for two-way drainage. The samples were prepared by wet pluviation to achieve a loose to medium dense sand. To measure the relative density and unit weight of the samples, tube samples were taken as well as records of the weight and volume of sand in each box. The averaged soil properties of the prepared samples are summarized in *Table 3.1-1*.

<table>
<thead>
<tr>
<th>$D_{50}$</th>
<th>$e_{\text{min}}$</th>
<th>$e_{\text{max}}$</th>
<th>$G_s$</th>
<th>$\gamma_{\text{dry}}$</th>
<th>$\gamma_{\text{sat}}$</th>
<th>$D_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>mm</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>kN/m$^3$</td>
<td>kN/m$^3$</td>
<td>%</td>
</tr>
<tr>
<td>0.19$^{(1)}$</td>
<td>0.49$^{(1)}$</td>
<td>0.76$^{(1)}$</td>
<td>2.65$^{(1)}$</td>
<td>16</td>
<td>20</td>
<td>40</td>
</tr>
</tbody>
</table>

$^{(1)}$ Values from Schneider and Lehane (2006)

The model pile used for the tests is an aluminium tubular pile of 10 mm diameter, 130 mm length and 1 mm wall thickness. It is fitted with 8 pairs of strain gauges with a higher concentration of strain gauges in the top half of the pile where highest bending moments generally occur. The strain gauges are protected by a layer of epoxy which increases the pile diameter by about 1 mm. The scaled prototype properties for 40g testing are summarized in *Table 3.1-2*. 
Two pile tests were conducted in each strong box sample. The piles were jacked into place by the actuator at 8 pile diameters from the boundary of the box and 10 pile diameters from the adjacent pile test site. Loading was applied towards the centre of the box to avoid boundary effects from the solid edges.

Cyclic loading was applied from the actuator using a mechanical arm attached to the pile cap which has a prototype height above ground level of 0.81 m. The arm allowed freedom for pile head movements in the vertical and horizontal direction. The final setup of the centrifuge test with the pile and loading arm in place is shown in Figure 3.1.1.

**Table 3.1-2. Scaled prototype pile properties**

<table>
<thead>
<tr>
<th>L (m)</th>
<th>D (m)</th>
<th>t (m)</th>
<th>EI (kNm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.78</td>
<td>0.44</td>
<td>0.06</td>
<td>57766</td>
</tr>
</tbody>
</table>

*Figure 3.1-1 Photo of drum centrifuge test in flight*
3.1.3 Pile behaviour observations

The findings from the UWA experiments are combined here with other experimental data referred to above to summarise effects of cyclic loading on pile head displacement, maximum pile bending moments, soil-pile stiffness and $p$-$y$ curves.

Test data were digitally extracted from the literature where sufficient information was available to allow for data manipulation and comparison. A total of 31 cyclic test data were collated, 4 from the centrifuge tests conducted for this study, 5 from centrifuge tests reported in Verdure et al. (2003), 7 from centrifuge tests described by Rosquoët et al. (2007) and 15 from 1g tests given in LeBlanc, Houlsby & Byrne (2010). The relevant details of the tests are summarised in Table 3.1-3. Adjustments have been made to published $\xi_b$ values as $H_{us}$ adopted for this study is the horizontal force required to displace the pile head by 10% of the pile diameter. This level of displacement will cause large movements of the structure which extends beyond the pile head in prototype scale.

The centrifuge tests in this study focused on severe wave loading with $\xi_b > 1$, whereas other studies as summarised in Table 3.1-3 covered wave loads with $\xi_b$ from 0.3 to 1.

Note also that the piles from the various references have different embedded length to diameter ($L/D$) and load eccentricity to diameter ($e/D$) ratios which are believed to be less significant compared to the cyclic load characteristics, and therefore, should not interfere with the findings of this study.

To assess the changes in pile response under cyclic loading, all data were normalised by the initial readings from the static loading phase at $H_{max}$. Maximum bending moment along the length of the pile after $N$ cycles ($M_{max,N}$) was normalised by initial maximum bending moment ($M_{max,1}$). Initial displacement ($y_1$) and soil-pile stiffness ($K_1$), and subsequent pile head displacements and stiffnesses after $N$ cycles ($y_N$ and $K_N$), were
determined from the pile head load displacement profiles as illustrated in the schematic diagram in Figure 3.1-2.

<table>
<thead>
<tr>
<th>Test</th>
<th>Dr</th>
<th>L/D</th>
<th>e/D</th>
<th>ζc</th>
<th>ζb</th>
<th>Nmax</th>
</tr>
</thead>
<tbody>
<tr>
<td>This paper – centrifuge tests at 40g in saturated sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
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<td>40</td>
<td>13.0</td>
<td>1.84</td>
<td>0.00</td>
<td>1.32</td>
<td>46</td>
</tr>
<tr>
<td>S4</td>
<td>40</td>
<td>13.0</td>
<td>1.84</td>
<td>-1.00</td>
<td>1.19</td>
<td>18</td>
</tr>
<tr>
<td>S5</td>
<td>40</td>
<td>13.0</td>
<td>1.84</td>
<td>-0.37</td>
<td>1.33</td>
<td>20</td>
</tr>
<tr>
<td>S6</td>
<td>40</td>
<td>13.0</td>
<td>1.84</td>
<td>0.43</td>
<td>1.78</td>
<td>20</td>
</tr>
<tr>
<td>Verdure et al. (2003) – centrifuge tests at 40g in dry sand</td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>C1-40</td>
<td>95</td>
<td>16.7</td>
<td>2.22</td>
<td>0.60</td>
<td>0.80</td>
<td>15</td>
</tr>
<tr>
<td>C1-60</td>
<td>95</td>
<td>16.7</td>
<td>2.22</td>
<td>0.40</td>
<td>0.80</td>
<td>15</td>
</tr>
<tr>
<td>C2-20</td>
<td>95</td>
<td>16.7</td>
<td>2.22</td>
<td>0.80</td>
<td>0.80</td>
<td>50</td>
</tr>
<tr>
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<td>2.22</td>
<td>0.60</td>
<td>0.80</td>
<td>50</td>
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<td>50</td>
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<tr>
<td>Rosquoët et al. (2007) – centrifuge tests at 40g in dry sand</td>
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<tr>
<td>P32</td>
<td>86</td>
<td>16.7</td>
<td>2.22</td>
<td>0.50</td>
<td>0.96</td>
<td>12</td>
</tr>
<tr>
<td>P33</td>
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<td>15</td>
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<td>19</td>
</tr>
<tr>
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<td>2.22</td>
<td>0.00</td>
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<td>18</td>
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<tr>
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<td>2.22</td>
<td>0.25</td>
<td>0.96</td>
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<tr>
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<td>86</td>
<td>16.7</td>
<td>2.22</td>
<td>0.75</td>
<td>0.96</td>
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</tr>
<tr>
<td>LeBlanc, Houlsby &amp; Byrne (2010) – 1g tests in dry sand</td>
<td></td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>L6</td>
<td>8</td>
<td>4.5</td>
<td>1.19</td>
<td>0.00</td>
<td>0.30</td>
<td>8200</td>
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<td>L7</td>
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<td>4.5</td>
<td>1.19</td>
<td>0.00</td>
<td>0.41</td>
<td>18200</td>
</tr>
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<td>0.51</td>
<td>8400</td>
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<td>1.19</td>
<td>0.00</td>
<td>0.60</td>
<td>17700</td>
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<td>4.5</td>
<td>1.19</td>
<td>0.00</td>
<td>0.80</td>
<td>8600</td>
</tr>
<tr>
<td>L11</td>
<td>8</td>
<td>4.5</td>
<td>1.19</td>
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<td>0.60</td>
<td>8510</td>
</tr>
<tr>
<td>L12</td>
<td>8</td>
<td>4.5</td>
<td>1.19</td>
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<td>7400</td>
</tr>
<tr>
<td>L13</td>
<td>8</td>
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<td>8800</td>
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<tr>
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<td>8</td>
<td>4.5</td>
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<td>4.5</td>
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<td>0.42</td>
<td>8090</td>
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<td>L17</td>
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<td>1.19</td>
<td>0.00</td>
<td>0.62</td>
<td>7423</td>
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<tr>
<td>L18</td>
<td>75</td>
<td>4.5</td>
<td>1.19</td>
<td>0.00</td>
<td>0.81</td>
<td>17532</td>
</tr>
<tr>
<td>L19</td>
<td>75</td>
<td>4.5</td>
<td>1.19</td>
<td>-0.50</td>
<td>0.62</td>
<td>9003</td>
</tr>
<tr>
<td>L20</td>
<td>75</td>
<td>4.5</td>
<td>1.19</td>
<td>-0.80</td>
<td>0.62</td>
<td>9814</td>
</tr>
<tr>
<td>L21</td>
<td>75</td>
<td>4.5</td>
<td>1.19</td>
<td>0.50</td>
<td>0.62</td>
<td>9862</td>
</tr>
</tbody>
</table>
Figure 3.1-2. Schematic diagram of applied load against pile head displacement for calculating soil-pile stiffness

**Pile head displacements**

The normalised pile head displacements from the collated data are compared in Figure 3.1-3. Although the data in LeBlanc, Houlsby & Byrne (2010) were presented as normalised pile rotation values, the normalised pile rotation at small displacements is nearly identical to the normalised displacements, and therefore, the data are comparable.

Figure 3.1-3. Pile head displacement accumulation
The trend of normalised displacements with $N$ is roughly linear on a log-log scale. Therefore, $y_N$ can be represented by a power law, given in Equation 3.1-3 as:

$$y_N = y_1 N^{\alpha_y}$$  \hspace{1cm} (3.1-3)

where $\alpha_y$ is a fitting coefficient which is indicative of the accumulation rate of pile head displacement. The $\alpha_y$ values are plotted against $\zeta_c$ and $\zeta_b$ in Figure 3.1-4 and Figure 3.1-5 respectively.

The results indicate that displacement accumulation increases (i.e. as $\alpha_y$ increases) with decreasing $\zeta_c$ from $\zeta_c = 1$ (static loading) to a maximum at around $\zeta_c = -0.5$ and then reduces towards $\zeta_c = -1$ (two-way loading) where ‘negative accumulation’ ($\alpha_y < 0$) was observed in a two-way cyclic test (test S4). This negative accumulation is consistent with centrifuge tests in dense sand by Klinkvort & Hededal (2013). Figure 3.1-4 also shows that similar trends for $\alpha_y$ occur at all $D_r$ values. A tentative equation to estimate upper bound values of $\alpha_y$ is provided below:

$$\alpha_y = (1 - e^{\zeta_c^{-1}})(0.5 \zeta_c^3 - 0.25\zeta_c + 0.25)$$  \hspace{1cm} (3.1-4)

A wider range of $\zeta_c$ and $\zeta_b$ needs to be investigated to develop a function which incorporates both variables.

The effects of $\zeta_b$ were investigated in LeBlanc, Houlsby & Byrne (2010) for one-way cyclic tests ($\zeta_c = 0$) and back figured $\alpha_y$ values are presented in Figure 3.1-5. An increase in $\alpha_y$ with $\zeta_b$ is observed which is in line with expectations as more displacements are expected for larger values of $H_{\text{max}}$. It is also observed that $\alpha_y$ is somewhat larger for denser soils which may be partly because $y_1$ values in dense sand are considerably lower than those in loose sand. Results from other one-way tests presented in Figure 3.1-5 have lower $\alpha_y$ values at larger $\zeta_b$. More test data is required to determine whether this behaviour is related to the pile $L/D$ and or significantly larger $\zeta_b$ values.
Figure 3.1-4. Effect of $\zeta_c$ on pile head displacement accumulation coefficient, $\alpha_y$

Figure 3.1-5. Effect of $\zeta_b$ on pile head displacement accumulation coefficient, $\alpha_y$
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**Pile bending moments**

The normalised maximum pile bending moments are presented in *Figure 3.1-6*. It is evident that, as with normalised displacements on *Figure 3.1-4*, the normalised moments vary linearly with $N$ when plotted on a log-log scale. Moments after $N$ cycles may therefore be represented as:

$$M_{\text{max},N} = M_{\text{max},1}N^b$$  \hspace{1cm} (3.1-5)

where $b$ is a coefficient indicative of the change in the bending moment with each cycle.

The variation of the $b$ coefficient with $\zeta_c$ is plotted in *Figure 3.1-7*, with indicative trends lines shown for the various soil densities. The general trend observed is that $b$ increases with $\zeta_c$ and is greater in denser sand. These $b$ values are very small and indicate that changes in bending moment over the design life of a monopile are likely to be relatively small.

It is also interesting to note that $b$ can be negative (i.e. bending moment reduces with cycles) in the loose to medium dense sand with $\zeta_c \leq 0$. A reduction in moment is explained by an increase in soil reaction as the pile displacement compacts the sand after each cycle. Whether the bending moment will stabilize or increase after a larger number of cycles will need to be further investigated.

For the test with $\zeta_c > 0$ in the loose to medium dense sand, a small increase in bending moment is observed which may be due to the low amplitude in the cyclic load (greater $\zeta_c$ value) or the larger $H_{\text{max}}$ (greater $\zeta_b$ value). However, the effect of $\zeta_b$ on bending moment is inconclusive from this study due to the limited number of tests at the same $\zeta_c$ value with varying $\zeta_b$. 
Chapter 3. Lateral piles in sand

Figure 3.1-6. Normalised maximum pile moments

Figure 3.1-7. Effect of $\zeta_c$ and $D_r$ on maximum pile moments
Soil-pile stiffness

The soil-pile stiffness has been calculated at the point of load application which is at the pile head for all the test data used in this study (see Figure 3.1-2). The normalized stiffness values are compared in Figure 3.1-8. A large initial increase in stiffness from $K_1$ was expected and is observed in each test case which then becomes linear on log scale as $N$ increases. This behaviour may be represented by Equation 3.1-6:

$$K_N = K_1 \{1 + C \ln(N)^{0.2} \}$$

(3.1-6)

where $C$ is a coefficient relating to the change in stiffness and is plotted against $\zeta_c$ in Figure 3.1-9. From Figure 3.1-9, $C$ is observed to increase with $\zeta_c$. This is likely due to smaller displacement (and hence strain) accumulation at lower amplitudes of cycling compared to the strain induced by the initial static load. $C$ also appears to be smaller for larger $D_r$ as looser sands have more potential to compact and hence improve their stiffness.
Figure 3.1-8. Normalised soil-pile stiffness

Figure 3.1-9. Effect of $\zeta$ on soil-pile stiffness
**Soil reaction p-y curves**

Under cyclic loading, p-y curves at various depths along the pile exhibit hysteresis in any given cycles, as shown in *Figure 3.1-10*. To simplify presentation of the cyclic p-y plots, Verdure *et al.* (2003) and Rosquoët *et al.* (2007) plotted p-y curves during static loading and used symbols to represent $p$ at $H_{\text{max}}$ for each load cycle. In so doing, Verdure *et al.* (2003) and Rosquoët *et al.* (2007) observed a reduction in $p$ with number of cycles. This reduction explains the increase in bending moments observed in *Figure 3.1-7* and Verdure *et al.* (2003) suggest that this occurrence may be due to the soil exceeding its peak strength. It is also unknown whether this degradation will continue, although it is expected to stabilise as the soil reaches its critical state strength. The opposite trend was observed in test S2 and S5 where $p$ increased with cycles in looser sand; this trend was also observed in Baek *et al.* (2014). Simplified p-y curves from S5 are compared to S6 in *Figure 3.1-11* to illustrate the occurrence of $p$ both increasing and decreasing with cycling. The increase in $p$ in test S5 is also consistent with the reduction in bending moment in *Figure 3.1-7* and compaction of the looser sand by the cyclic action.

![Figure 3.1-10. p-y curves at various depths for test S5](image-url)
3.1.4 Conclusions

This paper has collated data from the literature and combined these with results obtained in centrifuge tests recently conducted in loose to medium dense silica sand at UWA. These recent tests focused on the effects of cyclic load characteristics and relative density on the lateral pile response in sand. A total of 31 tests were used in this study to deduce the following trends:

(i) Pile head displacement accumulation increases with $\zeta_b$, and increases as $\zeta_c$ reduces from 1 (static loading) to -0.5. The rate of displacement accumulation is greatest at a $\zeta_c$ value of about -0.5 then reduces towards a $\zeta_c$ value of -1 (two-way cycling). A tentative equation (Equation 3.1-4) based on the experimental data is provided which, when used with Equation 3.1-3, provides designers with an upper bound estimate for displacement accumulation at $\zeta_c > -1$.

(ii) Bending moment accumulation may increase or decrease with increase in cycles depending on the values of $\zeta_c$ and $D_r$. Bending moment accumulation is generally greater for denser soils.

Figure 3.1-11. $p$-$y$ curves for test S5 and S6 showing $p$ at $H_{max}$
(iii) Lateral pile stiffness increases significantly after the initial static load is applied and subsequently the increases vary approximately with the logarithm of the number of applied cycles. The change in stiffness increases with $\zeta_c$.

(iv) Soil reaction can reduce or increase during cycling, depending on the load characteristics and $D_r$. Loose to medium dense sands with negative $\zeta_c$ values in the UWA centrifuge tests show an increase in $p$ with cycling. Other tests have shown a decrease in $p$ with cycling in dense sands with positive $\zeta_c$ values.

Further testing is still required to cover a wider range $D_r$, $\zeta_c$ and $\zeta_b$ combinations to confirm the trends inferred in this study.
3.2 Effects of sand density and cyclic load sequences

A systematic study into the response of monopiles to lateral cyclic loading in medium dense and dense sand was performed in beam and drum centrifuge tests. The centrifuge tests were carried out at different cyclic load and magnitude ratios, while the cyclic load sequence was also varied. The instrumentation on the piles provides fresh insights into the ongoing development of net stresses, bending moments and deflections as cycling progresses. Parallels between the test results and corresponding cyclic triaxial tests are drawn. The paper combines the results from this study with those from previous experimental investigations to provide design recommendations for monopiles subjected to unidirectional cyclic loading.

3.2.1 Background

The preferred foundation type for offshore wind turbines is the monopile (or single pile), which has been used for 80% of currently installed offshore turbines (Pineda & Tardieu, 2017). Existing methods for prediction of the lateral response of these piles, such as the API (2011) recommendations, are largely based on research on small diameter piles reported by Reese et al. (1974), Murchison & O’Neill (1984), and others. Concern over the applicability of these recommendations to monopiles, which often have diameters in excess of 5m and slenderness ratios of as low as 3 to 4, has motivated a considerable body of research in the past decade. One of the important foundation considerations is the accommodation in design for lateral cycling of monopiles due to wind and wave loading over the life of the turbine. Such cycling can lead to the accumulation of a significant permanent rotation of a monopile and overlying tower, rendering the turbine unserviceable. Wind turbine tilt tolerance and permanent foundation rotation at the soil surface is specified by the wind turbine manufacturer as it varies for different turbines. Golightly (2014) quotes a typical limiting rotation of 0.5° inclusive of a 0.25°
construction tolerance suggested by DNV (2016), therefore giving an allowable accumulated rotation due to cycling of 0.25°.

This paper focuses on the response of monopiles in sand to (uni-directional) lateral cycling and draws on evidence reported in recent experimental research as well as observations made in new centrifuge-scale lateral pile cycling experiments to provide guidance to enable assessment of the accumulation of permanent rotations for monopiles in sand. Numerical research into the prediction of such rotations is still at the development stage (Achmus, Kuo & Abdel-Rahman, 2009; Giannakos, Gerolymos & Gazetas, 2012; Su & Li, 2013; Rudolph, Bienen & Grabe, 2014; Depina et al., 2015; Zachert et al., 2016) and the experimental trends identified here can support the calibration of future numerical models.

To assist comparison with recent experimental research, the nature of the cyclic loads is defined using the cyclic load ratio ($\zeta_c$) and cyclic magnitude ratio ($\zeta_b$), defined as follows:

$$\zeta_c = \frac{H_{\text{min}}}{H_{\text{max}}} = \frac{M_{\text{min}}}{M_{\text{max}}} \quad (3.2-1)$$

$$\zeta_b = \frac{H_{\text{max}}}{H_{\text{ref}}} = \frac{M_{\text{max}}}{M_{\text{ref}}} \quad (3.2-2)$$

where $H_{\text{min}}$ and $H_{\text{max}}$ are the minimum and maximum horizontal loads applied to the pile with corresponding moments applied to the pile at the soil surface of $M_{\text{min}}$ and $M_{\text{max}}$; for two-way loading, $H_{\text{min}}$ and $H_{\text{max}}$ are negative and positive respectively. The reference horizontal force and applied moment ($H_{\text{ref}}$ and $M_{\text{ref}}$) are those corresponding to monotonic loading at failure or at a reference displacement or rotation ($y_{\text{ref}}$ or $\theta_{\text{ref}}$) at the soil surface.

As a geotechnical failure for a laterally loaded pile in sand under monotonic loading can require large pile rotations, it has become common practice to define $H_{\text{ref}}$ at a smaller $\theta_{\text{ref}}$ value, such as the value of 4° employed by Leblanc, Houlsby & Byrne (2010) and Klinkvort & Hededal (2013).
A typical response to uniform lateral cycling is shown on Figure 3.2-1, which also defines the cyclic load and rotational stiffness \((K_r)\) parameters. Uniform cyclic loading causes a progressive accumulation of permanent pile rotation (and pile head displacement), with the additional rotation developed in each cycle reducing as the number of cycles \((N)\) increases.

![Figure 3.2-1. Schematic diagram for cycle number assignment and determining rotational stiffness](image)

Although accumulated rotation is often considered to vary with the logarithm of the number of cycles (Long & Vanneste, 1994), lateral pile experiments described by Leblanc, Houlsby & Byrne (2010), Klinkvort & Hededal (2013), Truong & Lehane (2015) and Li et al. (2015), show that, for a given level and type of cycling, the ratio of rotation accumulated after \(N\) cycles \((\theta_N)\) to the maximum (positive) rotation reached in the first cycle \((\theta_1)\) is best represented for rigid piles as a power function of \(N\) i.e.

\[
\theta_N = \theta_1 N^{\alpha_r}
\]

(3.2-3a)

where \(\alpha_r\) is referred to here as the accumulation coefficient (with respect to rotation).
The equivalent equation written in terms of displacements ($y$) at the ground surface is:

$$y_N = y_1N^{\alpha_y} \quad \text{(3.2-3b)}$$

where $\alpha_y$ is the accumulation coefficient with respect to lateral pile displacement. Li et al. (2015), and others, show that $\alpha_y$ is a little larger than $\alpha$. This difference can be explained by an increase with cycling of the depth about which the pile rotates due, for example, to the formation of a ‘post-hole’ near the surface. The formation of post-hole is consistent with the slight increase in the maximum pile bending moment with $N$ observed by Verdure et al. (2003), and Rosquoët et al. (2007). Cuéllar et al. (2012) used optical measurement techniques to deduce that lateral cycling causes an increase in sand density (and sand stiffness) with accompanying downward migration (or convection) of sand grains as the post-hole near the surface increases in size. The boundary conditions existing after $N$ cycles therefore differ from those after the first cycle of loading and this effect should be acknowledged when employing expressions such as Equation 3.2-3, which relate the pile response after $N$ cycles to that at $N = 1$.

Long & Vanneste (1994) collated existing field test data from 34 studies and concluded, in general, that pile head movements were dependent on the nature of applied cyclic loads, the sand relative density ($D_r$) and the pile installation method. The subsequent systematic experimental investigations of Rosquoët et al. (2007), Klinkvort & Hededal (2013) and Truong & Lehane (2015) show that the accumulation coefficient ($\alpha_y$) depends primarily on the cyclic load ratio ($\zeta_c$) and to a lesser extent the cyclic magnitude ratio ($\zeta_b$), and varies from a negative value for two-way loading ($\zeta_c = -1$) to between about 0.05 and 0.2 at $\zeta_c$ values in the range of -0.5 to 0.75 (noting $\zeta_c = 0$ corresponds to one-way loading with $H_{\min} = 0$). LeBlanc et al. (2010) and Albiker et al. (2017) present formulations based on 1-g small scale tests in which the accumulation coefficient is considered independent of sand relative density, although scale issues associated with the very low stress level
prevalent in these tests need to be acknowledged. It is also of note that Albiker et al. (2017) observed a dependency of the accumulation coefficient on the lateral load eccentricity as well as a difference between rigid and flexible piles.

Cyclic loading applied to monopiles is not uniform and a number of studies have examined the ability of the method of superposition presented by Lin & Liao (1999) to predict pile response under various packages of (uni-directional) cycles. Field tests in dense sand reported by Li et al. (2015) show that this method provided a reasonable predictive approach for cyclic histories involving one-way loading with progressively increasing cyclic magnitude ratios. However, Peralta & Achmus (2010) showed in 1-g model experiments that the order in which a given set of cyclic packages was applied did influence the final values of accumulated rotations.

It is evident from the foregoing that a range of experimental observations have been made, largely in centrifuge scale model tests, but also in other 1-g small-scale laboratory tests and field scale tests. This paper extends this database of information using a carefully planned series of centrifuge tests and subsequently compiles all observations to develop recommendations for assessment of the accumulation of permanent rotations for monopiles installed in a range of sand densities. The paper also investigates the potential of using cyclic triaxial tests to estimate the relevant value of the accumulation coefficient (Equation 3.2-3) and, based on the centrifuge test results, formulates a proposal to treat piles subjected to variable lateral cycling histories. The pile instrumentation employed facilitates examination of the development with cycling of the maximum and residual pile bending moments and quantification of the effects of cycling on post-cyclic monotonic pile response.
3.2.2 Centrifuge tests

Lateral cyclic pile tests were performed in the drum centrifuge at the University of Western Australia (UWA) and in the beam centrifuge at the Technical University of Denmark (DTU). The preliminary series of tests performed in the drum centrifuge used a relatively small model pile and comprised application of only 50 cycles of lateral load. The series of beam centrifuge tests was more comprehensive and used a larger model pile with a greater number of cycles and load eccentricity; this test series involved a systematic investigation of the effects of cyclic load characteristics and cyclic load sequences, and also is the first test series to employ medium dense sand as well as dense sand in the centrifuge at in-situ stress levels. The test schedule along with the corresponding cyclic load characteristics for all the tests conducted are provided in Table 3.2-1 where the UWA monotonic test is prefixed with UWA-M, DTU monotonic tests are prefixed with DTU-M, UWA cyclic tests are prefixed with UWA-C, and DTU cyclic tests are prefixed with DTU-C. Note that in this study, $H_{ref}$ is defined at a rotation at the sand surface equal to $0.5^\circ$ to be consistent with serviceability requirements for wind turbine foundations. The flexural rigidities of the centrifuge piles are relatively high and such that, as shown later, rotated in a rigid manner under application of loads (and hence in a similar way to full scale monopiles).

UWA drum centrifuge equipment and setup

The drum centrifuge tests were performed in ‘UWA sand’ (described below) at an acceleration of 250g. This centrifuge has an outer radius of 0.6 m, channel depth of 0.175 m and width of 0.3 m (Stewart et al. 1998). A schematic of the test set-up is presented in Figure 3.2-2. The sand sample had a final depth in the channel of 170 mm and was prepared in flight using procedures similar to those described by Lehane & White (2005). The test pile, with details provided in Table 3.2-2, was an aluminium hollow cylindrical tube, fitted with 8 pairs of strain gauges in a half bridge and protected by a
0.5 mm thick layer of epoxy; these gauges were calibrated at 1-g in the laboratory and enabled measurement of bending strains and hence bending moments throughout the tests. The pile was initially installed at 1-g using a tool attached to the drum actuator. The installation tool was then exchanged for the lateral loading arm which had a wheel end to provide a single point of contact for load application. This arrangement allowed the pile head to rotate freely but could only apply one-way loading. After pile installation, the sample was slowly re-saturated while ramping up to 250g. Lateral loading commenced at least two hours after reaching 250g. The load controlled cycles were applied in a triangular waveform at a slow speed of 0.015 mm/s to ensure fully drained conditions.

<table>
<thead>
<tr>
<th>Test no.</th>
<th>Centrifuge</th>
<th>g level</th>
<th>$D_r$ (%)</th>
<th>Total no. of Cycles</th>
<th>Cyclic load characteristics</th>
<th>Load 1</th>
<th>Load 2</th>
<th>Load 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\zeta_c$</td>
<td>$\zeta_b$</td>
<td>$\zeta_c$</td>
</tr>
<tr>
<td>UWA-M</td>
<td>Drum</td>
<td>250</td>
<td>68</td>
<td>Mono</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>UWA-C-1</td>
<td>Drum</td>
<td>250</td>
<td>68</td>
<td>50</td>
<td>0.01</td>
<td>1.04</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>UWA-C-2</td>
<td>Drum</td>
<td>250</td>
<td>68</td>
<td>50</td>
<td>0.33</td>
<td>1.05</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>UWA-C-3</td>
<td>Drum</td>
<td>250</td>
<td>68</td>
<td>50</td>
<td>0.71</td>
<td>1.05</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>UWA-C-4</td>
<td>Drum</td>
<td>250</td>
<td>68</td>
<td>50</td>
<td>0.04</td>
<td>0.48</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>UWA-C-5</td>
<td>Drum</td>
<td>250</td>
<td>68</td>
<td>50</td>
<td>0.33</td>
<td>0.48</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>UWA-C-6</td>
<td>Drum</td>
<td>250</td>
<td>68</td>
<td>50</td>
<td>0.72</td>
<td>0.48</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>DTU-M-1</td>
<td>Beam</td>
<td>60</td>
<td>60</td>
<td>Mono</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>DTU-C-1</td>
<td>Beam</td>
<td>60</td>
<td>64</td>
<td>1500</td>
<td>0.05</td>
<td>0.49</td>
<td>0.04</td>
<td>0.74</td>
</tr>
<tr>
<td>DTU-C-2</td>
<td>Beam</td>
<td>60</td>
<td>58</td>
<td>1500</td>
<td>0.03</td>
<td>1.00</td>
<td>0.12</td>
<td>0.49</td>
</tr>
<tr>
<td>DTU-C-3</td>
<td>Beam</td>
<td>60</td>
<td>51</td>
<td>1500</td>
<td>0.01</td>
<td>0.74</td>
<td>0.02</td>
<td>0.49</td>
</tr>
<tr>
<td>DTU-C-4</td>
<td>Beam</td>
<td>60</td>
<td>52</td>
<td>900</td>
<td>0.50</td>
<td>0.50</td>
<td>-0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>DTU-C-5</td>
<td>Beam</td>
<td>60</td>
<td>63</td>
<td>900</td>
<td>-0.53</td>
<td>0.47</td>
<td>-1.15</td>
<td>0.45</td>
</tr>
<tr>
<td>DTU-C-6</td>
<td>Beam</td>
<td>60</td>
<td>50</td>
<td>900</td>
<td>-1.13</td>
<td>0.45</td>
<td>0.46</td>
<td>0.45</td>
</tr>
<tr>
<td>DTU-M-2</td>
<td>Beam</td>
<td>60</td>
<td>88</td>
<td>Mono</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>DTU-C-7</td>
<td>Beam</td>
<td>60</td>
<td>85</td>
<td>1500</td>
<td>0.01</td>
<td>0.53</td>
<td>0.02</td>
<td>0.80</td>
</tr>
<tr>
<td>DTU-C-8</td>
<td>Beam</td>
<td>60</td>
<td>95</td>
<td>900</td>
<td>0.53</td>
<td>0.55</td>
<td>-0.42</td>
<td>0.58</td>
</tr>
</tbody>
</table>
Table 3.2-2. Summary of test pile properties from this study and tests for comparison study

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>D_model (m)</td>
<td>0.011(1)</td>
<td>0.040(1)</td>
<td>0.028(1) and 0.040(1)</td>
<td>0.018</td>
<td></td>
</tr>
<tr>
<td>D_prototype (m)</td>
<td>2.75</td>
<td>3.92</td>
<td>3.0</td>
<td>0.72</td>
<td>0.34</td>
</tr>
<tr>
<td>t_metal (m)</td>
<td>0.001</td>
<td>0.0015</td>
<td>Solid</td>
<td>0.0015</td>
<td>0.014</td>
</tr>
<tr>
<td>t_epoxy (m)</td>
<td>0.0005</td>
<td>0.001</td>
<td>0.002</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D/t</td>
<td>7.3</td>
<td>16</td>
<td>n/a</td>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td>E_I_prototype (GNm²)</td>
<td>88.1</td>
<td>74.4</td>
<td>77.7(2) to 522(2)</td>
<td>0.476</td>
<td>0.038(2)</td>
</tr>
<tr>
<td>Sand type</td>
<td>Fine Silica Sand</td>
<td>Fontainebleau Silica Sand</td>
<td>Fontainebleau Silica Sand</td>
<td>Fontainebleau Silica Sand</td>
<td>Blessington fine sand</td>
</tr>
<tr>
<td>Rf %</td>
<td>68</td>
<td>50 to 99</td>
<td>79 to 96</td>
<td>86</td>
<td>100</td>
</tr>
<tr>
<td>L/D</td>
<td>11.4</td>
<td>6</td>
<td>6</td>
<td>16.7</td>
<td>6.5</td>
</tr>
<tr>
<td>e/D</td>
<td>2</td>
<td>3</td>
<td>15</td>
<td>2.22</td>
<td>1.17</td>
</tr>
<tr>
<td>Toe condition</td>
<td>Open</td>
<td>Open</td>
<td>Closed</td>
<td>Open</td>
<td>Open</td>
</tr>
<tr>
<td>Installation</td>
<td>Jacked at 1g</td>
<td>Jacked at 60g</td>
<td>Jacked in lower g field</td>
<td>Driven at 1g</td>
<td>Driven</td>
</tr>
</tbody>
</table>

(1) Includes thickness of epoxy
(2) Assuming Young’s modulus of elasticity of 200GPa.

DTU beam centrifuge equipment and setup

The 2.5 m radius beam centrifuge tests at DTU was employed to carry out lateral pile tests in dry Fontainebleau sand (described below) at an acceleration of 60g. The strongbox
had an internal diameter of 527 mm and internal height of 460 mm. The sand sample was pluviated at 1-g into the strongbox using a tube that was manoeuvred manually in a circular motion at a constant drop height above the sand surface. Drop heights of 50 mm and 200 mm were employed to achieve measured average relative densities of 57% and 90% respectively. The sand was levelled off to give a final sample height 337 mm. The relative densities of the samples were derived from the measured total sample weight and volume and are reported in Table 3.2-1. Full details of the centrifuge and the sand preparation method are provided in Leth (2013).

The loading frame was secured on top of the strongbox, as shown in the schematic diagram in Figure 3.2-3. This frame allowed inflight pile installation and subsequent lateral loading without ramping down the centrifuge to switch pile head connections. Lateral loading was applied via the loading frame which was driven by a motor across the top of the sample. Load controlled cyclic loading was applied in a periodic trapezoidal form (see inset on Figure 3.2-8) with a period of 10s. A hinge at the top of the pile head allowed rotation during the application of lateral load. The pile head was, however, fixed in the vertical direction resulting in axial tensile forces when lateral loads were applied. These forces were measured and found to be minimal at the relatively small pile head rotations under consideration in this study; their effects are nonetheless likely to have had some influence during the post-cyclic tests when piles were subjected to rotations in excess of 5°. Numerical analyses reported by Karthigeyan, Ramakrishna & Rajagopal (2006) and Hazzar, Hussien & Karray (2017) indicated that the presence of axial pile forces has a minimal effect on the lateral sand response.

The (hollow steel) model pile employed had the dimensions and properties summarised in Table 3.2-2. The pile was fitted with 15 pairs of calibrated strain gauges arranged in a half bridge and protected by 1 mm thick layer of epoxy. Lateral movement of the pile was
measured by two laser displacement transducers set at 20 mm and 70 mm above the sand surface.

![Figure 3.2-3. Schematic elevation of DTU beam centrifuge pile test setup](image)

**Strain gauge interpretation**

Both the UWA and DTU test series utilised instrumented piles to obtain full bending moment profiles during loading. The profiles of bending moments ($M$) were used to interpret corresponding profiles of the net soil pressures ($P$), pile rotation ($\theta$) and lateral displacements ($y$) using the standard beam equations (where $EI$ is the pile flexural rigidity) i.e.
\[ p = P \times D = -\frac{d^2 M(z)}{dz^2} \]  

(3.2-4)

\[ y = \int M(z)/EI \, dz \]  

(3.2-5)

\[ \theta = \frac{dy}{dz} \]  

(3.2-6)

Various curve fitting methods were trialled when deriving net pressures from bending moments. Overlapping cubic polynomials, proposed by Yang & Liang (2014), were found to be the most suitable and the interpreted \( P-y \) responses gave the most accurate predictions of load-displacement curves observed in monotonic tests when re-input into a standard load transfer (\( P-y \)) laterally loaded pile program. It should be noted that Fan & Long (2005), Zania & Hededal (2011) and Suryasentana & Lehane (2016) have used numerical analyses to show that \( P-y \) curves derived from bending moment profiles are independent of the pile rigidity.

Two constants are required to determine lateral pile displacements (\( y \)) using Equation (3.2-5) and these are usually obtained using measurements of displacement at two different locations. Both sets of centrifuge tests had only one displacement reading near the pile head (as the top laser in the DTU test malfunctioned) and therefore, to obtain a second displacement value, it was assumed that zero pile deflection occurred at the (lowest) location where the net pressure was zero. This assumption was consistent with observations in the DTU tests when both laser displacement transducers were working.

**Sand sample properties**

The UWA and Fontainebleau silica sands have almost identical gradings, each having a mean effective particle size \( (D_{50}) \) of 0.18 mm and a uniformity coefficient of about 1.6. Detailed descriptions of the properties of the UWA and Fontainebleau sands are provided in Chow *et al.* (2017) and Latini (2017) respectively. Cone Penetration Tests (CPTs) performed in both sets of sand samples confirmed the measurements of relative densities
and also showed that the sample preparation process was successful in producing uniform samples. Triaxial tests, with test details given in *Table 3.2-3*, were performed on medium dense \((D_r = 55\%)\) and dense \((D_r = 90\%)\) samples of Fontainebleau sand to assist interpretation of the lateral cyclic tests. Prior to shearing, the samples were consolidated isotropically to 100 kPa, which corresponds to the vertical effective stress at half the embedment of the piles in the DTU centrifuge. A single monotonic test and two one-way cyclic tests with maximum deviator stresses \((q_{max})\) of 0.5 and 0.75 times the ultimate deviator stress \((q_f)\) were carried out for each sand density. For the medium dense sand sample subjected to cycling at 0.5\(q_f\), two additional packages of one-way cycles with maxima of 0.75\(q_f\) and 0.2\(q_f\) were applied to compare the response to different cyclic load packages with a similar loading regime in the centrifuge. The responses observed in all of the triaxial tests are compared below with the lateral pile test results.

### Table 3.2-3. Cyclic triaxial test

<table>
<thead>
<tr>
<th>Test no.</th>
<th>TX-M-1</th>
<th>TX-C-1</th>
<th>TX-C-2</th>
<th>TX-M-2</th>
<th>TX-C-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Mono</td>
<td>Cyclic</td>
<td>Cyclic</td>
<td>Cyclic</td>
<td>Mono</td>
</tr>
<tr>
<td>(D_r) (%)</td>
<td>55</td>
<td>55</td>
<td>55</td>
<td>55</td>
<td>90</td>
</tr>
<tr>
<td>(q_f) (kPa)</td>
<td>319</td>
<td>342</td>
<td>-</td>
<td>-</td>
<td>421</td>
</tr>
<tr>
<td>(q_{max}) (kPa)</td>
<td>-</td>
<td>242</td>
<td>170</td>
<td>235</td>
<td>-</td>
</tr>
<tr>
<td>(q_{max}/q_f)</td>
<td>-</td>
<td>0.7</td>
<td>0.5</td>
<td>0.7</td>
<td>-</td>
</tr>
<tr>
<td>N</td>
<td>-</td>
<td>1000</td>
<td>500</td>
<td>500</td>
<td>-</td>
</tr>
<tr>
<td>(\alpha_{triax})</td>
<td>-</td>
<td>0.143</td>
<td>0.13</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

#### 3.2.3 Pile test observations

**Monotonic tests**

The variations of the normalised applied lateral load \((H)\) with rotation \((\theta)\) and normalised displacement \((y/D)\) at the ground surface for the monotonic tests are plotted on *Figure 3.2-4*; test details are provided in *Table 3.2-1*. The normalisation adopted for
lateral load is the same as that employed by LeBlanc, Houlsby and Byrne (2010), where \( \sigma_{vl}' \) is the in-situ vertical effective stress at the base of the pile. This normalisation allows comparison of Test UWA-M, which was conducted in saturated conditions, with Tests DTU-M-1 and DTU-M-2 which were performed in dry sand.

It is evident that capacity at large rotations and displacements will increase with the sand relative density \( (D_r) \) and the pile slenderness ratio \( (L/D) \). However the normalised reference lateral loads \( (H_{ref}) \), defined in this study at a small deformation of 0.5° rotation, show a small dependence on \( D_r \) for a fixed \( L/D \). Normalised \( H_{ref} \) values are 0.15, 0.47 and 0.54 in tests UWA-M, DTU-M-1 and DTU-M-2 respectively. The trends found in centrifuge tests of Klinkvort et al. (2013) and Dyson & Randolph (2001) suggest that the softer variation of \( H/\sigma_{vl}' DL \) with \( y/D \) seen in test UWA-M is, at least in part, due to its 1-g installation. The difference between the two pile rigidity indices \( (EI) \), given in Table 2, also contributes to the softer overall pile behaviour in UWA-M. The ratio of cycling induced displacements to the monotonic displacement for the UWA piles, discussed

\[
\begin{align*}
\text{UWA-M} & \quad D_r = 68\% \\
& \quad e/D = 2 \\
& \quad L/D = 11.4 \\
& \quad \sigma_{vl}' = 257\text{kPa} \\
\text{DTU-M-1} & \quad D_r = 60\% \\
& \quad e/D = 3 \\
& \quad L/D = 6 \\
& \quad \sigma_{vl}' = 223\text{kPa} \\
\text{DTU-M-2} & \quad D_r = 88\% \\
& \quad e/D = 3 \\
& \quad L/D = 6 \\
& \quad \sigma_{vl}' = 236\text{kPa} \\
\end{align*}
\]

Figure 3.2-4. Monotonic lateral pile test results
below, is likely to be less influenced by their installation at 1-g.

**Accumulation of permanent rotation**

The accumulation of permanent rotations with cycling in one-way cyclic tests ($\zeta_c \geq 0$) is investigated on Figure 3.2-5a, which uses logarithmic axes to present the variation with the number of cycles ($N$) of the measured maximum rotations after $N$ cycles ($\theta_N$). The near linear variations seen for each designated test are indicative of constant accumulation coefficients ($\alpha_r$) and confirm the general suitability of Equation (3.2-3a). It is evident on inspection of Figure 3.2-5a that the slopes of these variations tend to reduce with increasing density. While these slopes are not affected significantly by the cyclic magnitude ratio ($\zeta_b$), $\theta_N$ clearly increases with the level of cycling (i.e. with $\zeta_b$). This tendency is incorporated in Equation (3.2-3a) by allowing $\theta_N$ to vary directly with $\theta_1$.

Evidence in support of this approach is apparent on Figure 3.2-5b which shows very similar variations for respective relative densities of $\theta_N/\theta_1$ ratios with $N$ (using logarithmic axes) for tests with a range of $\zeta_b$ values but similar $\zeta_c$ values (of close to zero).

Under 2-way cycling ($\zeta_c < 0$), the test with $\zeta_c = -0.53$ and $\zeta_b = 0.47$ (DTU-C-5) showed a continual increase in the accumulated rotation in the direction of the load cycle bias, with $\alpha$ similar to that of one-way cycling at the same density. However for test DTU-C-6, with near symmetrical two-way loading ($\zeta_c = -1.13$ and $\zeta_b = 0.45$), the accumulated rotation is very small and the pile moves slightly ‘backwards’ from the initial forward movement direction.
Figure 3.2-5. (a) Pile rotations at sand surface for one-way cyclic tests, and (b) Normalised pile rotations for $\zeta = 0$. 

(a) 

(b) 

Figure 3.2-5. (a) Pile rotations at sand surface for one-way cyclic tests, and (b) Normalised pile rotations for $\zeta = 0$. 

Note: The figure shows graphs of pile rotation at sand surface versus number of cycles for different pile groups, with annotations indicating the pile group and percentage of rotation. The normalised pile rotations are also shown for different pile groups, with annotations indicating the pile group and percentage of rotation for $\zeta = 0$. The graphs are plotted on a log-log scale.
Changes in pile lateral stiffness with cycling

A large change in rotational stiffness of a monopile over its lifetime could be an important consideration for assessment of the dynamic response of offshore wind turbines. The unload-reload rotational stiffness, $K_{r,N}$ (as defined in Figure 3.2-1) increased with the number of cycles in all experiments. However, the increases after about 10 cycles were very small (i.e. $K_{r,10} \sim K_{r,500}$) and $K_r$ stabilised at value referred to here at $K_{r,\text{stab}}$. The ratio of this stabilised value to the $K_r$ value observed at $N = 1$ ($K_{r,1}$) is plotted on Figure 3.2-6 against the cyclic magnitude ratio ($\zeta_b$) for the test piles installed in medium dense and dense Fontainebleau sand.

![Figure 3.2-6. Average relative increase in reload stiffness from $K_{r,1}$ in DTU tests](image)

It is evident from Figure 3.2-6 that sand density and cyclic load ratio ($\zeta_c$) have little influence on the relative change in rotational stiffness and the dominant factor affecting this change is the cyclic magnitude ratio ($\zeta_b$). However, even for this ratio, when cyclic load levels are within expected design levels (i.e. $\zeta_b < 0.75$), the maximum observed
relative increase in stiffness is only 1.6. Similar observations were made by Verdure et al. (2003), Rosquøët et al. (2007) and Klinkvort & Hededal (2013). Zania (2014) shows that a 60% increase in head stiffness for a typical offshore monopile can lead to a 5 to 10% increase in the wind turbine eigen-frequency. For a wind turbine designed based on the soft-stiff approach, this increases the potential for resonance of the turbine with the blade passing frequency.

**Pile bending moments**

Bending moments at peak lateral load ($M_{\text{max}}$) at $N = 1$, 100 and 500 are shown on Figure 3.2-7a and 3.2-7b for two one-way tests in medium dense sand (DTU-C-1 and DTU-C-2). It is seen that, for DTU-C-1 with a typical design $\zeta_b$ value of 0.49, the maximum pile bending moment increased by about 30% after 500 cycles and the location of this maximum increased from a normalised depth ($z/D$) of 2 to 2.75. Maximum moments for one-way tests in the denser sand samples with the same $\zeta_b$ value (Test DTU-C-7) increased by only about 10% over the same number of cycles. These trends are consistent with the observed formation of a post-hole at the ground surface during cycling, with a larger post-hole forming in the less dense sand. Interestingly, for very high level cycling ($\zeta_b = 1.0$) in both medium dense and dense sand, there is little increase in bending moment (e.g. see profiles for DTU-C-2 on Figure 3.2-7b). As seen on Figure 3.2-7c, changes in maximum moment are also minimal for two-way cycling in test DTU-C-6, although the moment increases on the reverse loading side due to the negative cycling bias ($\zeta_c = -1.13$), with the point of maximum moment moving upwards from $z/D \sim 2.5$ to $z/D \sim 1.5$.

The minimum bending moments ($M_{\text{min}}$) plotted on Figure 3.2-7a, and Figure 3.2-7b correspond to residual bending moments for the two one-way tests, DTU-C-1 and DTU-C-2. These ‘locked-in’ moments are comparable in tests DTU-C-1 and DTU-C-2 but, in
test DTU- C-1, correspond to almost 50% of $M_{\text{max}}$ at $z/D = 3.5$ after application of 500 cycles. The existence of such significant ‘locked-in’ or residual moments, while also having been observed by Kirkwood & Haigh (2014), is often not recognised in typical $p$-$y$ analyses and reflects the changes due to cycling of the density and stress regime as well as the sand surface profile.

\textit{Pile deflections}

The pile deflections derived from integration of the bending moment profiles (\textit{Equation 3.2-5}) plotted on \textit{Figure 3.2-7} are at the peaks and troughs of the applied lateral cycling. The profiles are indicative of a rigid pile response with rotation occurring at a normalised depth ($z/D$) of 4.5 to 5 diameters, which is approximately 80% of the overall pile length. The UWA piles had a longer $L/D$ ratio of 11.4 and also displayed a semi-rigid response with rotation occurring at $z/D \sim 7$. The lateral displacements in the two-way cyclic test (DTU-C-6) after 300 cycles are the same as, or smaller than, those experienced at the peak of the first cycle.
Chapter 3. Lateral piles in sand

Figure 3.2-7. Typical profiles of bending moment, net pressure and pile deflection

(a) Test DTU-C-1 ($\xi_c = 0.05$, $\xi_b = 0.49$, $D_r = 64\%$)

(b) Test DTU-C-2 ($\xi_c = 0.03$, $\xi_b = 1.00$, $D_r = 58\%$)

(c) Test DTU-C-6 ($\xi_c = -1.13$, $\xi_b = 0.45$, $D_r = 50\%$)


**Net pressures on piles**

Net pressures derived by differentiation of the moment profiles (Equation 3.2-4) for tests DTU-C-1, DTU-C-2 and DTU-C-6 are also provided on Figure 3.2-7. The pressure distribution shows more clearly than the pile deflection that the depth of the point of rotation increases from a normalised depth of 4.5 at \( N=1 \) to 4.7 at \( N=500 \) (noting that the rotation point is the deepest level at which the net pressure is zero). With an increasing number of cycles, the calculated net pressures at shallow depths in test DTU-C-1 reduce while those at deeper levels increase; these trends are in line with the formation of the depression (or post-hole) at the rear side of the pile. Net pressures derived in Test DTU-C-2 at maximum lateral load and at the peaks of the cycles showed little change with cycling, as reflected by the stable profile of maximum moment evident on Figure 3.2-7b.

There is an increase in the magnitude of the maximum absolute net pressures in DTU-C-6 (due presumably to densification) and this leads to an upward movement of the level of pile rotation (where net pressures are zero) and a reduction in the depth to the location of maximum net pressure.

**Effects of cyclic load sequence**

Monopiles can be subjected to a wide range of cyclic loading events. To investigate the dependence of accumulated pile head rotations on cyclic history for uni-directional loading, the centrifuge experiments involved application of three packages of cycles applied in different sequences. The measured pile head rotations (\( \theta_N \)) in medium dense dry Fontainebleau sand for one-way cycling (\( \zeta \sim 0 \)) for three 500 cycle packages with different cyclic magnitude ratios (\( \zeta_b \sim 0.5, 0.75 \) and 1.0) are plotted on Figure 3.2-8a for three sequences. The final accumulated rotations after the total of 1500 one-way cycles are 1.25°, 1.65° and 1.9° and the difference between these rotations reflects the degree of dependence on the order of the load cycling. It is also evident that application of higher
levels of cycling after initial lower level cycling leads to greater rotations than if high level cycling precedes lower level cycling.

Figure 3.2-8. Pile rotation at peak load under various cycles in (a) cyclic load sequences with \( \zeta_c \approx 0 \) and (b) \( \zeta_b \approx 0.5 \)
Figure 3.2-8b presents a similar set of measurements for another set of cyclic packages each with 300 cycles and the same $\zeta_b$ value of approximately 0.5 but with different $\xi$ values. The dependence on the sequence of the cyclic packages is more evident for these cases. It is seen, in test DTU-C-4 for example, that there is a dramatic reduction in rotation in the final of the three packages when the value of $\chi$ becomes progressively more negative (i.e. the loading moves from one-way to two-way). This is a sharp contrast to the response seen in Test DTU-C-5 which had a positive $\chi$ in the last package and ended up having three times the rotation that was measured in test DTU-C-4.

**Post-cyclic response**

The variation of the normalised lateral load with pile head rotation measured during post-cyclic monotonic pushes performed in the centrifuge tests are compared on Figure 3.2-9 with the responses measured in monotonic tests. The plotted responses for the post-cyclic tests start from the permanent rotation induced by their cyclic histories summarised in Table 3.2-1. Although the vertical restraint at the top of the DTU piles may have had a small effect on the magnitude of the capacities recorded (as discussed previously), both the UWA and DTU tests indicate that the post-cycling capacity of piles which did not suffer a significant permanent rotation during cycling (i.e. typically less than $0.25^\circ$) is the same or greater than the monotonic capacity. Post-cycling capacities measured after piles experienced a rotation of $0.5^\circ$ tend to be lower than the monotonic capacities, although this finding is not of major consequence for wind turbine monopiles, which need to restrict total rotations to less than $0.5^\circ$. 

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Figure 3.2-9. Post-cyclic response in (a) UWA tests (Dr = 68%) and (b) DTU tests with Dr given in parentheses
3.2.4 Discussion

Accumulation of displacement with uniform cycling

It has been seen that Equation (3.2-3) provides a reasonable representation of the accumulated rotations measured in the DTU and UWA centrifuge tests. The accumulation coefficient in these tests was not sensitive to the cyclic magnitude ratio ($\zeta_b$) but varied with the sand density and the cyclic load ratio ($\zeta_c$). These test results are now compared with measurements reported in field tests by Li et al. (2015) and in similar centrifuge experiments reported by Klinkvort & Hededal (2013) and Rosquoët et al. (2007). Measured trends are examined using the $\alpha_y$ coefficient (Equation 3.2-3b) as the latter two of these investigations did not report pile head rotations. For the tests in this study, the value of $\alpha_r$ was typically equal to ($\alpha_r$ - 0.04). It should also be noted that the study of Rosquoët et al. (2007) indicated pile displacement profiles that were characteristic of a flexible pile response, rather than the rigid pile rotation mechanism observed in the remaining experiments. Relevant details for all experiments are provided in Table 3.2-2.

Close examination of the results of all tests indicated that $\alpha_y$ was largely independent of $\zeta_b$. Values of $\alpha_y$ recorded in pure one-way cyclic tests (with $\zeta_c$=0) are plotted against the initial sand relative density ($D_r$) on Figure 3.2-10a. $\alpha_y$ clearly reduces with increasing $D_r$ and also tends to be a little larger at low pile slenderness ratios ($L/D$). No systematic relationship between $\alpha_y$ and the eccentricity of applied load ($e/D$) was apparent. The following equation, which is plotted on Figure 3.2-10a, yields a near upperbound value for $\alpha_y$:

$$\alpha_y = 0.3 - 0.22D_r \quad \text{for } \zeta_c = 0, \ D_r > 0.5, \ L/D < 7 \quad (3.2-7)$$

All $\alpha_y$ values are plotted against $\zeta_c$ on Figure 3.2-10b, which uses Equation 3.2-7 to normalise $\alpha_y$ values for effects of relative density.
Figure 3.2-10. Relationship of (a) $\alpha_y$ with $D_r$ for $\zeta_c \sim 0$ and (b) $\alpha_y$ normalised for effects of $D_r$ for varying $\zeta_c$ ratios.
An upper bound curve to the dataset is shown on Figure 3.2-10b and leads to the following proposal for design of typical monopiles (for which $L/D < 7$):

$$\alpha_y = (0.3 - 0.22D_r)[1.2(1 - \zeta_c^2)(1 - 0.3\zeta_c)] \quad \text{for } D_r > 0.5 \quad (3.2-8)$$

Figure 3.2-10b shows that there is a progressive accumulation of permanent displacement when the cyclic load ratio $\zeta_c$ exceeds about -0.5. However, negative $\alpha_y$ values, and hence a reduction in accumulated displacements, are more typical when $\zeta_c$ is less than -0.5.

Assessment of the accumulation coefficient, $\alpha$

Equation (3.2-8) was derived for tests in fine, uniform silica sands, such as UWA, Fontainebleau and Blessington sands, and is applicable for assessment of cycling induced displacement for monopiles in these kinds of sands. It is of interest to examine if standard cyclic triaxial testing can be used to provide guidance to designers on the likely value of $\alpha$ in other sand types.

To examine this potential, monotonic and one-way cyclic triaxial tests were performed on Fontainebleau sand, as described above and with details summarised in Table 3.2-3. The measured variations of axial strain ($\varepsilon_a$) with the number of (one way) cycles ($N$) are plotted on Figure 3.2-11 using logarithmic axes. It is evident that $\varepsilon_a$ varies with $N$ in an analogous way to the variations of $\theta$ with $N$ presented on Figure 3.2-5. The slopes of these trend lines (on logarithmic axes), referred to as $\alpha_{\text{triax}}$, average at 0.14 for the medium dense sand with a single value of 0.09 measured in the dense sand and are plotted for comparative purposes on Figure 3.2-10a. While factors such as the mode of cycling and the sample stress level may lead to modest changes in $\alpha_{\text{triax}}$, it is evident that, as for $\alpha_r$ and $\alpha_y$, $\alpha_{\text{triax}}$ reduces with an increase in sample density and is virtually independent of the cyclic magnitude level ($q_{\text{max}}/q$).

The value of $\alpha$ cannot be expected to be the same as $\alpha_{\text{triax}}$, simply because cyclic loading of piles is a boundary value problem with changing boundaries, whereas a triaxial test
measures the response of an element. Nevertheless, the similarity of their controlling factors and their relatively close agreement in terms of magnitude (see Figure 3.2-10a) indicate that triaxial cyclic testing can provide insights for designers into expected lateral cyclic response in sands for which no previous experience exists.

![Graph showing triaxial tests in Fontainebleau sand](image)

**Figure 3.2-11.** Triaxial tests in Fontainebleau sand (a) comparison of monotonic and cyclic tests and (b) variation of axial strain with number of cycles
Chapter 3. Lateral piles in sand

Effects of loading sequence

As shown on Figure 3.2-8, the DTU centrifuge experiments incorporated application of a series of uni-directional cyclic load packages (each with a set number of cycles and given $\phi$ and $\zeta$ ratios). It was seen that the loading sequence did affect the magnitude of the final value of rotation accumulated although the effect was more significant when both one-way and two-way cyclic wave packages were applied. Simple ways to assess the likely amount of the maximum permanent rotation are presented in the following.

For the tests involving one-way load packages, best predictions are obtained using the superposition approach, described by Lin & Liao (1999). In application of this method, the accumulation coefficient is found from Equation (3.2-8) and $\theta_l$ can be obtained from a site specific monotonic test or from predictive methods such as Suryasentana & Lehane (2016) or API (2011). Predictions for the rotations at the peak of the cycles for specific cyclic packages applied (see Table 3.2-1) in four DTU tests are compared with the observations on Figure 3.2-12a to 3.2-12d. The value of $\alpha$ employed for these calculations was taken to be 0.04 less than $\alpha_r$ (calculated using Equation 3.2-8), in line with observations in these tests. Reasonable agreement is apparent although it is evident that the final predicted rotation can be in error by up to 30%. As expected, the approach leads to insignificant increases in rotation when the cyclic magnitude ratio ($\zeta$) is lower than in a preceding load package.

It has been seen that significant permanent rotations do not occur under two-way loading when the cyclic load ratio ($\zeta$) is less than about -0.5. There is not a simple superposition approach that can be used to predict the trends with cyclic histories involving negative $\zeta$ values. However, given that the main interest of the designer is the assessment of the maximum rotation under a given set of cyclic packages, it is proposed that predictions are obtained for the packages arranged in order of increasing accumulation coefficient (e.g.
as obtained from \textit{Equation 3.2-8}), and then applying the superposition method and assuming no rotation accumulation when $\zeta$ is less than -0.5. This approach is compared on \textit{Figures 3.2-12e to 3.2-12h} and seen to provide an upper bound to the rotation that can be expected for these cyclic packages.

It is of interest to note that a number of different one-way cyclic packages were applied in the triaxial tests on Fontainebleau sand, referred to above. The test results showed excellent agreement with the superposition approach, which suggests that the discrepancies observed in \textit{Figure 3.2-12a to 3.2-12c} (which plot results for the same cyclic packages applied in different sequences) reflect the changes developing adjacent to a cyclically loaded pile in sand; these changes include ongoing modifications to the size and geometry of a post-hole, alterations to the in-situ sand densities and increases in residual net stresses (which alter the applied cyclic load ratios at any given level in the soil).
Figure 3.2-12. Variations of measured and calculated rotation accumulation for DTU tests

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3.2.5 Conclusions

A new series of centrifuge tests involving lateral cycling of instrumented piles in sand has shown that:

(i) The rotation generated by lateral cycling of piles is proportional to the rotation experienced under monotonic loading at the same peak cyclic load and varies with the number of cycles raised to a power, referred to as the accumulation coefficient ($\alpha$).

(ii) The accumulation coefficient ($\alpha$) depends primarily on the cyclic load ratio ($\zeta_c$) and the sand relative density. Highest permanent rotations are developed by short piles in looser sands subjected to one-way loading or biased one-way loading ($\zeta_c > -0.5$).

(iii) The accumulation coefficient measured in one-way cyclic triaxial tests is less than that observed in cyclic lateral pile tests, but shows a similar dependence on relative density and cyclic load ratio.

(iv) Residual (locked-in) net lateral stresses develop during cycling and can reach values that are up to 50% of the maximum moments induced by the peak lateral load. The existence of these stresses are a reflection of the changes in boundary conditions around laterally cycled piles, which include the development of post-holes and changing sand densities.

(v) Post-cyclic capacity of monopiles is similar to the monotonic capacity if the permanent rotation experienced during cycling is less than the typical serviceability limit of 0.5°.

(vi) A combination of these centrifuge results with data from other similar investigations has enabled general design guidelines to be developed to assist assessment of the development of permanent pile displacements and rotations.
3.2.6 Acknowledgements

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Chapter 4.  Lateral piles in residual soils

4.1 CPT based P-y curves

There are no generally accepted guidelines for the design of laterally loaded piles in residual soil. Existing methods for sands, cemented sands, clays and weak rock do not match observed behaviour. This paper addresses the great shortage of case history data by focussing on the assessment of the performance of piles under lateral load in one partially saturated residual soil which was characterised using a range of laboratory and in-situ tests. The instrumented lateral pile tests showed that the lateral stiffness and strength vary directly with the CPT end resistance \(q_c\) and these observations are used to formulate CPT-based load transfer \((P-y)\) curves. This formulation is shown to lead to good estimates of the lateral response of piles tested in other residual soils with cone friction ratios in excess of 3%.

4.1.1 Background

Residual soil is the weathered material remaining in-situ after soluble parent rock constituents have been removed. The mechanical properties of residual soils are known to exhibit significant differences over relatively short distances due to variabilities in the mechanical and chemical weathering processes as well as the differences in properties of the parent rock and of saturation levels (Wesley, 1990). These characteristics complicate the selection of geotechnical parameters for foundation design. In addition, residual soil samples are often friable and difficult to sample and consequently the Cone Penetration Test (CPT) end resistance \(q_c\) or Standard Penetration Test (SPT) blowcount \(N_{SPT}\) are often the only information available to designers.

Although widespread around the world, geotechnical design recommendations for residual soils are particularly scarce (Wesley, 1990; Brown & Mayne, 2012). This scarcity became apparent during the design phase of a number of large infrastructure projects in
the residual soils in Northern Australia and provided the motivation for the study presented here. This study focuses on the assessment of the performance of piles under lateral loads in one, well characterised, partially saturated residual soil. The information assessed from a series of instrumented pile tests in this particular soil together with trends indicated in lateral load tests in other residual soils are then employed to examine the potential of predictive approaches based on the CPT data.

The standard method for predicting the lateral response of piles is the load transfer method which models the pile as a series of beam elements and the ground as a series of non-linear, non-interactive, lateral springs referred to as p-y or P-y curves, where p is the net lateral resistance per unit length of pile, P is the net lateral pressure and y is the lateral displacement at any given depth. API (2011) is currently the Industry standard for laterally loaded pile design and, while recommending P-y formulations for clay and sand, it does not provide any guidance for residual soils, many of which are not fully saturated and possess both cementation/bonding (c’) and frictional components (φ’) of strength. In the absence of any further information, three options adopted by designers for more fine grained residual soils are (i) to apply the API (2011) recommendations for a stiff clay to the residual soil, (ii) allow for the c’-φ’ nature of the material using a P-y formulation developed by Evans & Duncan (1982) and (iii) assume the soils behaves as a weak rock and use the formulation given in Reese & Van Impe (2011). The ultimate net pressures (P_u) for the stiff clay, a c’-φ’ soil and weak rock as proposed by API (2011), Evans and Duncan (1982) and Reese and Van Impe (2011) are calculated using Equations 4.1-1, 4.1-2 and 4.1-3 respectively (and are examined later in this paper):

\[
P_u(\text{stiff clay}) = \min \left[ s_{u,UU} \left( 3 + \frac{\sigma_v'}{s_{u,UU}} + 0.5 \frac{z}{D} \right), 9s_{u,UU} \right] \quad (4.1-1)
\]
where $s_{u,UU}$ is the undrained shear strength measured in unconsolidated undrained (UU) triaxial tests, $z$ is the depth below the ground surface, $D$ is the pile diameter and $\sigma'$ is the vertical effective stress.

$$P_u(c'-\phi'soil) = AP_{ult\phi} + P_{ultc} \quad (4.1-2)$$

where $P_{ult\phi}$ is the ultimate resistance calculated using the Reese et al. (1974) formulation for sand and $P_{ultc}$ is derived using the API (2011) soft clay formulation, substituting undrained strength ($s_u$) for $c'$. The contribution from the frictional component is reduced by an empirical factor $A$ which decreases with depth.

$$P_u(\text{weak rock}) = \alpha_{rf}q_{UCS} \left(1 + 1.4 \frac{z}{D}\right) \quad \text{for } 0 \leq \frac{z}{D} \leq 3 \quad (4.1-3a)$$

$$P_u(\text{weak rock}) = 5.2\alpha_{rf}q_{UCS} \quad \text{for } \frac{z}{D} > 3 \quad (4.1-3b)$$

where $q_{UCS}$ is the unconfined compressive strength and $\alpha_{rf}$ is a strength reduction factor.

For weak rocks that are difficult to sample, $q_{UCS}$ can be estimated assuming the Mohr-Coulomb equation as:

$$q_{UCS} = 2c'\tan\left(45 + \frac{\phi'}{2}\right) \quad (4.1-4)$$

A number of studies have examined the suitability of various $P$-$y$ formulations in residual soil. For example, Cho et al. (2007) compared $P_u$ values given by Equations 4.1-1 and (4.1-3) with $P_u$ values determined from lateral tests on instrumented 760mm bored piles installed at three different residual soil sites in North Carolina. The comparisons showed that the weak rock method over-predicted $P_u$ values but the stiff clay method underestimated $P_u$ and the lateral pile stiffness; $c'$ and $\phi'$ values were not available for the soils at these sites. Simpson and Brown (2003) also found that the stiff clay method under-predicted the stiffness of six piles installed in the Piedmont residual soils in Alabama and
furthermore noted that that the API (2011) sand method greatly over-estimated pile head deflections, presumably due to the presence of a $c'$ component of strength in the soil.

These comparisons highlight the need for improved methods for assessing lateral pile performance in residual soil and provided the impetus for the study presented in this paper. A detailed examination of soil characteristics and lateral load tests at a residual soil site at Bullsbrook, Western Australia, is first presented before investigating a CPT-based $P$-$y$ formulation that would have wider application to a range of residual soils.

4.1.2 Soil characterisation at Bullsbrook test site

The test site at Bullsbrook, which is 45 km North East of Perth in Western Australia, is located within the Chittering metamorphic belt, comprising interwoven gneiss and schist (Wilde, 2001). Seismic cone penetration tests (SCPTs) and undisturbed block sampling were carried out to assist characterization of the residual soil properties.

Laboratory tests

Trial pits excavated at the test site revealed two units of interest, namely Unit B1 which is red-brown in colour and generally extends from 0.5 m to 1.0 m depth and overlies Unit B2 which is yellow-brown with a golden metallic lustre. Cubic block samples with a side length of 0.2 m were retrieved from pits and transported in wooden boxes to the laboratory after being wrapped in numerous layers of cellophane. Thin sections of these samples revealed that both B1 and B2 are residual schists with a relic schistose structure which was more apparent in B2. X-ray diffraction tests indicated varying mineral compositions with B1 being predominantly composed of quartz with approximately 20% kaolin and 20% illite, whereas B2 was predominantly biotite with approximately 20% kaolin and 20% quartz.

Mean results from soil classification tests on disturbed bag samples of B1 and B2 retrieved from depths between 0.5 m and 2 m are provided in Table 4.1-1. The higher
plasticity indices (PIs) and linear shrinkage (LS) values of B1 are consistent with XRD observations of a greater quantity of clay minerals within its clay fraction. Both units classify as being mixed soils with roughly equal fines content and coarse fraction and the deeper unit (B2) has a higher degree of saturation ($S_r$) than that of B1.

**Table 4.1-1. Average soil properties of bag samples retrieved from between 0.5 m to 2 m**

<table>
<thead>
<tr>
<th>Unit</th>
<th>$\gamma_b$ (kN/m$^3$)</th>
<th>Clay (%)</th>
<th>Silt (%)</th>
<th>Sand (%)</th>
<th>Gravel (%)</th>
<th>$w$ (%)</th>
<th>$S_r$ (%)</th>
<th>LS (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PI (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>16.7</td>
<td>28</td>
<td>20</td>
<td>43</td>
<td>9</td>
<td>13</td>
<td>0.43</td>
<td>2.73</td>
<td>9.5</td>
<td>43</td>
<td>25</td>
</tr>
<tr>
<td>B2</td>
<td>18.3</td>
<td>20</td>
<td>22</td>
<td>49</td>
<td>9</td>
<td>16</td>
<td>0.63</td>
<td>2.75</td>
<td>4.3</td>
<td>43</td>
<td>31</td>
</tr>
</tbody>
</table>

Direct shear, oedometer, unconsolidated undrained triaxial (UU) and wetting and drying tests were conducted on block samples retrieved from depths of 0.6 m and 1.7 m. To investigate the effect of saturation on the soil behaviour, where possible, tests were carried out at the fully saturated condition (i.e. at $S_r = 1$) as well as at the natural in-situ water content. The measured average strength and consolidation properties are summarised in **Table 4.1-2**.

**Table 4.1-2. Average soil properties, strength and soil behaviour on undisturbed block samples**

<table>
<thead>
<tr>
<th>Unit</th>
<th>Depth (m)</th>
<th>Matric suction (kPa)</th>
<th>$\sigma_{fy}$ natural (kPa)</th>
<th>$\sigma_{fy}$ $S_r=1$ (kPa)</th>
<th>$\phi'$ (°)</th>
<th>$c'$ (kPa)</th>
<th>$c'$ natural $S_r=1$ (kPa)</th>
<th>$S_u_{UU}$ natural $S_r=1$ (kPa)</th>
<th>$S_u_{CU}$ natural $S_r=1$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>0.6</td>
<td>140</td>
<td>400</td>
<td>50</td>
<td>35</td>
<td>71</td>
<td>5</td>
<td>63</td>
<td>26</td>
</tr>
<tr>
<td>B2</td>
<td>1.7</td>
<td>55</td>
<td>110</td>
<td>50</td>
<td>35</td>
<td>5</td>
<td>5</td>
<td>83</td>
<td>17</td>
</tr>
</tbody>
</table>

Wetting and drying tests were performed using the oedometer type pressure plate device (Perez-Garcia et al., 2008) to obtain a relationship between the soil saturation level and matric suction i.e. the soil water characteristic curves (SWCC). The drying curves are plotted on **Figure 4.1-1** and indicate matric suction values of the order of 140 kPa for the B1 sample and 55 kPa for the wetter B2 sample. These SWCC curves show a more gradual increase in matric suction ($s$) with reduction in $S_r$ than indicated by the SWCC.
curves derived using the empirical relationships with index properties proposed by Fredlund & Xing (1994) for typical clays and sands. The measured wetting curves display the same trends as shown on Figure 4.1-1 but are offset to suction values about 20% lower than on the drying curves.

![Figure 4.1-1. Soil water characteristic curve (SWCC) on Bullsbrook residual soil](image)

Specimens for 1D compression and shear box tests were extruded from rings that were pushed into the block samples. The compressions observed in oedometer tests after specimens were loaded initially to the in-situ vertical total stress are plotted on Figure 4.1-2. Tests were performed for specimens at their in-situ $S_r$ value and at $S_r = 1$; full saturation was achieved by flooding the oedometer bath and leaving for a period of 24 hours prior to compression. Tests were also performed on normally consolidated reconstituted saturated specimens to derive the intrinsic compression lines (Burland, 1990). As is typical of structured soils (Burland, 1990; Leroueil & Vaughan, 1990), it is seen on Figure 4.1-2 that the curves for the intact soil are well above those of the reconstituted specimens (i.e. exist at a higher void ratio at a given stress level). The tests
indicated $c_v$ values for reconstituted material of less than 10 $m^2$/year whereas the saturated intact specimens, with their more open structure, gave $c_v$ values at in-situ stresses of approximately 150 ± 50 $m^2$/year. The permeability of the in-situ soil was estimated to be $1 \times 10^{-8}$ m/s i.e. at least 10 times more permeable than typical clays.

The compressive indices of saturated specimens (as well as those inferred using total stresses for the unsaturated specimens) are generally similar for both units at vertical effective stresses in excess of 300 kPa (= 0.22 ± 0.03). However it is clear that the yield stress, expressed as a total stress, $\sigma_{vy}$, depends significantly on the degree of saturation (and hence suction), varying from 400 kPa for Unit B1 with $S_r = 0.43$ to 110 kPa for Unit B2 with $S_r = 0.63$. The higher pre-yield stiffness of the partially saturated soils can be expected to lead to higher stiffness adjacent to lateral loaded piles in these soils.

Direct shear tests were carried out on unsaturated and saturated specimens and sheared at a slow rate of 0.012 mm/min with the aim of achieving fully drained conditions for the saturated specimens and constant suction conditions for the unsaturated specimens. Peak

![Figure 4.1-2. Oedometer tests on Bullsbrook residual soil](image-url)
shear stresses \( (\tau_p) \) developed in these tests are plotted on Figure 4.1-3 against the vertical consolidation stress, \( \sigma_{vc} \), which was applied for 24 hours before shearing.

Figure 4.1-3. Direct shear test results for Bullsbrook residual soil on samples (a) B1 at 0.6m depth, and (b) B2 at 1.7m depth
It is evident that the friction angle ($\phi'$) for fully saturated samples from both Units B1 and B2 is approximately 35°. Higher strengths are generally observed for the partially saturated intact specimens and this can be expected to be due, at least in part, to greater matric suction (see Figure 4.1-1). Russell & Khalili (2006) suggested that the peak shear strength for soils ($\tau_p$) with the levels of matric suction of the order of that shown on Figure 4.1-1 can be estimated from the following equation:

$$\tau_p = c' + [(\sigma - u_a) + \chi(u_a - u_w)] \tan \phi'$$

where $\chi = \left[\frac{(u_a - u_w)}{s_{ae}}\right]^{-0.55}$

where the air entry value, $s_{ae}$, can be obtained approximately as the intersection of tangents to the SWCC curve linking the fully saturated and the partially saturated state portion of the drying curve. Values of $s_{ae}$ of 70 kPa and 30 kPa were interpreted from Figure 4.1-1 for samples B1 and B2 respectively. The best fit strength envelopes to the partially saturated specimens derived using Equation 4.1-5 with these $s_{ae}$ values, the matric suction given on Figure 4.1-1 (assuming these did not change during shearing) and a friction angle of 35° are plotted on Figure 4.1-3. The envelopes provide a good match to observations by employing only a small $c'$ component of 5 kPa for both B1 and B2 specimens in Equation 4.1-5. It can therefore be concluded that the greater strength of B1 is due primarily to higher suction levels because of a lower saturation level and not because of a higher bond strength ($c'$) between soil particles.

Eight unconsolidated undrained (UU) triaxial tests were carried out on carved specimens from the B1 and B2 block samples. The mean UU undrained strengths ($s_{u,UU}$) recorded for B1 and B2 were 63 kPa and 83 kPa respectively, but both sets of measurements showed considerable variability with a standard deviation in $s_{u,UU}$ of about 20 kPa. The lower strength of B1 samples is surprising given its greater in-situ suction and may reflect losses in suction that took place during the specimen carving process (which required a period of exposure to air of about 2 hours). The low strength of some of the specimens
can also be explained by premature failure on (observed) steeply dipping planes of weakness.

**In-situ tests**

A total of 26 Cone Penetration Test (CPTs) were performed in a 300 m² area in the vicinity of the pile tests, with two of the tests including the seismic module to measure profiles of shear wave velocity. These tests highlighted variable rock head levels (refusal depths between 1.2 m and 3.4 m) and very significant natural variability of the overlying residual soils. Pore pressures recorded during cone insertion were erratic and often negative but did not reveal a water table. Positioning of the piles within the test area was finalized on inspection of the CPTs and pile nos. 1, 2, 3 and 4 were located within 0.5 m of the CPTs labelled P1, P2, P3 and P4 on Figure 4.1-4, which presents the cone resistance ($q_c$), friction ratio ($F_r$), pore pressure ($u_2$), and interpreted small strain shear modulus ($G_0$). The measured saturation ($S_r$) from laboratory tests are also presented in Figure 4.1-4 to compare with the in-situ tests.

The $q_c$ profiles show a hard desiccated sandy layer in the upper 0.5 m. This layer is underlain by the residual soils of interest to the pile tests described below, for which $q_c$ within Unit B1 is is 6.3 ± 2.7 MPa reducing to 5.6 ± 1.9 MPa in Unit B2. The $q_c$ values measured in closest proximity to the block samples tested (with the properties listed in Table 4.1-1 and 4.1-2) were 7.3 MPa and 5.0 MPa in the Units B1 and B2 respectively, indicating soil properties at the CPT locations vary a little from those of the tested samples.

The soil behaviour type chart of (Robertson, 2016) classifies the residual soil as a dense dilative sandy clay or clayey sand while the Schnaid, Lehane & Fahey (2004) chart, which relates structure to the $G_0/q_c$ ratio and effective stress level, indicates that structural/bonding effects are relatively small in Unit B2 when allowance is made in the
effective stress term for the suction present in this unit; no $G_0$ data are available for Unit B1.

*Figure 4.1-4. In-situ test data and measured $S_r$ values at Bullsbrook*
If cone penetration is assumed to be undrained and the soil was fully saturated, application of a typical cone factor ($N_k$) of 15 to the cone resistance of the B1 and B2 samples suggests respective undrained strengths of about 480 kPa and 330 kPa. These strengths are between 5 and 10 times larger than those measured in UU tests, indicating that $s_{u,UU}$ (measured in the manner described) is not a representative measure of the in-situ strength.

### 4.1.3 Field tests

**Test setup**

4 No. steel pipe piles, each with an outer diameter of 127 mm, wall thickness of 19 mm and length of 2 m were employed in the field tests at Bullsbrook. The tests were performed by jacking piles apart from each other, where P1 and P3 reacted against P2 and P4 respectively. An annotated photo of the test set up is presented in Figure 4.1-5 and details of the 4 test piles are provided in Table 4.1-3. Each pile was instrumented with 10 pairs of strain gauges in a half bridge connection down its length to enable derivation of bending moment profiles from the strain outputs. Associated calibration coefficients were derived in load testing performed in the laboratory prior to the field tests. The strain gauges were installed in a 2 mm deep trough machined on the outside of the pile and sealed with epoxy.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Pile 1</th>
<th>Pile 2</th>
<th>Pile 3</th>
<th>Pile 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>D (m)</td>
<td>0.127</td>
<td>0.127</td>
<td>0.127</td>
<td>0.127</td>
</tr>
<tr>
<td>t (m)</td>
<td>0.019</td>
<td>0.019</td>
<td>0.019</td>
<td>0.019</td>
</tr>
<tr>
<td>EI (kNm²)</td>
<td>1221</td>
<td>1221</td>
<td>1221</td>
<td>1221</td>
</tr>
<tr>
<td>L (m)</td>
<td>1.39</td>
<td>1.41</td>
<td>1.35</td>
<td>1.48</td>
</tr>
<tr>
<td>e (m)</td>
<td>0.26</td>
<td>0.26</td>
<td>0.21</td>
<td>0.25</td>
</tr>
<tr>
<td>L/D</td>
<td>10.9</td>
<td>11.1</td>
<td>10.6</td>
<td>11.6</td>
</tr>
<tr>
<td>e/D</td>
<td>2.05</td>
<td>2.05</td>
<td>1.65</td>
<td>1.97</td>
</tr>
<tr>
<td>Soil plug length (m)</td>
<td>0.86</td>
<td>0.55</td>
<td>0.28</td>
<td>0.31</td>
</tr>
<tr>
<td>$H_{alt}$ (kN)</td>
<td>50.0</td>
<td>n/a</td>
<td>51.5</td>
<td>51.5</td>
</tr>
</tbody>
</table>
The piles were jacked into the ground using a CPT truck as reaction at an average rate of 20 mm/min to embedment depths of between 1.85 m and 2 m. The average soil plug length formed during installation was 0.5 m, indicating that the majority of installation occurred in a fully-plugged mode. After installation, the top 0.5 m of soil was carefully excavated to the upper level of Unit B1 and testing commenced 7 days later.

Lateral loading was applied using a hydraulic jack placed between the piles in series with a compression load cell and steel cylinders extending to the piles (see Figure 4.1-5). Lateral pile head displacements were measured by attaching laser displacement transducers to reference beams with supports at least 1.5 m away from the piles. The weaker of each pair of piles was loaded to failure in a period of about 15 mins and the average displacement rate at the ground surface at the onset of failure was about 3 mm/min. Strain gauges remained within the elastic range throughout the tests and it was clear that pile failure occurred due to limiting passive soil resistance with the pile rotating.
in a semi-rigid mode about some point at depth. Soil heave and cracking were apparent immediately in front of each pile, and a gap to a depth of 0.2 m could be observed to form on the active side of the piles.

**Data processing**

The measured bending moments and pile displacements were used to calculate the ultimate net soil pressures $P_u$ and to derive $P-y$ curves for each pile test. The $P$ and $y$ values at any particular load level can be calculated from the standard Euler-Bernoulli beam equations:

$$p = P \times D = -\frac{d^2 M(z)}{dz^2} \quad (4.1-6)$$

$$y = \iiint M(z)/EI \, dz \quad (4.1-7)$$

where $D$ is the pile diameter, $z$ is the depth below soil surface, $M$ is the pile bending moment and $EI$ is the flexural rigidity of the pile. Various curve fitting methods were used to derive the $P-y$ curves, which were then assessed for accuracy by comparing the measured load displacement results with the responses predicted using these $P-y$ curves as input into a standard beam-spring load transfer program. The method proposed by Yang and Liang (2014) was found to give minimal errors to derive $P$ and involved differentiation of bending moment profiles using overlapping cubic polynomials. To determine the pile deflection profile, a 6th order polynomial was used to fit the bending moment data and integrated twice as indicated in *Equation 4.1-7*. Integration constants for pile deflection were solved using the displacement measured at the pile head and inferring zero displacement at the point of pile rotation, where the calculated net soil resistance is zero.
Test results

Typical lateral pile test results obtained are presented on Figure 4.1-6. It is clear from Figure 4.1-6a that, despite the site variability, the lateral capacity ($H_{ult}$) of three of the four piles (P1, P3 and P4) is approximately 50 kN while that of P2 is estimated to be about 55 kN. Figure 4.1-6b plots the bending moments at a lateral load ($H$) of 50 kN, indicating that the maximum moment occurs at a normalised depth ($z/D$) of 4 and has a magnitude of about 30 kNm; this moment is less than the yield moment of the pile sections (of 42 kNm) confirming that ‘short pile’ or geotechnical failure took place. The pile deflection profiles are shown on Figure 4.1-6c and 4.1-6d. It is seen that the pile rotates in a semi-rigid fashion with rotation occurring about a point at a depth of eight pile diameters (or 1 m below the base of the pit). Similar deflections occur for all four piles at $H = 30$ kN, but differences emerge at the larger load of 50 kN when failure is imminent.

It is of interest to compare the $H_{ult}$ value of 50 kN with the charts presented by Fleming et al. (2009) for the geotechnical lateral capacity of piles in clay and sand. The chart for clay indicates that to achieve this $H_{ult}$ value would require a mobilised undrained strength ($s_u$) for the deposit of approximately 110 kPa while that for sand would require a soil friction angle ($\phi'$) of about 56°; neither of these back-figured strengths is consistent with the soils data discussed previously.
Figure 4.1-6. Bullsbrook pile test results (a) load displacement at ground surface, (b) bending moment at 50kN, (c) pile deflection at 30kN, and (d) pile deflection at 50kN
**Ultimate pressures**

The ultimate lateral net pressures \( (P_u) \) recorded by Piles P1, P3 and P4 (i.e. piles that reached ultimate conditions) are plotted against normalised depth \((z/D)\) on Figure 4.1-7a and indicate maximum net pressures of about 1 MPa at \( z/D = 4 \). These \( P_u \) profiles are compared on the same figure with the net ultimate stresses predicted using Equation 4.1-1 to 4.1-3, using (i) measured \( s_{uu} \) values employed for the stiff clay model, (ii) apparent \( c' \) values for the \( c'-\phi' \) model and (iii) \( q_{ues} \) derived from Equation 4.1-4 with the apparent \( c' \) values for the weak rock formulation. It is clear that ultimate pressures estimated using the stiff clay and \( c'-\phi' \) approaches are significant under-estimates of the measurements. The weak rock formulation appears to provide reasonable predictions at shallow depths in Unit B1, but greatly underestimates \( P_u \) below \( z/D = 4 \) in Unit B2, due to the low unconfined compressive strength of this unit (as interpreted using Equation 4.1-4).

The range of \( P-y \) curves measured at Bullsbrook is presented in normalised form on Figure 4.1-7b up to \( z/D = 4 \), where maximum moments were recorded. All curves are seen to fall within a relatively narrow band and indicate that 80\% of ultimate net pressures \( (P_u) \) are generated at \( y/D = 0.03 \pm 0.01 \). These normalised curves are compared with the curves recommended for the three formulations considered on Figure 4.1-7a. The \( \varepsilon_{50} \) value (which is the axial strain at 50\% mobilised undrained strength in a UU test) employed for the stiff clay formulation was 0.007, as measured in the UU tests whereas the softer of the range of responses recommended for the weak rock model was employed. No additional assumptions were required for the \( c'-\phi' \) model.

It is seen on Figure 4.1-7b that the measured normalised \( P-y \) response differs quite significantly from the three sets of predictions. The weak rock method greatly over-estimates the normalised stiffness while the stiff clay model leads to a large under-prediction of stiffness. It is clear that none of the three methods provides reasonable predictions for the net ultimate stresses and normalised \( P-y \) response.
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Figure 4.1-7. Existing methods compared to field test measurement (a) net stress variation with normalised depth, (b) Normalised P-y response
4.1.4 Proposed new CPT $q_c$ formulation

Li, Igoe & Gavin (2014) and others, have shown that the CPT-based method of Suryasentana and Lehane (2014) provides reasonable predictions for $P$-$y$ curves in sand, while Truong and Lehane (2014) show that CPT-based $P$-$y$ curves for laterally loaded piles in clay are also promising. The rationale behind the link between the CPT $q_c$ for $P$-$y$ curves is the dependence of $q_c$ on the lateral stiffness of the surrounding soil mass in addition to the shear strength of the ground. For partially saturated soil, such as at Bullsbrook, it is also anticipated that, as shown numerically by Russell and Khalili (2006), the CPT $q_c$ value reflects strength and stiffness values for the in-situ level of matric suction. Therefore, given the potential for a CPT method and in the light of the poor agreement between measurements and various existing methods seen on Figure 4.1-7, a CPT-based $P$-$y$ approach is now formulated and subsequently compared with lateral pile response in other residual soils.

*Figure 4.1-8* presents the maximum recorded net pressures ($P_{\text{max}}$) normalised by $q_c$ for each of the test piles loaded to failure (P1, P3 and P4); the $q_c$ values employed were those measured using the CPT closest to the respective piles. The same trend with depth is observed for P1 and P3 but lower $P_{\text{max}}/q_c$ ratios are recorded at shallower depths for P4, which is possibly due to site variability and lower $q_c$ resistances at the pile location than measured by the closest CPT.

The increase of $P_{\text{max}}/q_c$ with normalised depth to a relatively constant value is similar to that shown for piles in clay where wedge type failure at shallow depths transitions to a flow-around type failure at depth. It is noted that $P_{\text{max}}$ values are less than ultimate net stresses ($P_u$) in the vicinity of the depth of pile rotation ($\sim z/D = 8$), where pile movements are small.
Based on the trends indicated on Figure 4.1-8, the following equation is proposed for $P_u$:

$$P_u = N_{pq} q_c \quad \text{where} \quad N_{pq} = 0.03 + 0.25 \tanh \left[ \frac{0.5 z}{D} \right]$$

(4.1-8a)

A degree of uncertainty exists relating to the value of $N_{pq}$ at shallow depth due to the effects of the 0.5 m excavation of the pit on the $q_c$ values (where were recorded prior to excavation). The best-fit curve to the normalised $P$-$y$ curves is shown on Figure 4.1-7b and is given as:

$$\frac{P}{P_u} = \tanh \left[ 6 \left( \frac{y}{D} \right)^{0.5} \right]$$

(4.1-8b)

$P$-$y$ springs derived directly from Equation 4.1-8a and 4.1-8b were input to a load-transfer program (Oasy ALP, 2016) to predict the lateral pile response at Bullsbrook. Examples are shown on Figure 4.1-9 which compares the calculated pile head load-displacement
response, pile deflected shapes and bending moment profiles with measurements obtained for piles P1 and P2. These confirm the suitability of the formulations to provide reasonable estimates of the pile head response but also the distribution of lateral displacements and bending moments with depth.

![Figure 4.1-9. Pile test predictions for Bullsbrook site using Equation 4.1-8](image)

Comparison of Equation 4.1-8 with other case histories

The ability of Equation 4.1-8 to provide reasonable predictions of lateral pile responses measured in three other residual soils is now investigated. The field tests were conducted in: (i) residual soil in the Piedmont plain in Alabama, reported by Simpson and Brown (2003), (ii) basalt residual soil in Rio Grande do Sul, southern Brazil, presented by Consoli et al. (2016) and (iii) granite residual soil in Iksan, South Korea, described by Choi et al. (2013). CPT $q_c$ data were available at all sites, and the soil properties and pile test configurations at each are summarised in Table 4.1-4.
Both the Piedmont and Bullsbrook residual soils are derived from a mica-schist and have a similar fines content. The Rio Grande do Sul residual soil has a very high clay fraction while the Iksan material is a coarse grained deposit with a fines content of only 12%. The measured cone friction ratios of the deposits, as summarised in Table 4.1-4, are in line with these classification details. The classification system based on $G_0/q_c$ ratios for residual soils, as described by Schnaid et al. (2004), indicates that the Bullsbrook, Piedmont and Rio Grande do Sul residual soils are likely to have some microstructure/bonding whereas $G_0/q_c$ ratios for the Iksan granite residual soil show little evidence of structure.
The lateral pile responses were calculated using the Oasys ALP program with Equation 4.1-8 used to derive $P$-$y$ curves from the CPT $q_c$ data at each test location. Allowance was made for the reduction in the flexural rigidity of the concrete piles as loading progressed. This could be done for the tests in Piedmont soil using the moment-curvature relationship for the piles provided by Simpson and Brown (2003) while this relationship was derived from first principles using the quoted steel and concrete details for the piles in Rio Grande do Sul. The piles in Iksan were reported as being full rigid.

The measured and calculated pile head load displacement curves for the three case histories are compared with the measurements on Figure 4.1-10, Figure 4.1-11 and Figure 4.1-12. It is seen that:

(i) The calculated response in the Piedmont soil is slightly softer than observed while deflection profiles are relatively well predicted. The degree of saturation of the Piedmont soil is close to unity, suggesting that Equation 4.1-8 may also be applicable for saturated residual soils.

(ii) Calculations and measurements for both the 400 mm diameter bored piles and 100 mm steel pipe piles in the fine grained, partially saturated basalt residual soil are in good agreement. The calculations also evidently predict the depth to pile fixity with reasonable accuracy.

(iii) Both the stiffness and capacity of the (primarily granular) granite residual soil are over-estimated. The capacity of the ‘Case 3’ pile is not presented on Figure 4.1-12 but this was also over-predicted by the same degree as Case 1. Case 2 is very poorly predicted, but this was also noted by Choi et al. (2013) to be inconsistent with the other tests.

These comparisons indicate that Equation 4.1-8 can be used with some confidence for laterally loaded pile predictions in fine grained saturated and partly saturated residual
soils. The equations are not, however, suitable for application in coarse grained soils with low cone friction ratios such as the South Korean granitic residual soil; this material differs significantly from the Bullsbrook soil used to develop *Equation 4.1-8*.

*Figure 4.1-10. Predictions and measurements for pile tests in Piedmont residual soil* (Simpson and Brown, 2003)
Figure 4.1-11. Predictions and measurements for lateral pile tests in basalt residual soil (Consoli et al. 2016)
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4.1.5 Conclusions

A series of instrumented lateral pile tests in a partially saturated residual schist soil, coupled with a parallel series of characterisation tests on the same material, has clearly demonstrated the inadequacy of existing approaches for estimating lateral pile response in these soil types. The residual soil examined indicated relatively low levels of cementation/bonding and a considerable influence of variations of matric suction due to variable degrees of saturation. A CPT based formulation for $P$-$y$ load transfer curves was developed specific to this soil using the measured $P$-$y$ responses with the implicit assumption that lateral stiffness and ultimate net stresses vary directly with the CPT $q_c$ value. This formulation is shown to lead to good estimates of the lateral response of piles tested in other residual soils with cone friction ratios in excess of 3%.

Figure 4.1-12. Predictions and measurements for lateral pile tests in granite residual soil (Choi et al. 2013)
4.1.6 Acknowledgements

The authors gratefully acknowledge the funding provided by Arup consulting engineers for the research presented in this paper. Thanks are also extended to Niels Kofler, who allowed use of his property for the field test, the assisting technicians Brad Rose, James Waters and Khin Seint and the UWA researchers Marcel Chee, Jessica de Paula and Le Viet Doan.
Chapter 5. Concluding remarks

5.1 Research outcomes

An extensive range of lateral pile tests in the centrifuge and some in the field have been carried out with corresponding laboratory tests, in-situ tests, and numerical analyses, to provide new insights for lateral pile behaviour in soft clays, sands and residual soils. The research was motivated by the demonstrated need for updating of recommendations for $P_y$ load transfer curves for piles in soft clays and residual soils, and the need to provide designers with additional insights into the accumulation of pile head rotation under the action of cyclic lateral loads.

The major findings of this thesis corresponding to the research aims discussed in Chapter 1.2 is summarised in the following.

(i) Interpreted ultimate net pressures ($P_u$) from drum centrifuge tests have indicated the importance of identifying which undrained shear strength ($s_u$) measurement should be used when adopting empirically derived methods to predict ultimate net pressures ($P_u$). Figure 5.1-1 presents the lateral capacity factor ($N_p = P_u/s_u$) with $P_u$ normalised by $s_{u,Tbar}$ from the T-bar and $s_{u,CU}$ derived from consolidated triaxial compression tests. It is evident that $N_{p,Tbar}$ is greater than $N_{p,CU}$ since $s_{u,Tbar}$ is typically 0.85$s_{u,CU}$. The current API (2011) recommendation for $N_p$ is also presented in Figure 5.1-1 and it is observed that using $s_{u,Tbar}$ with API (2011) will under estimate $P_u$ whilst using $s_{u,CU}$ will provide a reasonable estimate for $P_u$. However, the API (2011) recommendation is based on methods which adopt the unconsolidated triaxial compression strengths ($s_{u,UU}$) which is typically 0.7$s_{u,CU}$. This suggest the API (2011) method, when used correctly with $s_{u,UU}$, is likely to under predict the lateral capacity of the pile.
(ii) The results from a series of centrifuge lateral pile tests in kaolin supported by finite element analyses clearly illustrate the importance of pile shape on the lateral load transfer \((P-y)\) curves. Higher net pressures can develop for square and H-piles compared to circular piles (refer to Figure 5.1-1) and this arises because of the larger volume of clay per unit pile width involved in the failure mechanisms surrounding the square and H-pile sections. The mechanisms for the circle and square pile are shown in Figure 5.1-2 from 3D finite element analyses. A newly proposed \(P-y\) curve \((Equation \ 2.2-8\) with consideration for \(s_u\), shape and soil depth was derived in Chapter 2.2 and reproduced below.

\[
P = 10.5s_u Cu \tanh \left[5.45 \left(\frac{Y}{B}\right)^{0.52}\right]\left[1 - 0.75e^{-0.6z/B}\right]S_p \tag{2.2-8}
\]

Where \(S_p\) is a shape correction factor = 1.0 for circle, 1.25 for square and 1.35 for H pile.
(iii) Centrifuge tests in soft clay revealed lateral pile capacities developed after one-way cycling are closely comparable to monotonic capacities and relatively unaffected by high level cycling. *Figure 5.1-3* shows unchanged monotonic and post-cyclic $P_u$ values and also indicates the conservative nature of $P_u$ predictions using API (2011) recommendations. It is also concluded from these tests that SLS failures for monopiles in soft clay are likely to control design as only 10 one-way cycles with a cyclic load ratio ($H_{max}/H_{1°}$) of 0.55 was sufficient to induce SLS failure (pile rotation $> 0.5°$).
Figure 5.1-3. $P_r$, comparison from monotonic test, post cyclic tests, and API (2011) prediction in soft clay.

(iv) Centrifuge tests for short rigid piles in sand showed pile rotation ($\theta_N = \theta_1 N^{\alpha_r}$) and pile displacement ($y_N = y_1 N^{\alpha_y}$) varied with the number of cycles ($N$) raised to a power, referred to as the accumulation coefficient ($\alpha_r$ for rotation accumulation; $\alpha_y$ for displacement accumulation). The value $\alpha$ was found to depend primarily on the cyclic load ratio ($\zeta_c$) and the sand relative density (see Figure 5.1-4). Highest permanent rotations/displacements were developed by short piles in looser sands subjected to one-way loading or biased one-way loading ($\zeta_c > -0.5$). The centrifuge test results plus additional results from the literature was collated to propose Equation 3.2-9 for estimating pile displacement accumulation.

$$\alpha_y = (0.3 - 0.22 D_r) [1.2(1 - \zeta_c^2)(1 - 0.3\zeta_c)] \quad \text{for } D_r > 0.5 \quad (3.2-9)$$
The accumulation of axial strains in one-way cyclic triaxial tests on Fontainebleau sand bear a close similarity to the development of permanent rotation of piles subjected to one-way lateral cycling in the same sand ($\alpha_{\text{triax}} \sim 0.14$ for medium dense sand and 0.09 for dense sand). It is concluded that one-way cyclic triaxial tests performed on any particular sand deposit can be used to obtain insights into the likely performance of laterally cycled monopiles in that sand.

**Figure 5.1-4. Pile rotation accumulation coefficient in sand.**

(v) The accumulation of pile head rotation in sand was observed to be dependent on cyclic loading sequence in the centrifuge tests. Sequences of one-way cyclic packages arranged in increasing amplitude resulted in greater rotations and sequences with two-way cyclic packages resulted in very different final rotations. Whilst the method of superposition by Lin & Liao (1999), inspired by Miner’s rule (Miner, 1945), provides a reasonable prediction for one-way cyclic packages, it has been shown not to be applicable for two-way packages. However,
permanent rotations developed under two-way cyclic loading are insignificant compared to one-way loading and an upper bound approximation method provided in Chapter 3.2 can be used for design.

(vi) Residual soils are difficult to characterise because of inherent variability arising from spatial variations in composition and in the degree of weathering, alteration, bonding and partial saturation. Characterisation of two block samples of Bullsbrook residual soil revealed the relative importance of \( c' \) and matric suction. Such characterisation studies are not usually possible for real projects and it is concluded that design methods need to be based on in-situ test data.

Existing methods for stiff clays, weak rock and \( c'-\phi' \) soils were found to underestimate \( P_u \) for the partially saturated residual schist soil at Bullsbrook and led to the derivation of a new CPT \( q_c \) based \( P_u \) prediction as presented in Figure 5.1-5 and formula reproduced below \((Equation 4.1-8)\). This formulation assumed implicitly that that lateral stiffness and ultimate net stresses vary directly with the CPT \( q_c \) value. This formulation was shown to lead to good estimates of the lateral response of piles tested in other residual soils with cone friction ratios in excess of 3%.

\[
P_u = N_{pq}q_c \quad \text{where} \quad N_{pq} = 0.03 + 0.25 \tanh\left[0.5 \frac{z}{D}\right] \tag{4.1-8a}
\]

\[
\frac{P}{P_u} = \tanh\left[6 \left(\frac{y}{D}\right)^{0.5}\right] \tag{4.1-8b}
\]
Figure 5.1-5. $P_u$ prediction using existing methods (left) and new CPT $q_c$ formula (right)
5.2 Further studies

5.2.1 P-y curves

The study of different shaped piles presented in this thesis may prompt further studies examining various piles shapes to gain increased efficiencies. Further comparative studies examining modifications to pile shapes, multidirectional capacity, constructability and relative costs may lead to adoption of a new pile shape/material.

New CPT based $P$-$y$ curves for residual soils presented in this study were shown capable of predicting existing case histories fairly well in the partially saturated and near fully saturated conditions. For completeness, future work on this topic should consider carrying out lateral pile tests at the same site when the site is fully saturated and when it is partially saturated. As the CPT and lateral pile mechanism is similar to that of an expanding cavity, theoretical solutions of existing cavity expansion methods in unsaturated soils may assist in understanding the lateral pile behaviour. As with other saturated soils, the effects of loading rate should also be investigated.

5.2.2 Cyclic loading

As for most other investigations, this thesis assumed unidirectional constant amplitude cyclic loading. Further research is required to examine the impact of multidirectional loading to supplement the small amount of research conducted in this area. A challenging issue with multidirectional loading is the interpretation of the bending moments from the strain gauge instrumentation. Unidirectional loading with varying packages of constant amplitude cyclic loading is easier to process and has been useful for understanding the evolving soil conditions.

There now exists a substantial amount of data on cyclically loaded piles in sand which can be used to: (i) investigate the potential of a macro model based on soil element tests; (ii) investigate cyclic $P$-$y$ behaviour to provide insight on the mechanisms and energy
dissipation occurring in the system; (iii) interpretations of $P_u$ and $P_y$ curves at post-cyclic loading and (iv) comparison of the centrifuge tests with 3D finite element analysis using cyclic soil models.

There is still much work to do on the cyclic lateral loading of piles in clay which will take great effort as clay preparation requires time and cannot be easily fixed if there are issues with the sample. Nevertheless, industry and academia will benefit from centrifuge models of piles in clay.
Reference list


API (2011) *API Recommended Practice 2GEO, ISO 19901-4 2003 (Modified)*.


Chapter 5. Concluding remarks


Chapter 5. Concluding remarks


Chapter 5. Concluding remarks


Wilde, S.A. (2001) *Jimperding and Chittering Metamorphic Melts, Southwestern Yilgarn Craton, Western Australia - a field guide (Record 2001/12).*


Appendix A – UWA drum centrifuge

Figure A1. Sand sample prepared in channel after surface preparation

Figure A2. Drum centrifuge with actuator in the centre
Appendix A – UWA drum centrifuge

Figure A3. Wet pluviation to create strong box sand samples

Figure A4. Push tube samples to measure density of sand after pile tests

Figure A5. Pile showing 8 pairs of strain gauges connected in half bridge to 8 outputs
Figure A6. Load arm calibration in cantilever mode

Figure A7. Pile jacked into sample at 1g via installation tool for verticality
Appendix B – DTU beam centrifuge

Figure B1. Beam centrifuge with strong box sample and loading tower mounted on the swing

Figure B2. Sand preparation in strong box
Figure B3. Model pile

Figure B4. Cone penetrometer 10mm in diameter

Figure B5. Loading tower set up
Appendix B – DTU beam centrifuge

Figure B6. Laser set at 20mm and 70mm from the sand surface
Appendix C – Bullsbrook field tests

Figure C1. Fitting strain gauges onto the pile prior to epoxy coating

Figure C2. Pile installation
Figure C3. Gap formed behind the pile and cracks formed in front of the pile

Figure C4. Block sampling
Appendix D – Notes for future experiments

After carrying out and interpreting data for over 20 lateral pile tests, it is appropriate to mention here a number of important factors that have been identified to be useful for designing future experiments.

Pile strain gauge locations

The spacing of strain gauges are important as the bending moment profile is required to determine pile deflection and net soil pressures. Future experimental studies on layered stratigraphy may also find a need to target strain gauge spacing to record useful bending moment profiles for interpretation. Carrying out preliminary analyses either using existing $P$-$y$ curves in a beam-spring model or 3DFE analyses will identify possible points of inflection where strain gauges are required. Note that although the bending moment is small at the pile toe, for semi-rigid and rigid pile behaviour the change in bending moment near the toe is required to identify whether there exists significant base shear.

Centrifuge equipment

Model piles with lateral loading requires a lot of wiring to connect to the strain gauges and therefore the wiring can be very bulking and awkward to stabilise in the centrifuge. Any bulky connections should be connected to the tool table ensuring there is enough wire length for lateral pile movement.

Stiffness of the loading apparatus is also important as this will affect load controlled and displacement controlled loading. If displacements are also measured from the loading apparatus then any flexibility in the system will need to be accounted for when interpreting the data. To mitigate this issues, the loading apparatus should be designed with stiff members.
Temperature effects

Temperature fluctuations have been known and were observed in this study to affect the readings from the data acquisition box. The strain gauges on the equipment and the data acquisition equipment are susceptible to large changes in temperature, e.g. taking the equipment from the cold lab out to the field in the sun, or submerging a CPT into the cold centrifuge water. Time must be allowed for the equipment temperatures to regulate before setting the measurements to zero before a test and for field testing it is advised to keep all data acquisition equipment in the shade. If possible, equipment calibrations should be conducted under similar temperatures as the tests.

Sampling of residual soils

Residual soil samples are very difficult to attain due to its friable nature. Carving block samples to obtain undisturbed samples is seen as appropriate compared to rotary drill which may induce vibrations to the sample and compared to push tube samples where the sampler may refuse in gravelly or hard layers. Cutting small blocks out of the large block sample then pushing short cutters through to obtain small samples for direct shear and oedometer tests is a simple process but carving large triaxial specimens may take up to 2 hours which can cause drying and disturbance, leading to a loss of matric suctions. In this case, Shelby tube samples pushed in with a CPT truck are more suited for collecting triaxial specimens in this material. Using a CPT truck is advantageous as the CPT can be conducted first to select locations without hard layers. To obtain samples at deeper depths, small narrow pits can be dug so the CPT truck can drive along and collect samples. This method is possibly limited to about 3m for a typical backhoe excavator which should be sufficient depth to characterise the soil for a lateral pile, noting pile resistance in the top 3 diameters is most important.
Appendix E – MATLAB applications

All the test piles used for this research were instrumented with 8 to 15 strain gauges with data collected continuously during loading. There was a total number of 36 successful tests carried out which amounts to an abundance of data (8 tests in soft clay in the drum, 14 tests in sand in the drum, 10 tests in the beam, and 4 field tests). At each load increment, a curve fit was adopted for the bending moment profile to obtain a function with depth so that deflection, shear force and net soil resistance with depth can be calculated. The deflection and net soil resistance profiles are used together to form $P-y$ curves and these derived $P-y$ curves are validated in the load-transfer program Oasys ALP. If the load displacement response calculated using the derived $P-y$ curves match those from the centrifuge tests then the curve fitting was successful. This process was repeated a large number of times for the cyclic loads. Applications were created in MATLAB during this research to automate the process which is able to handle the large data more efficiently than Microsoft Excel. The apps created are discussed in this Appendix and is currently being improved for robustness and further interpretation of the data.
**SPile.m**

This App is for the interpretation of statically loaded instrumented piles. The variables for the pile test should be already saved in a `.mat` workspace and can be called with the App in the Command Window.

```matlab
SPile('workspace.mat')
```

The App has a self-explanatory user interface that requires input of names of the variables and numerical parameters such as dimensions, pile properties, and values for curve fitting methods such as order of polynomial, depth increments and break points. There are also tick boxes to select or deselect calculation options.

The App allows comparing visually the different curve fitting methods and checking of $M$, $y$, $SF$, $P$ and $P$-$y$ curves. The curve fitting methods that have been coded in the App are:

(i) Natural Cubic Spline

(ii) Yang and Liang (2004) which is the overlapping of cubic polynomials over 5 data points. In this App, a single high order polynomial is used to fit the final derived $P$ profile to enable interpolation between data points.

(iii) High order polynomial

(iv) Piece-wise polynomial by selecting the polynomial degree and depth of break points for the polynomial.

Once the preferred method is selected, the interpreted data for that method can be automatically output into a Microsoft Excel file for further processing. Example screenshots of the App are presented below.
### Appendix E – MATLAB Applications

#### Pile Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>D (m)</td>
<td>0.04</td>
</tr>
<tr>
<td>L (m)</td>
<td>0.24</td>
</tr>
<tr>
<td>e (m)</td>
<td>0.12</td>
</tr>
<tr>
<td>EI (kNm²)</td>
<td>5.738423324</td>
</tr>
</tbody>
</table>

#### LVDT Locations and Select for y Calculation

<table>
<thead>
<tr>
<th>Location</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>e₁ (m)</td>
<td>0.07</td>
</tr>
<tr>
<td>e₂ (m)</td>
<td>0.02</td>
</tr>
</tbody>
</table>

#### Workspace Data Name

<table>
<thead>
<tr>
<th>Name</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>z (m)</td>
<td>Depth</td>
</tr>
<tr>
<td>H (kN)</td>
<td>H</td>
</tr>
<tr>
<td>BM (kN/m)</td>
<td>BM</td>
</tr>
<tr>
<td>SF (kN)</td>
<td>name</td>
</tr>
<tr>
<td>y (m)</td>
<td>name</td>
</tr>
<tr>
<td>y₁ (m)</td>
<td>ytop</td>
</tr>
</tbody>
</table>

#### Filter Large Data

- No. for calc: 20

#### Calculation Inputs

- Discretise depth for calculation:
  - Increment (m): 0.01
- Polynomial for displacement calculation:
  - Order: 5
- Rotation depth range:
  - Top (m): 0.18
  - Bottom (m): 0.23
- z/D increment for calculation:
  - z/D increment: 0.5
- Curve fit twice (new fit for SF):
  - BM poly order: 5
  - SF poly order: 5
- High order polynomial:
  - BM poly order: 5
  - SF poly order: 5
- Piecewise polynomial:
  - BM | SF
  - 0.02 | 0.02
  - 0.07 | -0.13
  - 0.15 | -0.21
  - 0.24 | -0.24
  - brk | brk

---

A13
Appendix E – MATLAB applications
CPile.m

The calculation methods in CPile.m for analysing cyclic data are the same as SPile.m and the App has extra plots to compare measurements at the peaks and troughs of each cycle as well as comparing data for individual cycles. It is recommended that a suitable curve fitting method is first selected for the corresponding static load test using SPile.m and P-y curves validated using a load transfer program.
SortCyclicData.m

For load controlled cycling, the test pile would at times become out of sync with the predefined loading waveform and therefore numbering of the data is best carried out on the measured data after testing. The SortCyclicData.m assigns a cycle number to the measured data, identifies whether the piles is reloading or unloading, and prepares the data for CPile.m.
Appendix F – Data Inventory

This data is stored at the School of Civil, Environmental and Mining Engineering at UWA for future analyses.
Chapter 2 - Lateral piles in clay

All relevant data is in the folder `\Chapter2_Clay`

\Chapter 2.1
Contains interpreted data in the spreadsheet `LP_CPT14.xlsx`

\Chapter 2.2
Contains interpreted data in the spreadsheet `LP_SoftClay.xlsx`

\Chapter 2.3
Contains interpreted data in the spreadsheet `LP_OMAE.xlsx`

\Drum\Calibration
Calibrations conducted for the loading arm, CHS pile, SHS pile and HS pile are in the excel file `Drum_Calibrations.xlsx`. Raw data in .txt format for the calibrations are also located here.

\Drum\Tbar
Raw data in .txt format for all the T-bars conducted are located here. The excel file `All_Tbars.xlsx` has all the T-bar profiles and corrected for point of contact to the clay surface and for shallow penetration effects.

\Drum\ALP
Oasys ALP models to assist in creating Figure 2.2-7

\MATLAB
Using the program SPile.m discussed in Appendix E, the interpreted P-y curves are located here as MATLAB .mat data files. The interpreted P-y curves and pile deflection data, as well as results of the P-y curve verifications are extracted into excel files in this folder. The P-y curve verifications were carried out using Oasys ALP and these corresponding ALP files are also saved here.

\PLX3D
Each Plaxis model has a .P3D file with a corresponding folder .P3DAT which holds all the data of the model. Both files must always be in the same directory or the .P3D file will not open. Due to the constant updating of Plaxis, these files can no longer be opened in the latest version but the results can still be viewed in Plaxis Output. Only the final versions of the models which have passed mesh validation as discussed in Appendix D are included.

- Models relevant to Chapter 2.1 (created in PLAXIS 3D2013)
  - OD880_NC.P3D (referred to as FE1 in text)
  - OD880_OC_R1.P3D (referred to as FE2 in text)
  - OD880_NC_e50.P3D (referred to as FE3 in text)
  - OD880_NC_Ir200.P3D (referred to as FE4 in text)
  - OD440_NC_R1.P3D (referred to as FE5 in text)
  - OD3000_NC.P3D (referred to as FE6 in text)
Appendix F – Data inventory

- Models relevant to Chapter 2.2 (created in PLAXIS 3DAE)
  - CHS.P3D (circular pile)
  - SHS.P3D (square pile)
  - HS.P3D (H section pile)
  - SHS_90.P3D (square pile rotated 90 degrees pile)

- Mesh validation checks are in the excel file Mesh_validations.xlsx
Chapter 3 – Lateral piles in sand

All related data is in the folder :\Chapter3_Sand

\Chapter 3.1
Contains interpreted data in the spreadsheet LP ISFOG.xlsx

\Chapter 3.2
Contains interpreted data in the spreadsheet LP_sand.xlsx

\Photos
Photos of the soil preparation, testing equipment and test setup.

\Calibrations
Calibrations conducted for the loading arm, CHS pile, SHS pile and HS pile are in the excel file Drum_Calibrations.xlsx. Calibrations for the DTU piles is in the excel file DTU_Calibrations.xlsx. Raw data in .txt format for the calibrations are also located here.

\Raw_data
Raw data from the lateral pile tests and CPTs.

\Lab_tests
Laboratory test certificates, reports, and excel data files.

\MATLAB
Using the program SPile.m discussed in Appendix E, the interpreted P-y curves are located here as .mat data files. The interpreted P-y curves and pile deflection data, as well as results of the P-y curve verifications are extracted into excel files in this folder. The P-y curve verifications were carried out using Oasys ALP and these corresponding ALP files are also saved here.
Chapter 4 – Lateral piles in residual soils

All related data is in the folder :\Chapter4_RS

\Photos
Photos of the pile preparations, equipment, installation, pile tests, sampling, and laboratory testing.

\Pile_Properties
Steel properties, pile dimension, pile design, and pile calibrations.

\CPT
CPT and SCPT raw data, test locations, and extracted CPT/SCPT data into excel.

\Lab_tests
Laboratory test certificates, reports, and excel data files

\Pile_Test
- Raw data in .txt format from tests are in the folder \Raw_data
- Interpreted data from tests are in the excel file BB_Pile_tests.xlsx

\MATLAB
Using the program SPile.m discussed in Appendix E, the interpreted P-y curves of the Bullsbrook field tests are located here as .mat data files. The interpreted P-y curves and pile deflection data, as well as results of the P-y curve verifications are extracted into excel files in this folder. The P-y curve verifications were carried out using Oasys ALP and these corresponding ALP files are also saved here.

\Calculations
- The excel file Calc – Residual Soil.xlsx contains all the calculations and interpretations of the laboratory tests, relevant CPTs, comparison with literature and derivation of new qc method.
  - Summary of the laboratory test values are in the tab “Lab”
  - Laboratory test interpretations are in the tabs “SWCC”, “DS”, “OED”, “TXL”
  - Relevant CPTs with plotted qc, Fr and u2 profiles are in “CPT”
  - Interpreted G0 from are in “SCPT”
  - Comparison of the field tests and derivation of the new qc method are in “Field tests”
  - Comparison of methods from the literature are in “P-y Lit”
- Lateral pile prediction compared with test results are collated for the Bullsbrook tests and case histories in the excel file Calc – Case Histories.xlsx. This file also includes a comparison of the CPT soil behaviour type index for all the case histories in the tab “Robertson Charts”
- ALP analyses, and where necessary Adsec analyses, for predicting cases history data are in the folder \Case_Histories
Other files

\:\Thesis
Contains the pdf document of this thesis.

\:\Papers
Contains the pdf documents of the submitted/published papers.

\:\MATLAB\_Apps
Contains the MATLAB applications and manuals created for this thesis as discussed in \textit{Appendix E}. 