p-y Curves for Fatigue Analyses of Offshore Drilling Conductor Systems

By

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BEng.(Hons)

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Doctor of Philosophy of

The University of Western Australia

Centre for Offshore Foundation Systems
Oceans Graduate School

February 2023
Dedication

To
Jose Parra
Who encouraged me to embark on this journey
THESIS DECLARATION

I, Mariajose Guevara, hereby declare that:

The contents of this dissertation are original and have not been submitted in whole or in part for consideration for any other degree of qualification at this, or any other, university, except where specific reference is made to the work of others.

No part of this work will, in the future, be used in a submission in my name, for any other degree or diploma in any university or other tertiary institution without the prior approval of The University of Western Australia and where applicable, any partner institution responsible for the joint-award of this degree.

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The thesis contains work that has been published or prepared to be submitted for publication, some of which has been co-authored.

The following statements illustrate the contribution of the candidate to each of the co-authored work.

Details of the work:


Location in the thesis: Chapter 2

Candidate contribution to work: The candidate performed the centrifuge tests under guidance of the co-authors. All the data processing from the tests and interpretation was also done by the candidate. The candidate interpreted the data from the participants and performed a parametric Finite Element analysis for the paper. A full initial draft of the paper was prepared by the candidate, and all the co-authors contributed to the final version.

Details of the work:


Location in the thesis: Chapter 3

Candidate contribution to work: The candidate performed the centrifuge tests under guidance of the co-authors. All the data processing from the tests and interpretation was also done by the candidate. A full initial draft of the paper was prepared by the candidate, and all the co-authors contributed to the final version.
Details of the work:


Location in the thesis: Chapter 3

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Details of the work:


Location in the thesis: Chapter 3

Candidate contribution to work: The candidate performed the centrifuge tests under guidance of the co-authors. All the data processing from the centrifuge tests was done by the candidate. The data interpretation from the centrifuge, laboratory and p-y apparatus tests was also done by the candidate. A full initial draft of the paper was prepared by the candidate, and all the co-authors contributed to the final version.

Details of the work:


Location in the thesis: Chapter 4

Candidate contribution to work: The candidate designed and performed the centrifuge test in the example application section of the paper under the guidance of the co-authors. The data
processing from the test and calibration of the model to the test results were also performed by the candidate. The first and second co-authors prepared the first draft of the paper and all co-authors revised and contributed to the final version.

**Details of the work:**


**Location in the thesis:** Chapter 4

**Candidate contribution to work:** The candidate developed the calibration methodology under guidance of the second and third co-authors. The coding, debugging and implementation of the methodology were performed by the candidate. The candidate also proposed a simplification for the PICSI model. A full initial draft of the paper was prepared by the candidate, and all the co-authors contributed to the final version.

**Details of the work:**


**Location in the thesis:** Chapter 5

**Candidate contribution to work:** The candidate developed a simplified methodology for fatigue p-y curves under guidance of the second and third co-authors. The Finite Element analysis using different p-y curves methods were performed by the candidate. The candidate interpreted the data from the Finite Element analyses and compared it to experimental data from centrifuge tests. A full initial draft of the paper was prepared by the candidate, and all the co-authors contributed to the final version.
The candidate’s statement regarding her contribution to each of the works listed above are correct.

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ABSTRACT

Fatigue of conductor-wellhead components of subsea drilling systems has become a critical design driver in the recent years, particularly due to the increase of drilling depths, riser dimensions and weight and dimensions of the lower packages. Drilling campaigns have become longer to cope with greater water depths, increasing the drilling period, and hence the stress cycles in the components of the drilling system. In addition, wellhead-conductor systems in need of decommissioning will have to support greater loads than they were designed for, as the weight of the new lower safety packages has increased. Therefore, accurate determination of the stresses in the components is critical.

The conductor, which is the most external layer of the system pipes below the seabed, is in permanent contact with the soil, which provides lateral support. When the drilling vessel moves due to environmental loads, the whole system moves, as it is connected through a riser system top-tensioned at the vessel. Even though during operations this movement is normally very small, the repeated cycling of these motions produce stress cycles at specific locations or “hot spots” of the system and could lead to fatigue damage at these locations. Accurate calculation of stress along the length of the conductor and casing string is therefore important for determining the fatigue life of individual system components. The soil-conductor lateral stiffness plays a very important part in the response of the whole structure because it determines the distribution of stresses along the components of the conductor-wellhead system. Due to the cyclic nature of the system movements, the soil stiffness changes, reducing in strength and stiffness due to remoulding, and regaining them as dissipation of pore pressures occurs. Therefore, understanding the changing soil-conductor stiffness due to the motions imposed by the environmental loads on the vessel is of paramount importance and is the topic of this thesis.

Firstly, physical modelling tests were conducted to understand the load-history impact on the soil-conductor stiffness behaviour on two types of soft soil: kaolin clay and reconstituted carbonate silt. The tests were conducted using a rigid pile setup in the centrifuge and p-y apparatus tests. Multiple cycling sequences were explored in the range of amplitudes expected at the top of the conductor during drilling operations. In addition, the sequences imposed were combinations of events of increasing and symmetrically decreasing cyclic amplitudes with dissipation periods in between, directed at recreating the load history experienced due to conductor motions during the well drilling. It was found that large amplitude cycling impacted
subsequent smaller amplitude cycling, by reducing the strength and stiffness of the soil-conductor response. It was also observed that pore pressure dissipation between and during cycling sequences lead to a regain in strength and stiffness of the soil-conductor response.

Secondly, a framework was developed to model the effect of load history on p-y curves by accounting for softening due to cyclic remoulding and recovery due to pore pressure dissipation. The model was developed for soils that tend to densify with shearing, usually soils that plot to the right of the critical state line. A calibration methodology for the framework was proposed using experimental data and an optimisation process, from which parameters were obtained for the two soil types studied. Comparisons with experimental data from a separate test (using a flexible pile) showed that the framework using the calibrated parameters provided a good match to the cyclically induced changes in bending moment in the experiment.

Finally, the impact of the p-y curves selected on the bending moment profile of a conductor and consequently on the fatigue life assessment of the components of a drilling conductor system was discussed. This was achieved by building a structural beam-p-y model using a finite element analysis software, and comparing the results generated by using different p-y curves. The model aimed at recreating the setup and loading sequence of centrifuge tests performed on a flexible pile embedded in carbonate silt, in order to compare how the p-y curves influenced the shape of the bending moment profile. A simplified p-y curve method for fatigue analysis was presented, based on the steady-state approach proposed by previous authors, and found to replicate the results of the test in fully-remoulded conditions. In addition, results using the fatigue p-y curves recommended by the new ISO 19901-4 draft were also analysed. These were found to overpredict the bending moments at depths close to the seabed, and underpredict them at lower depths were connections between conductor or casing joints could be located.

In summary, this research has studied the changes in strength and stiffness of soil-conductor interaction due to load history, remoulding and pore pressure dissipation, and provided approaches for their inclusion in design. Furthermore, this study highlights the impact of soil-conductor stiffness modelling on fatigue life assessment of offshore conductor-wellhead systems.
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NOTATION

Symbols used and as defined in each chapter

Chapter 1

\( k \)  
Secant stiffness

\( P \)  
Resistance of the soil per unit length of pile

\( y \)  
Lateral displacement of the pile

Chapter 2

\( COV \)  
Coefficient of variation

\( D \)  
Conductor/pile diameter

\( D_{50} \)  
Soil mass median diameter

\( EI \)  
Pile rigidity

\( H \)  
Load at pile head

\( g \)  
Gravity acceleration

\( G_s \)  
Specific gravity

\( k_{su} \)  
Undrained shear strength gradient with depth

\( N \)  
Number of cycles

\( N_p \)  
Lateral bearing factor

\( N_{T-bar} \)  
T-bar capacity factor

\( PI \)  
Plasticity index

\( P_u \)  
Ultimate soil pressure

\( R_{ave} \)  
Average roughness of the pile

\( S_t \)  
Soil sensitivity

\( s_u \)  
Soil undrained shear strength

\( y \)  
Lateral displacement of the pile

\( \alpha \)  
Pile-soil interface roughness factor

\( \gamma \)  
Shear strain

\( \tau_{cyc} \)  
Cyclic shear stress

\( \tau_{xy} \)  
Shear stress
Chapter 3

$D$  Conductor/pile diameter
$ch$  coefficient of lateral consolidation
$cv$  coefficient of consolidation
$F_h$  Lateral load
$F_{h,ult}$  Measured monotonic lateral capacity
$g$  Gravity acceleration
$G$  Secant shear modulus
$G_{max}$  Maximum secant shear modulus
$G_s$  Specific gravity
$H$  Head load
$k_{su}$  Undrained shear strength gradient with depth
$k_{p-p}$  Measured peak to peak secant stiffness  $k_{p-p} = \frac{\Delta P}{\Delta y / D}$
$K$  Normalised cyclic secant stiffness
$K_0$  Coefficient of earth pressure at rest
$N$  Number of cycles
$N_p$  Lateral bearing factor
$N_{T-bar}$  T-bar capacity factor
$P$  Horizontal soil pressure
$P_u$  Ultimate soil pressure
$PI$  Plasticity index
$S_t$  Soil sensitivity
$s_u$  Soil undrained shear strength
$s_u/\sigma'_{v0}$  Shear tress ratio
$t$  Time
$t_w$  Pile wall thickness
$T$  Dimensionless time, $T = t*ch/D^2$
$w$  Moisture content
$y$  Lateral displacement of the pile
$\gamma$  Shear strain
$\Delta F_h$  Cyclic range of horizontal load
$\Delta P$  Cyclic range of horizontal pressure
\( \Delta y \)  
Range of lateral displacement  
\( \Delta y/d \)  
Cyclic amplitude range  
\( \Delta y/2d \)  
Half the cyclic amplitude range  
\( \xi_1 \)  
Elastic strain scaling factor  
\( \xi_2 \)  
Plastic strain scaling factor  
\( \sigma'_{v0} \)  
Vertical effective stress  
\( \tau_{\text{cyc}} \)  
Cyclic shear stress  
\( \tau_{xy} \)  
Shear stress

**Chapter 4**

\( c_r \)  
Dimensionless parameter controlling rate of consolidation effect  
\( c_p \)  
Dimensionless parameter controlling power of consolidation effect  
\( c_v \)  
Coefficient of consolidation  
\( d \)  
Pile diameter  
\( dL \)  
Projected embedded portion of the pile  
\( D \)  
Damage Index  
\( d_a \)  
Dimensionless constant for damage generation amplitude effect  
\( d_r \)  
Dimensionless constant for damage generation rate  
\( d_p \)  
Dimensionless constant for damage generation power  
\( E \)  
Error function  
\( E_i \)  
Tangent stiffness of each segment of the backbone p-y curve  
\( EI \)  
Pile rigidity  
\( f_i \)  
Force carried by each spring-slider element of the Parallel-Iwan model  
\( F_h \)  
Lateral force  
\( g \)  
Gravity acceleration  
\( G \)  
Secant shear modulus  
\( G_{\text{max}} \)  
Maximum secant shear modulus  
\( G_s \)  
Specific gravity
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<td>$T$</td>
<td>Dimensionless time, $T = t^{*}c_v/d^2$</td>
</tr>
</tbody>
</table>
\( T_{cr} \) Scaled dimensionless time
\( u_p \) Plastic displacement in Parallel-Iwan spring
\( u_e \) Elastic displacement in Parallel-Iwan spring
\( V \) Scaled dimensionless pile velocity
\( w \) Moisture content
\( y \) Lateral displacement
\( y^{pp} \) Pile displacement
\( y_{ref} \) Lateral reference displacement
\( y(t) \) Time-history of displacements
\( z \) Depth below the mudline
\( \alpha \) Pile-soil interface roughness factor
\( \delta D \) Increment in damage
\( \delta H \) Increment in hardening
\( \delta t \) Interval of time
\( \delta y \) Displacement Increment
\( \delta y/d \) Normalised displacement increment
\( \Delta P \) Cyclic range of horizontal pressure
\( \Delta y \) Range of lateral displacement
\( \Delta y/d \) Cyclic amplitude range
\( \Delta y/2d \) Half the cyclic amplitude range
\( \kappa^* \) Slope of hardening path
\( \lambda^* \) Slope of damage = 0 line.
\( \xi_1 \) Elastic strain scaling factor
\( \xi_2 \) Plastic strain scaling factor

Chapter 5

\( A_s \) Parameter of ISO fatigue p-y curves
\( B_s \) Parameter of ISO fatigue p-y curves
\( c_r \) Dimensionless parameter controlling rate of consolidation effect
\( c_p \) Dimensionless parameter controlling power of consolidation effect
\(c_v\)  
coefficient of consolidation  

\(d\)  
Pile/Conductor diameter  

\(d_a\)  
Dimensionless constant for damage generation amplitude effect  

\(d_p\)  
Dimensionless constant for damage generation power  

\(d_r\)  
Dimensionless constant for damage generation rate  

\(g\)  
Gravity acceleration  

\(G\)  
Secant shear modulus  

\(G_{\text{max}}\)  
Maximum secant shear modulus  

\(h_p\)  
Variation of hardening slope  

\(k\)  
Secant stiffness of a p-y curve  

\(k_{p-p}\)  
Peak to peak secant stiffness  

\(k_{su}\)  
Undrained shear strength gradient with depth  

\(K\)  
Normalised secant stiffness  

\(K_{\text{max}}\)  
Normalised peak secant stiffness at small displacements  

\(L\)  
Pile length  

\(M_{\text{peak } Nzi}\)  
bending moment at a specific depth \(z_i\), and for the positive peak displacement at specific cycle \(N\)  

\(M_{\text{trough } Nzi}\)  
bending moment at a specific depth \(z_i\), and for negative peak displacement at specific cycle \(N\)  

\(n\)  
Parameter that determines the shape of the p-y curve  

\(n_g\)  
Radial acceleration of the centrifuge  

\(N_p\)  
Lateral bearing factor  

\(OCR\)  
Overconsolidation ratio  

\(p_{fa}\)  
Soil resistance for fatigue actions  

\(p_u\)  
Ultimate soil resistance  

\(P\)  
Lateral soil pressure  

\(P_f\)  
Lateral soil pressure for fatigue actions  

\(P_{\text{rem}} / P_u\)  
Normalised fully remoulded soil pressure  

\(P_u\)  
Ultimate soil pressure  

\(PI\)  
Plasticity index  

\(s_u\)  
Soil undrained shear strength  

\(t\)  
Time
$y$ Lateral displacement

$y_{rem}/d$ Normalised displacement at which $P_{rem}/P_u$ is reached

$(z/d)_{peak}$ Normalised depth at which the peak bending moment is observed

$\alpha$ Pile-soil interface roughness factor

$\delta t$ Interval of time

$\delta y$ Displacement increment

$\Delta M$ Bending moment range at a specific pile depth

$\Delta M_{max}$ Maximum bending moment range along the pile

$\Delta P$ Cyclic range of horizontal pressure

$\Delta y$ Range of lateral displacement

$\Delta y/2d$ Half the cyclic amplitude range

$\gamma$ Shear strain

$\gamma'$ Effective unit weight

$\kappa^*$ Slope of hardening path

$\lambda^*$ Slope of damage $= 0$ line.

$\sigma'_{v0}$ Vertical effective stress

$\xi_1$ Elastic strain scaling factor

$\xi_2$ Plastic strain scaling factor
CHAPTER 1 – INTRODUCTION

1.1. Background and motivation for the research

1.1.1. Riser system structure

A riser system is a subsea pipe that connects the operating floating facility with the wellhead system on the seabed. These risers have a top tension applied from the operating vessel and are used during the drilling stage. The top tension is applied to compensate for the changes in height produced by the waves and the heave. Flexible connections are located at the top and bottom of the riser to allow limited rotations to occur. This type of riser system is very sensitive to the vessel, wave and current motions.

A schematic illustration of a drilling riser system is shown in Figure 1-1: the riser is connected to the vessel by a flexible joint (UFJ: upper flex joint) and pulled at the top by a tensioning system on the vessel. The riser is connected at the bottom by a flexible joint (LFJ: lower flex joint) that connects then to the lower packages, which are heavy safety components of the riser system. These lower packages are connected at the bottom to the wellhead system, which is joined to the conductor and casing system that is used to drill the well and prevent the walls of the borehole from collapsing.

![Figure 1-1. Schematic of a drilling riser system and soil modelling.](image)
At some depth below the wellhead system, the conductor (which is the outermost pipe in the lower section) intersects the seabed. The soil around the conductor provides lateral support, which in turn plays a very important part in the response of the whole structure. When the vessel moves due to environmental loads, the whole system moves. Even though during operations this movement is small, the repeated cycling of these motions produce stress cycles at specific locations or “hot spots” within the system – and the resulting progressive and localised structural damage is called fatigue. Fatigue of conductors has become a critical design driver in recent years due to increases in drilling depth, riser dimension and weight, and the dimensions of the lower packages (Howells et al. 2015). Also, drilling campaigns have become longer to cope with greater water depths, increasing the drilling period from 60 days to up to 300 days (Sharma 2016).

Hot-spots (Figure 1-2) are locations where stress concentrates, such as discontinuities in the geometry, welds, changes in diameter or thickness, changes in stiffness such as those produced at the top of cement between the conductor and casing (DNV GL 2018). Although most hot-spots occur on the wellhead and upper portions of the conductor, some can be located below mudline: either in the connections between joints, or in the body of the conductor or casing. Accurate calculation of stress along the length of the conductor and casing string is therefore important for determining the fatigue life of individual system components.

Figure 1-2. Hot-spots location in a drilling wellhead-conductor system.

The standard used in current practice to assess the fatigue of offshore drilling conductor systems is the DNVGL-RP-E104 “Wellhead Fatigue Analysis” (DNV GL 2018), which
references to other standards for environmental and structural guidance (DNV GL 2016, 2017) and to API RP 2GEO (API 2014) for soil modelling guidance.

1.1.2. System modelling and response
The system is impacted by environmental loads that are described by combinations of wind, wave and current, and which may occur simultaneously (DNV GL 2018). Wave loading is typically the most critical for fatigue analysis, and is best described as a combination of seastate parameters (such as wave height, peak period and heading angle) with each seastate having a defined probability of occurrence (DNV GL 2017). This combination is called a wave scatter diagram and reflects either a yearly or seasonal occurrence. The parameters of each seastate of the wave scatter diagram are transformed into a random time domain of wave heights by using a spectrum shape, carefully selected to reflect the contributions of sea swell and waves specific to the site, and inverse fast Fourier transform (IFFT). The vessel motion in response to waves is characterised through response amplitude operators (RAO) which describe the motions of the vessel due to wave loading for different wave periods.

In order to build the model used for obtaining the stresses required to perform the fatigue analysis, a series of assumptions are required. These relate to the hydrodynamic response of the system, structural aspects of the riser-wellhead-conductor system, and soil-conductor interaction. While this project focuses on soil-conductor interaction, a brief description on the structural aspects of the (holistic) analysis is given below.

Traditionally, fatigue life is determined by running coupled (or global) time-domain analysis of the whole system including the vessel, riser, lower components, conductor, casing and soil (assuming no offset from the centreline, i.e., 2-way loading). However, an alternative model is mentioned in the DNVGL-RP-E204 (DNV GL 2018) for cases where more complex systems are modelled. In this (de-coupled) modelling approach, the wellhead-casing-soil system is represented as a simplified equivalent stiffness and used as a boundary condition for global analysis. The de-coupling could be implemented at different locations in the system, such as the High-Pressure Housing (HPH) represented as the top end of the casing string in the wellhead in Figure 1-2.

There are two types of fatigue analysis: analysis using material specific S-N curves; and analysis using fracture mechanics. The former is the recommended for most cases, except in special situations.
The first type of analysis involves calculating stresses at the designated hot spots of the structure for each loading case, and then multiply these stresses by a stress concentration factor (SCF) – which is the ratio of the hot spot stress range over the nominal stress range. This applies mostly to the locations of welds, joints, and holes; with DNV RP-203 (DNV GL 2016) providing a series of charts and equations to calculate appropriate SCF values.

The long-term distribution of factored stresses is then transformed into a stress histogram, which is comprised of blocks of constant stress range along with a number of repetitions. The stress histogram is used to calculate the damage produced by plotting the stress range and number of cycles on an $S$-$N$ curve, which is a graphical representation of the relation of fatigue life, expressed in number of cycles, and fatigue strength. DNV-203 depicts a series of $S$-$N$ curves for different materials and provides guidance on how to build one if none of those are appropriate.

Total fatigue damage is then estimated using the Palmgren-Miner linear cumulative damage method (Miner 1945), with the result then translated into an estimate of the unfactored fatigue life, expressed in years. Safety factors are then applied to determine the design fatigue life, depending on the element safety class as determined by the DNV standard (DNV GL 2018).

### 1.1.3. Assumptions regarding soil-conductor interaction modelling

Conductor-soil interaction is often modelled using the same approach developed for pile analysis: whereby the conductor is represented as an elastic beam that is restrained laterally by non-linear Winkler springs distributed along its length. The load is assumed to act at the top of the conductor and could comprise either a moment or lateral load, or a combination of both.

Functions that define the non-linear response of the springs are called $p$-$y$ curves, defined in terms of the resistance of the soil per unit length of pile ($P$) in response to a lateral displacement ($y$) of the pile. The gradient of the secant line that joins the origin with a specific point of the curve is called the secant stiffness ($k$) of the $p$-$y$ curve and is defined by Equation [1-1]:

$$
k = \frac{P}{y} \quad \text{[1-1]}$$

The soil characteristics required to define a $p$-$y$ curve usually vary with depth, because soil strength and stiffness depend on confining pressure – the greater the depth, the more the confinement and hence the stronger and stiffer the soil becomes. Therefore, the $p$-$y$ curves must be specified at discretised depths along the relevant length of the conductor.
The failure mechanism near the surface for clays is often taken as that presented in Figure 1-3, which was first proposed by Murff and Hamilton (1993). Close to the seabed, the portion of the soil that is pushed by the conductor fails in a wedge mechanism and the resistance comes from the friction of the failure planes of the wedge, and deformation and self-weight of the soil within the wedge. Below the wedge, soil flows around the pile in a localised manner.

![Figure 1-3](image)

**Figure 1-3.** Components of lateral resistance in clay close to the surface. Extracted from Murff and Hamilton (1993)

The current API RP 2GEO guidelines (API 2014) recommend curves that are broadly thought to be inappropriate to address the conductor-soil interaction for typical motions. These recommendations were originally developed from the results of monotonic pushover tests conducted on piles, and so are not suitable for small amplitude motions such experienced by conductors during typical drilling operations – typical motions at the wellhead of a riser system are below 10% of the conductor diameter, whereas full monotonic capacity may only be developed beyond 20% of the diameter (Jeanjean 2009). In addition, since the environmental loads on the system are cyclic in nature, p-y curves that better reflect this scenario are required.

### 1.2. Recent studies

Recent centrifuge experiments and finite element analysis (Jeanjean 2009) found that the monotonic p-y curves recommended in API (2014) for lateral analysis of piles are too soft for conductor analysis.
The centrifuge experiments involved an instrumented conductor pushed in kaolin clay, with strain gauges along its length to measure the bending moment. The study (Jeanjean 2009) concluded that the use of the current curves for conductor fatigue analysis, even though potentially conservative for the elements of the system below the seabed, leads to unconservative estimates of stress for the elements above the seabed (such as the wellhead and lower marine packages). This research proposed a new monotonic p-y curve and quantified the degradation of stiffness with load cycles.

The formulation proposed by Jeanjean (2009) curves was further developed in Jeanjean et al. (2017) by linking the procedure to simple shear test results by means of the scaling method proposed by Zhang and Andersen (2017). This procedure was originally developed from a database of simple shear tests and numerical modelling.

Further studies have since been conducted to address conductor lateral analysis (Zakeri et al. 2016a; b) based on centrifuge tests on Gulf of Mexico clay and kaolin, as well as p-y apparatus tests (Zakeri et al. 2017) on different types of clay. In these studies, the authors determined that the monotonic p-y curves proposed by Jeanjean (2009) require modification in terms of stiffness and strength in order to account for the effects of cyclic loading. Their study found that after 50-100 cycles of loading a “steady state” stiffness was reached, beyond which no further degradation occurred. The steady-state approach assumes that there is a unique unload-reload stiffness for a given displacement for which the stiffness and damping stabilise. A method to modification the monotonic p-y curve for fatigue analysis was later proposed in Zakeri et al. (2019), based on the earlier results from Zakeri et al., (2016a).testing in soft to very hard clays. Note that the method proposed by Zakeri et al., (2019) the transient phase before reaching the “steady state” is disregarded.

A recent method (Komolafe and Aubeny 2020) extends the “steady state” stiffness approach by modelling the transition from undisturbed stiffness to the fully remoulded state, for a range of displacements expected for drilling conductors.

None of the above approaches are able to model the entire load history, and its effect on soil-conductor stiffness. When soil is loaded cyclically in an undrained manner, its strength degrades due to pore pressure build up that reduces the effective stress within the soil particles (Hodder et al. 2013). However, if a cyclic episode is followed by a period of no motion, then dissipation of the generated pore pressure will lead to a reduction in moisture content and an
increase in density around the conductor. Centrifuge tests have shown that soil stiffness and strength increase after a consolidation period following cyclic loading (Hodder et al. 2009; Zhou et al. 2018). Recent studies (Clukey et al. 2017) have noted the effects that episodes of remoulding and reconsolidation have on soil-structure interaction for steel catenary risers, but the impact of this on fatigue life has not been fully investigated.

Testing of laterally loaded piles in a centrifuge (Zhang et al. 2011) have shown that although episodes of pile cyclic movement lead to a reduction in lateral resistance due to soil remoulding and pore pressure build up, subsequent periods of consolidation led to a similar magnitude of recovery. This study also observed that subsequent cyclic episodes showed a reduction in the level of softening of the soil. The authors link cyclic degradation of the soil laterally loaded by a pile with cyclic response of T-bar penetrometer tests, whereby the T-bar (Stewart and Randolph 1991) is a test in which a cylindrical bar is pushed into the soil to determine its undrained shear strength using a correlation factor that depends on the soil type.

Building on the above, recent studies (Doherty et al. 2019) indicate that the system response is different when it is loaded cyclically with increasing amplitude up to peak compared to a subsequently decreasing amplitude (post peak). This study also showed that an “event 1” (cyclic amplitude increasing and then decreasing) has lower stiffness than an “event 2” of the same characteristics, but allowing consolidation between the events. There is no account for this behaviour when assuming a steady-state condition for stiffness.

A new edition of the ISO 19901-4 (ISO/DIS, 2021, currently under review) proposes a new framework for both monotonic and fatigue lateral loading of conductors that aims to overcome limitations in the previous guidelines. The proposed ISO framework for monotonic p-y response in clays is based on the method originally proposed by Jeanjean et al., (2017), while the curve also provides recommendations for cyclic and fatigue p-y curves:

- The recommended p-y curves for monotonic loading can be developed using one of two methods: by scaling from simple shear laboratory tests (following the method proposed by Jeanjean et al., 2017) or by using default normalised curves.
- The p-y curves for fatigue analysis are intended to represent “steady state” conditions, and are based on the method proposed by Zakeri et al. (2019).
1.3. Research aims

This thesis aims to provide a full life understanding of changing soil-conductor stiffness due to motions imposed by environmental loads acting on the vessel, and their potential impact on the fatigue life assessment of the components of the system. This was achieved through three main aims:

**Aim 1: Understand soil-structure behaviour through physical modelling tests considering realistic load histories experienced due to conductor motions during drilling operations.**

Physical modelling tests were conducted to understand the load-history impact on soil-conductor stiffness in kaolin clay and reconstituted carbonate silt. The tests were conducted using a rigid pile setup in a geotechnical centrifuge and via testing in a purpose-built p-y apparatus (Zakeri et al. 2017). Multiple cycling sequences were explored over the range of amplitudes expected at the top of the conductor during drilling operations. This included combinations of events of increasing and (symmetrically) decreasing cyclic amplitudes, and with dissipation periods in between, directed at addressing the following questions:

- How does an offset affect the cyclic degradation?
- Is a steady state reached, such that no further changes in stiffness occur at the same cyclic amplitude level?
- How does the behaviour of the decreasing amplitude phase of an “event” compare with the increasing amplitude phase?
- How does a consolidation period after cycling impact future cyclic events?
- Can pore pressure dissipation and regain in stiffness happen at the same time as cycling occurs?

**Aim 2: Propose a solution to account for load-history in the soil-structure interaction analysis – specifically, the development of intelligent p-y curves that account for remoulding and loss of stiffness during the cycling, and for regain in strength and stiffening due to pore pressure dissipation.**

The second aim of the research project was to develop a framework to model the effect of load history on p-y curves by accounting for softening due to cyclic remoulding and
recovery due to pore pressure dissipation. The framework is inspired by critical state soil mechanics, while incorporating features observed from experimental testing.

The model was developed for soils that tend to densify with shearing, usually soils that plot to the right of the critical state line. Soils that plot *slightly* to the left of the critical state line tend to generate pore pressures when cycled and the model might be applied to these soils as well. Nevertheless, this thesis only considers normally consolidated soils to calibrate the model. A calibration methodology for the framework is proposed using experimental data and an optimisation process, from which parameters were obtained for two soil types: kaolin clay and reconstituted carbonate silt.

**Aim 3: Assess the potential sensitivity in fatigue life assessment of offshore drilling riser systems to the p-y models used.**

The third aim of the research project was to determine the impact of select p-y curves on the bending moment profile of a conductor, in order to discuss their potential impact on the fatigue life of the components of a drilling conductor system. This was achieved by building a model using the finite element analysis software LAP (Doherty 2017), and comparing the results generated by using different p-y curves. The model aimed at recreating the setup and loading sequence of centrifuge tests performed on a flexible pile embedded in carbonate silt, in order to compare how the p-y curves influenced the shape of the bending moment profile. A (new) simplified p-y curve method for fatigue analysis is presented as an alternate to the steady-state approach proposed by previous authors (and proposed in the upcoming ISO code). In addition, results are presented using the full-life p-y framework developed (White et al. 2022). Comparisons against the experimental data are drawn and the potential impact on fatigue life assessment discussed.

Figure 1.4 shows a map of the thesis aims and in which chapter they are addressed.
1.4. Thesis outline

The thesis is divided into four main chapters, outlined below:

- Chapter 2: The current state of practice for design of conductors and its implication for fatigue life analysis are described. The chapter justifies the development of updated p-y curves that reflect the change in stiffness and capacity produced by cyclic loading during the drilling campaign, to better assess the fatigue life of the wellhead system.
• Chapter 3: Key features from different cyclic loading conditions that impact soil-conductor lateral behaviour are explored. This is performed by analysing the experimental results from centrifuge, laboratory and p-y apparatus testing.

• Chapter 4: The p-y model developed to capture the features observed from experimental testing is presented. In addition, a methodology to calibrate the model from physical modelling tests using optimisation is presented, and the calibrated parameters validated.

• Chapter 5: A simplified p-y model based on the “steady-state” approach is developed and validated. Results generated using this model, the model outlined in Chapter 4, and the proposed ISO recommendations are discussed in the context of their impact for fatigue analysis of conductors.

The thesis concludes with Chapter 6, in which contributions from this research and potential research themes for future work are summarised.

References


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CHAPTER 2 – CURRENT STATE OF PRACTICE

Prologue

Chapter 2 comprises a paper submitted to the Journal of Geotechnical and Geoenvironmental Engineering titled: “Evaluating uncertainty with engineering judgement in predicting the lateral response of conductors – a case study” (Guevara et al., 2021).

The chapter presents the results of a prediction event in which practitioners around the world were invited to predict the monotonic and cyclic behaviour of a conductor in soft soil. The predictions were compared to physical model tests performed in the centrifuge on a slender flexible pile resembling a drilling conductor. In addition, companion laboratory tests were provided to the participants akin to characterisation campaigns routinely executed in industry. The chapter explores the aspects of the modelling and laboratory data that the participants found the most challenging when working on their prediction. Additionally, the predictions are analysed to determine the uncertainty of associated with engineering judgement in design.

Accompanying the variability analysis is a parametric analysis in which four different methods for generating p-y curves were evaluated. Aspects of the shape and maximum capacity of the curves such as stiffness and rate dependency are discussed with their implications on the fatigue life assessment of drilling conductor systems.

It was found that the p-y approach was more widely used and performed better in all of the four loading sequences when compared to finite element/ finite different approaches. Therefore, in the case of drilling conductors, further improvements would likely rely on more accurate estimation of the p-y curves and their evolution with cyclic loading.

Therefore, the chapter presents a justification for the development of improved p-y curves that reflect the change in stiffness and capacity produced by cyclic loading during the drilling campaign, to better assess the fatigue life of the wellhead system.
2.1. Evaluating uncertainty associated with engineering judgement in predicting the lateral response of conductors – a case study

Abstract

The paper presents the results from a prediction event, organised by the University of Western Australia (UWA) and the National Geotechnical Centrifuge Facility (NGCF), and performed as part of the International Symposium on Frontiers in Offshore Geotechnics (ISFOG 2020) to assess uncertainty in predicting the monotonic and cyclic lateral response of conductors. Geotechnical professionals from around the world were invited to predict the response of a model conductor (a flexible pile) subjected to a series of loading sequences in a centrifuge. A normally consolidated fine-grained soil was used in the tests, which was characterised by soil element and in-flight T-bar penetrometer testing. While some participants provided accurate predictions, the mean response was an overestimate of the monotonic and cyclic load at the pile head, which was significant for large and very small displacements. An analysis of the submissions is presented in order to quantify the variability of the predictions received, assess the consequences on design, and inform regarding uncertainty associated with engineering judgment in design.

2.1.1. Introduction

Predicting the cyclic response of soft soils is a significant challenge facing offshore geotechnical engineers. This is notably the case for drilling conductors, whereby modelling soil-conductor cyclic behaviour is critical for assessing the fatigue life of the system. Current API RP 2GEO guidelines (API 2014) recommend monotonic p-y curves that are based on the results of pushover testing performed on piles, and it is generally accepted that the recommended p-y curves underpredict lateral stiffness at modest displacement levels, which are the most relevant for conductor fatigue assessment. Without additional guidance, practitioners rely on past experience, published studies and judgement to support design, without documented consensus about methodology and choice of design parameters.

Acknowledging the need for a better understanding of soil-conductor interaction and the benefit of a transparent and collegial approach, an international prediction event was conducted as part of the ISFOG 2020 conference in Austin, Texas. The event was managed by the National Geotechnical Centrifuge Facility (NGCF) at the University of Western Australia (UWA). Four loading scenarios were considered in the event:
1. A monotonic pushover test, up to a displacement of 1 conductor diameter ($D$);
2. A two-way cyclic test at a displacement amplitude of ±0.02 $D$;
3. A two-way cyclic test at a displacement amplitude of ±0.10 $D$; and
4. A two-way cyclic test at load amplitude of ±153 kN.

Motivation for this case study is drawn from the success of previous events, such as the predictions of spudcan penetration for jack-up platforms (Van Dijk and Yetginer 2015) and of footing settlement (Doherty et al. 2018), with both cases comparing practitioner estimates to field observations. While Van Dijk and Yetginer 2015 focused on the selection of parameters and method that yield the most accurate prediction, Doherty et al. 2018 focused on variability in the predictions received. Both studies highlight the possible range in design outcomes – even when the ground conditions are well known, the load cases clearly defined, and calculation methods reasonably well developed.

Further motivation stems from the increasing use of probabilistic approaches in geotechnical research and practice. While these methods capture uncertainty in many aspects of the design process, they do not necessarily address the importance of judgement, as employed by practitioners. Previous studies have indicated the importance of human factors in design (Sowers 1993), while others have proposed approaches to account for uncertainty in data interpretation in design (Whitman, 2000).

In effect, this study is a unique elicitation exercise that helps to assign uncertainty associated with engineering judgment in design – achieved by performing highly controlled (centrifuge) tests in a uniform soil, along with a ‘common’ level of soil interpretation. The observed variability in predictions is likely to represent a lower bound scenario compared with real design cases, which may involve complex stress histories and foundation geometries, along with additional (subjective) interpretation of the soil. Accordingly, while focusing here on the study of a laterally loaded conductor, the findings are likely to have relevance for other scenarios.

Building on the above, a statistical analysis is first presented to quantify the variability of the predictions received. In order to provide insights on what could be considered best practice, a parametric analysis using the software LAP (Doherty 2017) and adopting methods used by several predictors, is then presented – and the results explained in terms of their implication for fatigue design of conductors and ultimate lateral capacity of piles.
2.1.2. Case investigated

The problem selected was the lateral cycling loading of a flexible pile (representative of an offshore conductor) under free head conditions. Data provided to the participants included:

- A detailed account of the testing procedure from installation to end of loading;
- Test setup, and the dimensions and mechanical characteristics of the pile; and
- Soil properties as determined from centrifuge T-bar penetrometer tests, core sampling (for water content) at the end of testing, as well as advanced laboratory testing (simple shear tests, triaxial compression and extension).

The data made available to participants is provided as supplemental data to this paper.

The pile geometry is shown in Figure 1-1 with prototype dimensions (at a testing acceleration of 80g) reported in Table 2-1. The pile was free to rotate and translate laterally at the load application point, located 3.36 m above the seabed, while the pile length and rigidity were chosen to minimise the rotation and translation at the pile toe. Participants were advised to assume the outer pile surface was fully rough.

![Figure 2-1. Problem Geometry.](image)

Table 2-1. Model and prototype dimensions.

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Model at 1g</th>
<th>Prototype at 80 g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (from hinge to toe)</td>
<td>270 mm</td>
<td>21.6 m</td>
</tr>
<tr>
<td>Outer Diameter (Aluminium pile)</td>
<td>12 mm</td>
<td>0.96 m</td>
</tr>
<tr>
<td>Wall thickness (Aluminium pile)</td>
<td>0.45 mm</td>
<td>0.036 m</td>
</tr>
<tr>
<td>Wall coating (Aluminium pile + Epoxy coating)</td>
<td>0.96 mm</td>
<td>0.0768 m</td>
</tr>
<tr>
<td>Total Outer Diameter</td>
<td>13.92 mm</td>
<td>1.114 m</td>
</tr>
<tr>
<td>Embedment</td>
<td>228 mm</td>
<td>18.24 m</td>
</tr>
</tbody>
</table>
2.1.2.1. Experimental approach

Model and instrumentation

The model pile was manufactured from aluminium 6061T6, with a Young’s modulus of 68.9 GPa and instrumented with 13 pairs of strain gauges 20 mm apart, starting 15 mm from the toe as illustrated in Figure 2-2. The pile was covered in epoxy, resulting in an outer diameter of 13.92 mm at model scale (1.114 m at prototype scale). The free head (zero moment) condition was achieved by placing a hinge at the load bracket, 2.5 $D$ from the top of the pile. Once the pile was installed, the hinge was located at 42 mm (model scale) or 3.36 m (prototype scale) above the seabed. For simplicity, throughout this paper the location of the hinge will be referred to as the pile head. Once installed, the downward displacement of the pile was restricted with steel cables connected to the hinge pins during the test.

![Figure 2-2. Illustration of centrifuge setup.](image)

The horizontal load applied to the pile was measured with a load cell located in the loading arm. The pile displacement and rotation at the load application point were measured using two lasers and a flat plate bracket as a target. The pile setup is shown in Figure 2-2.

In order to make their predictions, participants were provided with a rigidity ($EI$) of 770 MN.m$^2$ which was determined via a cantilever test that had been performed on an aluminium pile of outer diameter and wall thickness as indicated in Table 2-1, but without the strain gauges. When the same test was later performed on the strain gauged pile, it was observed that the epoxy required to cover the strain gauges and the cables had increased the rigidity of the pile by about 15% (i.e. $EI = 889$ MN.m$^2$). Further, the actual pile roughness was measured after the prediction event had started, using a roughmeter and measuring over six different portions of the epoxy-coated pile. This yielded an average roughness of the pile $R_{ave} = 0.445$ m, resulting
in a relative roughness of $R_{ave}/D_{50} = 0.44$. Based on this relative roughness, a roughness factor of $\alpha = 0.5$ (where $\alpha = 0$ represents a perfectly smooth interface and $\alpha = 1.0$ a fully rough interface) was considered the most appropriate for the pile-soil interface – which differs from the fully rough condition advised to the predictors.

To understand how these differences in stiffness and roughness would affect the results, a lateral push analysis was performed using the software LAP (Doherty 2017) and $p$-$y$ curves determined following the method proposed by Jeanjean et al. 2017. The load was applied at the hinge level, and the prototype pile length and total outer diameter used were those specified in Table 2-1. To estimate the ultimate lateral soil pressure ($N_p , s_u$) with depth, the undrained shear strength gradient ($k_{su}$) was set as 1.65 kPa/m, as obtained from the T-bar tests, and a no-gapping condition was assumed. The normalised $p$-$y$ curve selected was the harmonised curve for clays with $s_u \leq 100$ kPa (Jeanjean et al. 2017). Figure 2-3 compares the calculated bending moment profiles using the rigidly and interface roughness values provided to the predictors, with the ones calculated using the rigidity and interface roughness values that were re-evaluated after the prediction event.

![Figure 2-3](image.png)

**Figure 2-3.** Effect of changing pile rigidity and roughness on bending moment profile at 0.1$d$ and 1.0$d$ pile head displacement.

As shown, the effect on bending moment profile is small, with peak bending moment for a pile with $EI = 889$ MN.m$^2$ and $\alpha = 0.5$ being 17% higher (0.1$D$) and 6% lower (1.0$D$), respectively, than for the pile with $EI=770$MNm$^2$ and $\alpha = 1$. While not shown, the comparison in terms of pile head load is even more modest, with re-evaluated rigidity and roughness values, giving 2% lower pile head load (on average) than the pile conditions used by the predictors. It is
therefore concluded that it is appropriate to compare the model test results with the participant submissions.

**Centrifuge setup and experimental procedures**

The centrifuge campaign was conducted in the 1.8 m radius centrifuge at the University of Western Australia (Randolph et al. 1991). Tests were performed at an acceleration of 80 g using a strongbox of internal dimensions of 650 mm long, 390 mm wide and 325 mm height. The sample was prepared with 30 mm of sand to form a base drainage layer, on which a soil slurry (prepared with moisture content w of 140%) was placed. The sample was subsequently consolidated in-flight under an acceleration of 80 g for 20 days, creating a normally consolidated (NC) sample. Full consolidation was validated using the graphical method proposed by Asaoka (1978).

The piles were installed at 1 g to an embedment depth of 228 mm model scale (18.24 m prototype scale). This installation followed a specific process to minimise soil disturbance in the vicinity of the pile and to ensure the pile was held in vertical position while driving/drilling. To achieve this, the pile was driven by a 2-axis actuator, while the soil at the toe of the pile was removed with a drill bit connected to a rotary actuator (**Figure 2-4**). The rotary actuator was set to a speed sufficient to ensure the soil inside the pile was being displaced up, without being pushed into or pulled from the sample beneath the pile. When the pile reached the required embedment depth, both actuators where stopped, the lateral restrictions on the pile were removed, and the drill bit was extracted manually with the rotary actuator rotating in reverse. As a result of the installation process, the pile may be considered as wished-in-place for modelling purposes.

Upon installation, the centrifuge was spun to 80g, and a reconsolidation period of 1 to 2 hours (equal to at least the time the centrifuge was stopped for pile installation) was observed before the load testing started.

Once each test was finished, the centrifuge was stopped to remove the pile, and a non-instrumented tube of the same diameter and length was introduced in the hole left by the extracted pile. After the instrumented pile was cleaned, it was installed in another location in the same sample following the same procedure described above, separated at least 10 D from the locations of the previous tests and the walls of the box in the direction of loading.
Figure 2-4. Installation setup: a) Rotary actuator clamped to loading bracket; and b) Drill bit inside the pile during installation.

Soil characterisation

The soil tested is a reconstituted carbonate silt of low plasticity ($PI = 22\%$) and specific gravity ($G_s$) of 2.76. The soil classifies as a high plasticity silt (MH) under the US Classification System, although its $D_{50}$ is clay-size (0.001 mm). The profile of moisture content, obtained from core samples extracted from undisturbed locations in the box at the end of the testing, is shown in Figure 2-5a. The repeatability of the moisture content profiles indicates a uniform sample throughout the box, reassuring that the initial soil conditions for all tests were the same.

Figure 2-5. In situ testing results in prototype scale: a) Water content; b) Undrained soil strength $s_u$ measured with T-bar; and c) Degradation factor with cycle number.

The soil undrained shear strength was inferred from in flight (i.e. at 80g) T-bar penetrometer tests (Stewart and Randolph 1991), using a capacity factor $N_{T-bar} = 10.5$. The T-bar used is 5 mm in diameter and 20 mm in length. The tests comprised a monotonic push to a depth of 16-
17 m, then an extraction up to a depth of 14.5 m followed by a series of 20 cycles with a cyclic range of four T-bar diameters to assess soil sensitivity (Zhou and Randolph 2009a). Results from two T-bar tests performed at different stages during the testing campaign are shown in Figure 2-5b. The T-bar tests yield a strength gradient \( k_s \) of 1.65 kPa/m and a sensitivity \( S_t \) of 5, as observed from the degradation factor of 0.20 from Figure 2-5c, similar to those reported by Zhou et al. 2019 in the same material.

In addition to testing performed on the centrifuge sample, a number of laboratory element tests were performed on samples consolidated in tubes under a vertical pressure representing a depth of 6.0 m below the seabed. The results from CKaU simple shear tests at different strain rates and cyclic simple shear tests, are shown in Figure 2-6.

It is important to note that, while the carbonate silt was recovered from offshore, its response does not directly replicate that observed in-situ. Among other differences, in situ carbonate soil typically exhibits higher sensitivity and a different response to cyclic simple shear testing at relatively small strains (Watson et al. 2019). This results from the reconstitution process used in the laboratory, which cannot replicate the natural deposition process and hence does not capture the influence of soil fabric. It is noted (and participants were advised) that the strength gradient and sensitivity of the soil used in the prediction event appears broadly similar to deep water Gulf of Mexico clay.

![Simple shear test results showing: a) Monotonic response; b) Cycle number vs shear strain for different cyclic tests; and c) Stress vs strain for an individual test.](image)

**Figure 2-6.** Simple shear test results showing: a) Monotonic response; b) Cycle number vs shear strain for different cyclic tests; and c) Stress vs strain for an individual test.

**Load cases**

A total of four tests were performed: a monotonic pushover test and 3 two-way loading cyclic tests, with individual test sites separated by sufficient distance (at least 10 \( D \) from each other and from the walls of the box in the direction of loading) to avoid interaction effects. The details
for each test are summarised in Table 2-2, and a diagram showing the loading sequences for the cyclic tests is depicted in Figure 2-7.

Table 2-2. Test type and description.

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Test Type Description</th>
<th>Amplitude</th>
<th>Model</th>
<th>Prototype</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monotonic pushover (MT)</td>
<td>Forward push</td>
<td>1.07 D</td>
<td>14.90 mm</td>
<td>1.19 m</td>
</tr>
<tr>
<td>Cyclic test 1 (CT1)</td>
<td>Two-way displacement controlled</td>
<td>0.02 D</td>
<td>0.22 mm</td>
<td>0.02 m</td>
</tr>
<tr>
<td>Cyclic test 2 (CT2)</td>
<td>Two-way displacement controlled</td>
<td>0.10 D</td>
<td>1.37 mm</td>
<td>0.11 m</td>
</tr>
<tr>
<td>Cyclic test 3 (CT3)</td>
<td>Two-way load controlled</td>
<td>-</td>
<td>23.9 N</td>
<td>153 kN</td>
</tr>
</tbody>
</table>

Figure 2-7. Loading sequences of cyclic tests: a) CT1 ±0.02 d, triangular loading; b) CT2 ±0.10 D, triangular loading; and c) CT3 ±153 kN, sinusoidal loading.

The displacement rate for the monotonic pushover test (1 mm/s), the frequency of the cycles (1 Hz for CT1 and 0.5 Hz for CT2 and CT3, model scale) and the whole duration of each cyclic
tests (6.7 – 13.3 minutes at model scale, 29.6 – 59.3 days at prototype scale) where selected such that the soil behaviour is considered to be largely undrained – and participants were advised to ignore drainage effects in their analysis.

It is noteworthy to mention that for the cyclic test #1 (± 0.02 D) the actual amplitude achieved at the pile head was 0.018 m (instead of 0.02 m targeted). This is not expected to change the cyclic degradation behaviour significantly, although the peak load in the first cycle might be slightly lower than expected – and is to be considered when comparing tests data to predictions.

2.1.3. Prediction exercise

2.1.3.1. Output required from participants

The prediction event was advertised on the 4th International Symposium on Frontiers in Offshore Geotechnics webpage, as well on an individual basis. Participants were requested to make four predictions regarding the pile performance under the four loading conditions indicated in the previous section, although partial responses were also accepted. The output requested from participants for each test is indicated in Table 2-3. In addition, participants were asked to provide details of the method used, the parameters they adopted, and to describe what they considered to be the greatest source of uncertainty in their prediction.

<table>
<thead>
<tr>
<th>Tests</th>
<th>Output required</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monotonic pushover</td>
<td>a) The load at different stages of lateral displacement, from 0.05 $D$ to 1.07 $D$</td>
</tr>
<tr>
<td>(MT)</td>
<td>b) The bending moment profile at 0.5 $D$ and 1.0 $D$ lateral displacement</td>
</tr>
<tr>
<td>Cyclic test 1 (CT1)</td>
<td>a) The lateral force at the pile head for cycles 1, 40 and 400</td>
</tr>
<tr>
<td></td>
<td>b) The bending moment profile for cycles 1, 40 and 400 cycles</td>
</tr>
<tr>
<td>Cyclic test 2 (CT2)</td>
<td>a) The lateral force at the pile head for cycles 1, 40 and 400</td>
</tr>
<tr>
<td></td>
<td>b) The bending moment profile for cycles 1, 40 and 400 cycles</td>
</tr>
<tr>
<td>Cyclic test 3 (CT3)</td>
<td>a) The lateral displacement at the pile head for cycles 1, 40 and 400</td>
</tr>
<tr>
<td></td>
<td>b) The bending moment profile for cycles 1, 40 and 400 cycles</td>
</tr>
</tbody>
</table>

Participants were asked to provide the lateral force (or displacement) at the pile head for cycles 1, 40 and 400 for each cyclic test. It is assumed that participants provided their predictions at the peak displacement (or load) and, considering that cycles were applied without an offset (two-way loading), this peak is produced at the first quarter of the cycle - as indicated in Figure 2-7 by the red circle on each sequence plot.
2.1.3.2. Responses

A total of 29 individual predictions from 13 different countries were submitted. Some participants provided multiple submissions, meaning there were 24 individual participants, of which 15 were from industry, 6 were from academia and 3 represented collaborations between industry and academia.

Overall, 29 predictions were provided for the monotonic test, 27 predictions were provided for each of the displacement controlled cyclic tests, and 28 predictions were provided for the load controlled cyclic test.

In 20 predictions the participants modelled the soil-pile interaction using $p$-$y$ curves, whereas 8 predictions used either finite element or finite differences approaches (FE/FD), and 1 prediction used a combined finite element and $p$-$y$ approach.

The selection of the $p$-$y$ curves for the monotonic analysis varied significantly. Roughly 25% of participants that used $p$-$y$ curves indicated they used the method proposed by Jeanjean et al. 2017 in its original form (or as recommended in the draft version of API R2GEO 2020), whereas 15% used the recommendations in Jeanjean 2009. A further 25% of participants indicated that their curves were derived from the laboratory tests data provided, using the method proposed by Zhang and Andersen 2017. Roughly 30% of the predictions that used the $p$-$y$ curves were obtained using other methods proposed in the literature (API 2011; Komolafe and Aubeny 2020; Reese and Welch 1975; Wang et al. 2020; Yu et al. 2017), while the remaining participants did not specify how the curves were derived.

Regarding the $p$-$y$ curves used to predict the cyclic tests, roughly 15% of the participants reported to have used the method proposed by Zhang et al. 2017 and about 15% indicated they degraded their monotonic $p$-$y$ curves based on the cyclic contour diagrams for Drammen clay presented by Andersen 2015. Some predictors (around 15% of the $p$-$y$ predictions) used the method proposed by Komolafe and Aubeny 2020 to degrade their $p$-$y$ curves, whereas 10% of the participants constructed their cyclic $p$-$y$ curves using the method recommended by Jeanjean 2009. The remaining participants either derived specific cyclic $p$-$y$ curves or degraded their monotonic $p$-$y$ curves using other methods available in the literature (Ahmed et al. 2020; Wang et al. 2020; Yu et al. 2018; Zakeri et al. 2016), or did not specify which method they used.

There was a clear trend towards performing 3D analysis by those participants who chose to use finite elements / finite differences models, with only 25% of predictions stated to be derived
from 2D analysis (one participant did not specify which type of analysis was used). The adopted constitutive models varied between clay models (NGI APD, S-Clay-1S and UDCam-S), sand models (Ta-Ger) and silt models (PM4Silt), as well as general elastoplastic models with non-linear hardening behaviour rules. All participants indicated they calibrated their models based on the experimental data provided and/or internal databases from soils with similar characteristics.

Regarding the prediction that used a combined method, commercial finite element software was used with a constitutive model for clay (S-Clay-1S) to predict the monotonic response of the pile, with equivalent $p$-$y$ curves then back calculated from the results. These were degraded using the recommendations provided by Andersen 2015, and used to predict the cyclic behaviour of the pile. The majority of both academia and industry used recently published procedures, with only 10% of the predictions using approaches published before 2000.

In terms of modelling soil-pile cyclic behaviour, 21% of participants reported degrading the response using a cycle-by-cycle approach, whereas 27% followed cyclic strain accumulation procedures. From the information submitted, 24% derived parameters for their model from the simple shear tests results, and 21% determined the strength profile with depth based on the T-bar data provided. Additionally, 24% of participants indicated they incorporated rate effects in their analysis, with the majority stating they had increased the ultimate lateral resistance.

The greatest source of uncertainty in the prediction (reported by over 40% of the participants) was the applicability of the model selected for the particular soil and cycling loading regime. Participants stated that the model used was either calibrated from data for another soil type (kaolin, sand or GoM clay), or adapted from analysis of smaller displacements, or from cases with different cyclic loading regimes, or that they had to adapt the model in some way in order to apply it to the prediction event problem. The second greatest source of uncertainty (reported by 28% of the participants) was interpretation of the laboratory and centrifuge characterisation tests data. In this case, uncertainty related to the conversion of soil behaviour at an element level to the $p$-$y$ response, and considerations of loading rate, is thought to be critical to the predictions provided. Nonetheless, relatively few participants (28%) reported they had insufficient data to assign parameters for their models. Around a quarter of participants noted they were uncertain of the performance of their adopted method to account for the cyclic degradation of the soil, with 20% of the participants also suggesting that drainage might have occurred (introducing discrepancies between their predictions and the tests results).
2.1.3.3. Experimental and prediction results

In this section, results from the centrifuge tests (at prototype scale) are compared to the predictions. No manipulation was performed to the predictions provided, with the exception of the following:

- When the depth provided was not aligned with the depth requested, a linear interpolation was performed between the closest neighbouring depths.
- When the maximum bending moment of the profile provided was negative, the convention was modified to exhibit a positive peak bending moment.

Monotonic test

Figure 2-8 presents the load displacement curves for the centrifuge results and all predictions. The bending moment profiles for a lateral displacement of 0.5 $D$ and 1.0 $D$ at the pile head are presented in Figure 2-9. Note that for 1.0 $D$, one prediction is only partially shown as it lays outside the margins selected for the figure. As will be discussed in more detail in the statistical analysis and discussion, the predictions generally indicate higher pile head load for a particular displacement level, as well as higher peak bending moment.

![Figure 2-8. Monotonic load-displacement curves at the pile head.](image-url)
Figure 2.9. Monotonic bending moment profiles for 0.5D and 1.0D pile head displacement.

Cyclic tests
The pile head response against cycle number is presented in Figure 2.10, while the bending moment profiles (for N = 1 and 400) are presented in Figure 2.11.

Figure 2.10. Pile head response against cycles number (data from N=1, 40 and 400).
2.1.4. Statistical analysis and observations

2.1.4.1. Statistical analysis of the monotonic test

The pile head load predictions for monotonic displacements of 0.1 D and 1.0 D are presented using a bar plot in Figure 2-12a, along with the median of the predictions and the measured load. Note that individual predictors are designated by an individual number, allowing changes in position (relative to other predictions) to be tracked.

Roughly 70% of the submissions overpredicted the load, with the median values up to 22% higher than the measured load. For 0.1 D displacement at the pile head, the median value of pile head load from both p-y and finite element / finite difference (FE/FD) predictions are similar. For higher pile head displacements, however, the median value of the load predicted from FE/FD analysis jumps to 44% higher than the measured load. This suggests either that...
the soil constitutive models require better calibration for large displacement cases, or that other issues such as mesh distortion / mesh lock are influencing the results.

In order to identify trends representing the majority of the predictions, outliers were removed to obtain the coefficient of variation (COV) of the data. Outliers were identified, for all tests and stages, as those values outside the average value +/- two standard deviations. The impact of these outliers on the values of COV is illustrated in Figure 2-13, along with the median of the predictions normalised by the measured load.

Figure 2-13 indicates that the load predictions are best at a displacement of 0.40 $D$, for which the COV and normalised median pile head load are 0.20 and 1.11, respectively. The comparison is worse (higher COV and normalised pile head load) for both smaller and large displacements.

Figure 2-14 shows the load displacement curves grouped according to the $p$-$y$ methods that most participants used for their prediction. Figure 2-14a shows the predictions that used the method proposed by Zhang and Andersen 2017 for $p$-$y$ scaling from simple shear test results, combined with other methods in the literature to obtain the ultimate soil pressure ($P_u$). Two participants indicated they obtained the ultimate soil pressure with the guidelines provided in Jeanjean et al. 2017, whereas another participant indicated they used Murff and Hamilton 1993. The latter lies slightly below the test results, whereas the former two lie on the high side, possibly indicating that the selection of the $N_p$ value with depth plays a role as important as the shape of the $p$-$y$ curve on the overall load-displacement behaviour.
Figure 2-12. Observed versus predicted pile head load and peak bending moment for: a) Monotonic test; and b) Cyclic test 1 (CT1).
Figure 2-13. Coefficient of variation and normalised median pile head load vs displacement.

Figure 2-14. Predicted monotonic load-displacement curves at the pile head grouped by the most used methods: a) Predictions using p-y scaling from simple shear results; and b) Predictions using Jeanjean 2009 or Jeanjean et al. 2017.

The predictions that used the p-y methods based on Jeanjean 2009 or Jeanjean et al. 2017, either as published originally or as the modified version in the draft ISO guidelines (currently under review by member countries), are shown in Figure 2-14b. The predictions shown in this figure generate p-y curves based on parameters selected by the practitioner. Although the methods suggest a preferred laboratory test to determine the input parameters, the selection process is affected by the judgement of the practitioner in regards rate effects and past experience on similar soils.
In both Figure 2-14a and Figure 2-14b it is observed that the predictions that incorporated rate effects using the methods mentioned above (or at least where this was reported) all lie considerably higher than the test results. Here, the participants used two different approaches: they either increased the undrained shear strength profile by a percentage (based on the rate of the test compared to the rate of the provided laboratory tests), or they adjusted $P_u$ by some multiplier.

Peak bending moments for monotonic pile head displacements of $0.5 \, D$ and $1.0 \, D$ are presented in Figure 2-12a, along with the median of the predictions and the measured value. Around 62% of submissions overpredict the peak bending moment, although the median value is only 6% higher than that measured and the associated COV (excluding outliers) is only 0.17. The median depth at which the peak bending moment is estimated is 10.5 m (Figure 2-15a). This is close to the depth of 10 m obtained from centrifuge testing, noting that the range in peak bending moment depth from all predictions is between 7.0 m and 13.0 m, with a COV of 0.13.

Figure 2-15. Observed versus predicted depth to peak bending moment: a) Monotonic test, b) Cyclic test 1 (CT1); c) Cyclic test 2 (CT2); and d) Cyclic test 3 (CT3).
At 1.0\(d\) pile head displacement, however, 72\% of the submissions overpredicted the peak bending moment, with the median value of all predictions being 19\% higher than the measured value, and the COV increasing slightly to 0.20. The median depth to peak bending moment increases to 12.0 m, compared to a test measurement of 11.0 m. The range of depths below the seabed to peak bending moment ranges between 5.0 m and 14.0 m, with a COV of 0.11.

While it appears that the prediction of peak bending moment increases in scatter and degree of overprediction with increasing displacement, the depth to the peak is predicted accurately.

### 2.1.4.2. Statistical analysis of the cyclic tests

**Cyclic test: displacement controlled CT1 (± 0.02 \(D\))**

The comparison between prediction and model testing for CT1 is summarised in **Table 2-4**, and in **Figure 2-12b** and **Figure 2-15b**. It can be observed that roughly 80\% of the predictors overestimated the pile head load, with significant scatter in the data as demonstrated by the high reported COV. However, it is also noted that the magnitude of overprediction reduces with cycle number – from 46\% at \(N = 1\) to only 19\% at \(N = 400\).

**Table 2-4. Summary of results for CT1.**

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Pile head load</th>
<th>Peak bending moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Predicted / Measured</td>
<td>COV</td>
</tr>
<tr>
<td>All</td>
<td>(p-y)</td>
<td>FE/FD</td>
</tr>
<tr>
<td>1</td>
<td>1.46</td>
<td>1.28</td>
</tr>
<tr>
<td>40</td>
<td>1.24</td>
<td>1.11</td>
</tr>
<tr>
<td>400</td>
<td>1.19</td>
<td>1.16</td>
</tr>
</tbody>
</table>

With respect to the prediction method used, **Figure 2-12b** and **Table 2-4** indicate that for \(N=1\) the \(p-y\) methods outperformed the FE/FD methods, which is consistent with the monotonic push over test. The median value of the head load from \(p-y\) based predictions is 28\% higher than the measured value, versus 72\% higher for FE/FD based predictions. As also observed for the monotonic test, the predictions improve with increasing cycle numbers.

The majority of predictors (over 90\% for \(N = 1\) reducing to around 60\% for \(N = 400\)) over predicted the peak bending moment, with the median value reducing from 1.29 to 1.03 times the measured value as \(N\) increases from 1 to 400. While the predictors appear to have estimated the peak bending moment somewhat better than the pile head load, the COV values show a
similar scatter for both. As for the monotonic test, the predictors accurately estimated the depth to the peak bending moment (Figure 2-15b) at all cycles except for N=400.

**Cyclic test: displacement controlled CT2 (± 0.10 d)**

The results of the statistical analysis on CT2 are summarised in Table 2-5, and in Figure 2-15c and Figure 2-16a. More than 70% of the predictors overestimated the pile head load, with high scatter as demonstrated through the COV. The overprediction is lowest for low numbers of cycles but increases for N = 400.

**Table 2-5. Summary of results for CT2.**

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Pile head load</th>
<th>COV</th>
<th>Peak bending moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>All</td>
<td>All No outliers</td>
</tr>
<tr>
<td></td>
<td>Predicted / Measured</td>
<td>p-y</td>
<td>FE/FD All No outliers</td>
</tr>
<tr>
<td>1</td>
<td>1.16</td>
<td>1.13</td>
<td>1.32</td>
</tr>
<tr>
<td>40</td>
<td>1.16</td>
<td>1.03</td>
<td>1.53</td>
</tr>
<tr>
<td>400</td>
<td>1.24</td>
<td>1.14</td>
<td>1.64</td>
</tr>
</tbody>
</table>

Similar to CT1, the median pile head load for N = 1 determined using p-y methods is closer to the measured value than that derived using finite element / finite difference methods. As the number of cycles increases, the median value from p-y analysis remains low, while that obtained from FE/FD analysis increases up to 1.64 times the measured value - indicating that less degradation was considered in the FE/FD models.

Also consistent with CT1 is the observation that the majority of predictors overestimated the peak bending moment. However, unlike CT1, the median prediction of the depth to peak bending moment was 1 m higher than the measured depth for all cycles.
Figure 2-16. Observed versus predicted pile head load / displacement and peak bending moment predictions for a) cyclic test 2 (CT2), and b) cyclic test 3 (CT3).
The results of the statistical analysis on the load controlled cyclic test are summarised in Table 2-6, and in Figure 2-15d and Figure 2-16b. In this case, it can be seen that roughly half of the predictors accurately predicted the pile head displacement, but with relatively high scatter in the submissions (and high COV values). The comparison appears to get worse with an increase on the cycle number, leading to underprediction of displacement. This indicates that cyclic degradation is somewhat underpredicted relative to the test. When comparing prediction methods, the $p$-$y$ method seems to (on average) overpredict pile head displacement, while the FE/FD method seems to underpredict it.

**Table 2-6. Summary of results for CT3.**

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Pile head displacement</th>
<th>Peak bending moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Predicted / Measured</td>
<td>COV</td>
</tr>
<tr>
<td></td>
<td>All $p$-$y$ FE/FD</td>
<td>All No outliers</td>
</tr>
<tr>
<td>1</td>
<td>1.02 1.05 0.88</td>
<td>1.57 0.40</td>
</tr>
<tr>
<td>40</td>
<td>0.94 1.05 0.81</td>
<td>1.79 0.44</td>
</tr>
<tr>
<td>400</td>
<td>0.90 0.98 0.84</td>
<td>1.79 0.47</td>
</tr>
</tbody>
</table>

Regarding peak bending moment, the median value is close to the measured value, with low COV implying that load-controlled cyclic tests can be more accurately than displacement-controlled tests. The median depth to the peak bending moment is also well predicted.

### 2.1.5. Discussion and implications

To understand better how the selection of parameters and analysis method influence the predicted soil-conductor behaviour, a parametric analysis was performed using the software LAP (Doherty 2017) and considering variations in $N_p$, adopted $p$-$y$ curve shape and $s_u$ profile. In this case, the pile-soil interface was assumed partially rough ($\alpha = 0.5$), and the pile stiffness was set as 889 MN.m$^2$ (consistent with the measured values for the conductor). The load was applied at the hinge level, and the prototype pile length and outer diameter used were those specified in Table 2-1. The cases studied are reported in Table 2-7. The undrained shear strength gradient ($k_{su}$) shown in Table 2-1 was determined from the average of the strength from the CK$_0$U simple shear tests at the slow rate, divided by the representative depth of the laboratory tests (6.0 m), and assuming a seabed strength of 0 kPa. It is noted that the T-bar
strength gradient (1.65 kPa/m) yields a very similar value as the strength gradient calculated from the simple shear tests at slow rate (1.66 kPa/m) – despite the T-bars being performed at (considerably) higher strain rate than the slow simple shear tests. While not fully understood, this is thought to be attributed to partial soil softening induced during initial penetration of the T-bar (Einav and Randolph 2005; Zhou and Randolph 2009a;b) offsetting the increase due to loading rate.

Table 2-7. Data for parametric analysis.

<table>
<thead>
<tr>
<th>Case</th>
<th>Undrained shear strength gradient, $k_u$ (kPa/m)</th>
<th>Rate effects</th>
<th>Shape of $p$-$y$ curve</th>
<th>Lateral bearing capacity factor, $N_p$ (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>Not considered</td>
<td>From Jeanjean et al 2017, recommended curve for $s_u$&lt;100 kPa</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Average $s_u$ from simple shear tests at slow rate.</td>
<td>49.4% increase (based on 17.3% increase per log cycle from SS tests at slow and fast rate).</td>
<td>From Jeanjean et al 2017, no gap assumed</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Not considered</td>
<td></td>
<td>Scaled from simple shear test at slow rate using Jeanjean et al 2017</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Not considered</td>
<td></td>
<td>From Jeanjean et al 2017, recommended curve for $s_u$&lt;100 kPa</td>
<td>From Murff and Hamilton 1993</td>
</tr>
</tbody>
</table>

The $p$-$y$ curves used in the parametric analysis are shown in Figure 2-17, and were either scaled from the simple shear test at slow rate (Zhang and Andersen 2017, Jeanjean et al. 2017) or from the literature (Jeanjean et al. 2017). The head loads and bending moments using the $p$-$y$ curve scaling from Zhang and Andersen 2017 are observed to be very similar to the ones using the scaling from Jeanjean et al. 2017, and hence only the results using Jeanjean et al. 2017 are provided in this study (Case 3). The results of the parametric analysis are shown in Figure 2-18 and Figure 2-19 in terms of load displacement curves and bending moment profiles, respectively.
At 1.0 \( d \) displacement of the pile head, Case 2 and Case 3 estimated head loads above the measured in the test by 40% and 10%, respectively. This over-estimation is also observed in the bending moment profiles for the same level of displacement (Figure 2-19) in which the estimated peak bending moment is about 35% and 10% higher than the measured values, for Case 2 and Case 3, respectively. The head load from Case 1 was within 5% of the measured load, and the peak bending moment within 4%. The closest match to the experimental data was also obtained from Case 4, yielding a head load and peak bending moment within 2% of the measured values.
A head displacement of 1.0 $D$ is considered more relevant to pile foundations than it is to conductor fatigue life estimation.

Figure 2-19. Bending moment profiles from parametric analysis for a range of pile head displacements.
For displacements less than 0.1 $D$ (relevant for conductors) the measured load-displacement lies between the analysed cases, with Case 2 and Case 3 predicting a stiffer response, and Case 1 and Case 4 predicting a softer response.

As observed in the bending moment profile from Figure 2-19, at 0.02 $D$ and 0.10 $D$ pile head displacement, stiffer $p$-$y$ curves produce bending moment profiles whose peak is above the measured data. This is particularly true for Case 3, where the peak bending moment at 0.02 D head displacement is roughly 45% higher than the measured value. Overly stiff $p$-$y$ curves could lead to an underestimation of the bending stresses on the lower components, and an overestimation on the bending stresses in the upper components – which may be important for design, depending on where structural joints (along the conductor) are located.

Overall, the parametric study shows that selection of the undrained strength profile and stiffness of $p$-$y$ curves requires careful consideration. Of importance is the observation that the range of predictions (presented earlier) is considerably wider than the range of results from the parametric study – highlighting that judgement and experience plays a large role in the final results – with the same input data potentially leading to a large range in prediction.

2.1.6. Conclusions

This paper presents the results of a prediction exercise undertaken by the National Geotechnical Centrifuge Facility at the University of Western Australia. The problem chosen is the response of a model conductor in normally consolidated fine-grained soil, subjected to sequences of monotonic and cyclic loading in a centrifuge.

Across all tests, the participants tend to predict higher pile head capacity / stiffer response than seen in the experimental observations, albeit with some of the provided predictions being (very) close to the model response. The median of the predicted pile head load for the monotonic test is up to 46% higher than the measured value, and appears best at displacements between 0.2 $D$ and 0.7 $D$. Similarly, the peak bending moment is best predicted for low displacement, with the depth to reach the peak bending moment well predicted. Under cyclic loading, participants were able to predict the response more accurately during the load-controlled test, and in all cases the $p$-$y$ approach outperforms the FE/FD approach. A closer look at the results provides some potential avenues for improvement of conductor and pile design. Note that these observations are based on the median values of the predictions, and do not address individual cases:
In all cases, the prediction of the peak bending moment is more accurate than the prediction of the head load, with this remaining the case even at high cycle numbers.

The better performance of the p-y approach (constituting 20 of the 29 predictions) is a testimony of the experience gathered by the geotechnical community over the last 3 decades, and of the general robustness of this approach. Further improvements will likely rely on more accurate estimation of p-y curves for given materials.

While limited in number, the accuracy of the FE/FD approach reduces with increasing displacement. It is uncertain whether this is associated with numerical artefacts (i.e. mesh locking) or calibration of the constitutive models. Regarding the cyclic tests, the FE/FD approach seems to decrease in accuracy with cycle number for the load-controlled test and the displacement-controlled test at ±0.1 \( d \), but provides better predictions with cycle number for the displacement-controlled test at ±0.02 \( D \). While not specifically explored in this event, it is considered likely that FE/FD approaches are more time consuming than p-y analysis – suggesting the latter is a better tool for these design problems.

Care must be exercised when incorporating judgement in the analysis. It is noted that several participants chose to include rate effects – consistent with the parametric analysis, it appears that including rate effects leads to an overestimation of stiffness, resulting in predictions with higher pile head capacity / stiffer response – although other factors may also have influenced the results provided.

The significance of overpredicting pile head loads or peak bending moments is likely to be specific to the application. Use of an overly stiff p-y curve in conductors analysis may lead to an overprediction of the bending moment in the upper section, which is potentially conservative (and possibly cost prohibitive) for fatigue assessments; while at the same time may underpredict the bending moment over the lower section and lead to an unconservative outcome.

Given this was a highly constrained problem, using a uniform and relatively well characterised soil sample, and well controlled installation methodology, the level of scatter in the predictions indicates there is room for improvement – particularly in regard to recommended practice. The professional judgement of each engineer plays an important role in selecting the soil parameters for calculations, and this is reflected in the variability of the predictions. This variability is
explored in the statistical analysis included in this paper, and may help assign uncertainty associated with engineering judgment in design of conductors and piles.

Acknowledgements

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CHAPTER 3 – EXPERIMENTAL OBSERVATIONS

Prologue

In the previous chapter (Chapter 2) a justification for the development of improved $p$-$y$ curves that reflect the changes produced by cyclic loading was presented, as well as particular aspects that could potentially impact fatigue life estimation of the system.

This chapter (Chapter 3) explores the key features of different cyclic loading conditions that impact the soil-conductor lateral behaviour. This is performed by analysing the experimental results from different tests comprising centrifuge, laboratory and $p$-$y$ apparatus tests.

The Chapter is composed of three conference papers

The first paper was accepted for presentation at the ISFOG 2020 conference in Austin (pending presentation) and is titled: “Key features impacting soil-conductor lateral behaviour as illustrated by centrifuge tests” (Guevara et al., 2020). It analyses the loading aspects that influence the fully remoulded undrained lateral stiffness from centrifuge tests using a rigid hollow pile in reconstituted normally consolidated carbonate silt. Features such as changes in fully remoulded undrained stiffness, due to load history and pore pressure dissipation during cycling or non-cycling periods, are observed. A preliminary link between laboratory element tests and fully remoulded stiffness is established, albeit not considering load-history effects.

The second paper was accepted for presentation at the VSOE 2021 conference in Ho-Chi-Minh (pending presentation) and is titled: “Evolving soil-conductor stiffness due to multiple-episode cyclic loading” (Guevara et al., 2021). It explores the differences in fully remoulded undrained stiffness of two reconstituted normally consolidated materials (carbonate silt and kaolin clay) when subjected to the same cyclic loading sequences. It was found that the carbonate silt exhibits less immediate recovery in stiffness after large amplitude cycling than the kaolin clay, as well as less stiffening effect due to pore pressure dissipation periods between cycling. It was also observed that one-way and two-way cycling did not yield significantly different remoulded stiffness values.

The last paper was accepted for presentation at the ICSMGE 2021 conference in Sydney (pending presentation) and is titled: “A comparison study of soil-conductor stiffness from
centrifuge and laboratory testing” (Guevara et al., 2021). It presents a comparison between results from the centrifuge testing campaigns and tests performed with the BP-NGI p-y apparatus, on reconstituted normally consolidated carbonate silt and kaolin clay. It was observed that the fully remoulded undrained stiffness from both type of tests agreed when normalising by the measured monotonic capacity, without considering load-history effects. The link between stiffness from soil laboratory tests and centrifuge tests explored in the first paper is presented in this one as well, with the addition of data from the p-y apparatus, in both carbonate silt and kaolin clay.

The observations from these tests were used to develop a method to generate p-y curves that account for load history, presented in Chapter 4, and the data from the tests was used to calibrate the method, as further explained in Chapter 5.
3.1. Key features impacting soil-conductor lateral behaviour as illustrated by centrifuge tests

Abstract

Conductors are a type of pile used during subsea drilling operations to prevent hole collapse and to provide axial support to the well. The response of the conductor to lateral movement, as induced by environmental conditions, contributes to the assessment of fatigue damage of the entire wellhead system. Such assessment requires soil-structure interaction analysis, typically performed by modelling the soil-conductor lateral behaviour as non-linear springs called $p-y$ curves. While bespoke approaches do exist, current industry practice often involves the use of $p-y$ curves given in API RP2GEO, which were originally developed for foundation piles. Recent studies have shown that these curves do not adequately capture the soil-conductor response, especially at small lateral displacements. In addition, no account is given to load-history effects. This paper presents results from centrifuge testing of a rigid length of conductor installed in reconstituted samples of carbonate silt and subject to cycles of lateral displacement, with focus on identifying key features that influence soil-conductor behaviour. The results show that the degraded secant stiffness is impacted by load history – for example, after applying cycles of large amplitude displacement, the secant stiffness at smaller amplitude cycling will be significantly lower than if it had not previously experienced the more onerous loading. Furthermore, pore pressure dissipation between or during cyclic events can result in secant stiffness increasing. The results presented in this paper are part of an ongoing research project, aimed at improving fatigue design of subsea wells.

3.1.1. Introduction

Conductors are a critical part of top tensioned riser systems as used for drilling wells for offshore oil and gas exploration and extraction. A typical system is illustrated in Figure 3-1, showing the riser connected to the drill floor by an upper flexible joint (UFJ) and pulled at the top by a tensioning system on the drilling vessel. Closer to seabed, the riser is connected to a lower flexible joint (LFJ) attached to the lower packages, which are heavy components used to support safe drilling. These lower packages are positioned above the wellhead system, which itself is joined to the conductor and casing system.
During drilling, the vessel may move in response to environmental loads, and this motion translates to the components at the seabed (including the conductor). Under normal operations, the amount of (cyclic) lateral movement of the conductor is small. However, repeated cycling can produce stress cycles at specific locations – “hot spots” – where fatigue may become critical. In stiff soils, the hot spot will typically be located above the seabed; while for soft soils the hot spot may be located below the seabed. Assessing fatigue utilisation of the system is critical for safe design, and requires modelling of soil-conductor behaviour.

Soil-conductor interaction can be analysed using the approaches developed for pile analysis: the conductor is represented as an elastic beam which is restrained laterally by non-linear Winkler springs distributed along its length. Lateral and/or moment loads act at the top of the conductor, and the functions that define the non-linear load-displacement response along the pile are called $p$-$y$ curves. The curves are defined in terms of the resistance of the soil per unit length of pile ($p$) in response to a lateral displacement ($y$), often defined in terms of secant stiffness – which is the gradient of the line that joins the origin with a specific point of the curve.

![Figure 3-1. Schematic of a drilling riser system and soil modelling.](image)

The current API RP 2GEO guidelines (API, 2014) recommend monotonic $p$-$y$ curves that are based on the results of pushover testing performed on piles. It is generally accepted that the API curves underpredict lateral stiffness at modest displacement levels. Given the typical operational displacements at the wellhead of a riser system range between 2.5% and
10% of the conductor diameter (Zakeri et al., 2015), this is particularly important for soil-conductor analysis. Jeanjean (2009) report on a programme of centrifuge experiments and finite element analysis and proposed an alternate monotonic \( p-y \) relationship. This was further developed (Jeanjean et al., 2017) by linking the curve to simple shear testing via a scaling method proposed by Zhang and Andersen (2017).

While the monotonic \( p-y \) curves provide a backbone response, the behaviour of a riser system is inherently cyclic in nature. More recent studies (Zakeri et al., 2016, 2019) propose a method to account for the effects of cyclic loading. The study found that after 50-100 cycles of lateral loading the \( p-y \) response reaches a “steady state stiffness”, beyond which no further degradation occurred – and provide recommendations for design. Test results have been presented for a range of soils, although no similar studies have been performed on the carbonate soils found in the North West Region of Australia.

The studies by Zakeri et al. (2016, 2019) do not address load history or potential gain in stiffness due to pore pressure dissipation. Recent experimental testing has shown that these effects may be significant for a range of scenarios (e.g. White and Hodder, 2010; Zhang et al., 2011; Zhou et al., 2019a, 2019b). Moreover, the improvements of applying a load history, or ‘whole life’, approach in engineering design have been illustrated by Gourvenec et al. (2017), for the case of a fixed subsea mudmat. In that particular study, the foundation area could be reduced by half if the effect of consolidation due to self-weight or preloading was accounted for. Furthermore, the area of the foundation could be reduced even more if the foundation was designed to slide and the effects of load history were considered in the analysis.

Specifically for conductors, Doherty et al. (2019) showed how the \( p-y \) response (at a particular value of \( y \)) is softer when the conductor was previously subjected to higher amplitude cycling. The study also showed how consolidation between cyclic packages can lead to increased stiffness. Therefore, the unresolved questions are: Could the change in \( p-y \) stiffness be relevant for fatigue life estimation of conductors? What factors produce changes in the soil-conductor stiffness?

This paper presents results from centrifuge testing of a rigid length of conductor that was cyclically displaced in reconstituted samples of carbonate silt, focusing on key features observed to affect the response of the soil during the testing.
3.1.2. Test setup

Two centrifuge testing campaigns were conducted in the 1.8 m radius beam centrifuge at the University of Western Australia. The first campaign was conducted on both kaolin clay and carbonate silt, while the second was conducted on carbonate silt only. This paper presents results from the carbonate silt (CS) only.

The carbonate silt has low plasticity \( (PI \sim 20\%) \) and specific gravity \( (G_s) \) of 2.76. Tests were performed at an acceleration of 40 g using a strongbox of internal dimensions of 650 mm long, 390 mm wide and 325 mm height. Samples were prepared with 30 mm of sand to form a base drainage layer, on which the carbonate silt slurry (prepared with moisture content \( w \) of 140\%) was placed. The sample was then spun for \( \sim 5 \) days to achieve full consolidation. The final effective unit weight was determined based on the moisture content from core samples extracted from undisturbed locations within each sample, and the specific gravity of the soil grains. The moisture content profile is shown in Figure 3-2a.

![Figure 3-2a](image)

**Figure 3-2.** Sample properties: (a) Moisture content profile, (b) Typical undrained shear strength profile from T-bar tests.

Soil strength \( (s_u) \) was determined via a model T-bar penetrometer using a capacity factor \( N_{T-bar} = 10.5 \) (Stewart and Randolph, 1991), with tests performed on each day of testing to track any change in strength. A typical strength profile is shown in Figure 3-2b. The T-bar dimensions were 5 mm in diameter and 20 mm in length, and the tests were performed at a rate of 3 mm/s (to ensure undrained conditions). All tests included a cyclic stage to investigate remoulding, showing a sensitivity of 3.3.
To complement the T-bar tests, a number of soil element (laboratory) tests were performed on carbonate silt samples consolidated in tubes under a vertical pressure representing an embedment depth of 6.0 m. Results from resonant column and monotonic simple shear tests are presented.

The model pile was 3D printed in stainless steel and its diameter and wall thickness were 19.5 mm and 0.95 mm respectively \((D/t_w \text{ ratio} = 20.5)\), representing a prototype conductor of 780 mm (30 inch) diameter and 38 mm (1.5 inch) wall thickness. It was instrumented with two pore pressure transducers at different locations (30 mm and 60 mm from the pile tip). The pile was connected to a bending leg, which is used to determine the lateral load \(F_h\) applied to the conductor, based on the difference between moments measured by strain gauges located a specific distance from each other. The displacement and rotation of the pile were measured using two lasers and a flat plate bracket as a target. For the first campaign, an extension was used between the conductor and the bending leg to ensure that the deeper embedment would be achieved. The diagram of the setup for both campaigns is shown in Figure 3-3.

![Figure 3-3. Diagram of test setup: (a) First campaign setup; (b) Second campaign setup.](image)

The hollow piles were installed in flight. A total of 8 tests were performed per sample, with individual test sites separated by sufficient distance to avoid interaction effects (at least 6 \(d\) from each other and from the walls of the box in the direction of loading). Two different embedment depths of 4.5 \(D\) and 6 \(D\), where \(d\) is the pile diameter, were adopted for the first set of tests, while only one embedment depth (4.5 \(D\)) was used in the second set of tests.
No plugging was anticipated for these embedment depths. A wait period of 2.15 hours (model time) separated the installation process from the start of lateral loading, to allow excess pore pressure generated during installation to dissipate. The typical dissipation percentage was between 80% and 90% and the representative $c_h$ of the sample is 4 m$^2$/year. The wait period was initially estimated based on the solution given by Randolph and Wroth (1979) and subsequently confirmed by monitoring of the pore pressure transducers.

All the tests were displacement controlled at the actuator level. The pile was practically rigid for the range of loads applied. However, there was a very small amount of compliance in the connection of the pile and the bending leg. This was accounted for by developing a relationship between the applied load and the pile rotation (measured with the two lasers), which was then used to determine the pile lateral displacement at 50% of the penetration depth.

### 3.1.3. Testing programme

During the first campaign, most testing comprised episodes of displacement-controlled cyclic motion followed by a pore pressure dissipation (i.e. consolidation) period. The test sequences and descriptions are shown in Table 3-1. The post-episode pore pressure dissipation was assessed based on the Osman and Randolph (2012) solution for consolidation around a laterally loaded pile, and time periods were selected to achieve around 75% dissipation. The Osman and Randolph (2012) solution was also used to determine the time frame for which an episode could be considered effectively undrained ($\leq 25\%$ dissipation), and when the pore pressure dissipation effects would start to influence the response during an episode.
Table 3-1. Test type and description.

<table>
<thead>
<tr>
<th>Type 1 (Campaign 1): Monotonic test.</th>
<th>Test to determine lateral capacity ($F_{h,ult}$) for comparison to cyclic testing.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 2 (Campaign 1): Small to large amplitude test.</td>
<td>Continuous cycling via packets of 150 cycles at constant peak-to-peak (P2P) amplitudes increasing from 0.01D to 0.16D, followed by cyclic loading around an offset displacement. The pile was then “unloaded” via matching cyclic packets.</td>
</tr>
<tr>
<td>Type 3 (Campaign 1): Undrained 2-way loading.</td>
<td>Episodes of 2 way cyclic loading separated by periods of pore pressure dissipation. Each episode consisted on 5 individual packets of 100 cycles each, ranging from 0.04D to 0.16D.</td>
</tr>
<tr>
<td>Type 4 (Campaign 1): Undrained 1-way loading.</td>
<td>Episodes of 1 way cyclic loading, using P2P amplitudes and pore pressure dissipation periods comparable to Type 3.</td>
</tr>
<tr>
<td>Type 5 (Campaign 2): Impact of higher cycling.</td>
<td>Types 5.1, 5.2 and 5.3 represent individual tests, each performed in new (undisturbed) test locations. Each test comprises 400 cycles at 0.08D, with a higher amplitude (0.12D) event added at the middle. The number of high amplitude cycles varied between 2 (Type 5.1), 20 (Type 5.2) and 200 (Type 5.3).</td>
</tr>
<tr>
<td>Type 6 (Campaign 2): Undrained 2-way loading.</td>
<td>Episodes of 200 cycles of 0.12D (2-way P2P amplitude) with pore pressure dissipation periods in between.</td>
</tr>
<tr>
<td>Type 7 (Campaign 2): Long term cyclic response.</td>
<td>Comprised two packages of 2 way loading at 0.12D for 10,000 cycles, with pore pressure dissipation in between.</td>
</tr>
</tbody>
</table>

3.1.4. Results and discussion

Results from the centrifuge testing are examined via changes in a normalised cyclic secant stiffness ($K$). As will be shown, the secant stiffness varied with loading and dissipation
histories, and hence there is no “steady state” value. Accordingly, we have reported $K$ at different times, defined as:

$$K = \frac{\Delta F_h}{\Delta y/D}$$

where $\Delta F_h$ is the cyclic range of horizontal load (representing the integrated response of the distributed soil pressure, $p$, along the length of the pile); $F_{h,ult}$ is the measured monotonic lateral capacity from Test 1; $D$ is the pile diameter and $\Delta y$ is the range of lateral displacement at mid embedment depth of the conductor.

A plot of $F_h$ and displacement at the actuator level vs time can be seen in Figure 3-4a, for the first 2 episodes of test Type 3 at 4.5*D embedment depth. The figure also shows the first and last cycle of packets 2, 3 and 4 highlighted in red, green and magenta, respectively, for the first episode of the test. The shear vs displacement at mid-embedment depth of these highlighted cycles can be observed in Figure 3-4b.

![Figure 3-4](image)

**Figure 3-4.** Data from test Type 3 at 4.5*D embedment depth: a) Head load and displacement at the actuator level vs time (model scale); b) Head load vs displacement at mid-embedment level for the start and end cycle of the second, third and fourth packets of loading (model scale).

3.1.4.1. **Stiffness degradation under cyclic loading**

Figure 3-5 (left and bottom axis) presents $K$ against $\Delta y/2D$, where the reported $K$ is from the final cycle of each packet of the increasing amplitude part of the first episode of test Type 2, 3 and 4. The results show that $K$ decreased with increasing $\Delta y/2D$, consistent with previous studies. Also shown is the secant shear modulus (normalised by $G_{max}$) of the
carbonate silt as obtained from resonant column and monotonic simple shear (fast and slow) testing against shear strain scaled by a factor of 1.6 (right and bottom axis).

![Figure 3-5](image)

**Figure 3-5.** Normalised secant stiffness ($K$) vs cyclic amplitude from packages of increasing amplitude; and comparison with response from soil element tests.

The depth averaged $G_{max}$ for 6D and 4.5 D embedment values are 4.73 MPa and 3.55 MPa, respectively. These were estimated by extrapolating the results from resonant column and bender element tests to the mean effective stress at mid-pile level, and assuming a $K_0$=0.6.

There are clear similarities in the data. This is subject of ongoing work, and broadly consistent with approaches such as the $p$-$y$ scaling framework proposed by Zhang and Andersen (2017), and suggests that it may be possible to evaluate the cyclic $p$-$y$ response of conductors from soil element testing.

3.1.4.2. The effect of prior cycling on stiffness

Test Type 2, 3 and 4 also include stages where the cyclic amplitude was decreased (after being fully degraded at a higher cyclic displacement level), and the results are shown on **Figure 3-6**.

The results show that the degraded stiffness does not recover when cycling at lower amplitudes. This has the practical implication that once subjected to sufficient cycles at a higher amplitude, subsequent cycles with lower amplitude will have a lower stiffness than predicted from a “steady state” model based on tests with no previous higher amplitude cycles.
Other tests provide an indication of the loading required to cause degradation that will influence subsequent cycles. This is a subject of ongoing study, but some initial clues are provided from test Type 5.1, 5.2 and 5.3 as shown in Figure 3-7. Each test comprises 400 cycles at 0.08 $D$, with a number of high amplitude cycles (0.12 $D$) added at the middle. The number of larger ("disrupting") cycles was varied between 2, 20 and 200. The results show that changes in stiffness following the "disrupting" high amplitude packet depend on the number of high amplitude cycles – for a packet of 200 high amplitude cycles, the stiffness for subsequent smaller amplitude cycles does not return to its original stiffness, while fewer cycles did not lead to the same level of degradation. This suggests that the cyclic stiffness at a given amplitude depends on cyclic loading history and is a function of the cumulated displacement, and the amplitude of the displacements.

3.1.4.3. Recovery in stiffness due to dissipation of excess pore pressure

The regain in stiffness due to pore pressure dissipation is demonstrated from the Type 6 results, as shown in Figure 3-8. A total of 8 individual episodes of 200 cycles (at 0.12 $D$) were applied, with pore pressure dissipation between each episode resulting in a reduction in excess pore pressure of approximately 75%. During episodes of cyclic motion, the stiffness decreases due to the build-up of excess pore pressures – but after a period of reconsolidation the subsequent stiffness is higher. At the end of episode 8, the degraded (final cycle) stiffness ($K = 12$) was more than double that seen in the first episode ($K = 5$).
Figure 3-7. Normalised secant stiffness (K) vs Number of cycles for tests of a packet of 0.05d amplitude disrupted by 2 (Test 5.3), 20 (Test 5.1) and 200 (Test 5.3) cycles of 0.08d.

Figure 3-8. Normalised secant stiffness vs number of cycles for episodes of 200 cycles of amplitude 0.08d at mid-pile level separated by a consolidation period.

An alternate way to explore the change in stiffness due to pore pressure dissipation is to track changes in stiffness under long term cyclic loading. Results from a Type 7 test are plotted on Figure 3-9, and show that after the initial period of (undrained) softening, the stiffness begins to increase. A period of dissipation leads initially to a stronger gain in stiffness, which quickly degrades back to the long-term rising trend. At the end of cycling, the stiffness is roughly half the initial (monotonic) stiffness and 4 times greater than the minimum degraded value. The results are plotted against dimensionless time to make them comparable to different soil types with different horizontal coefficients of consolidation.
For soils with a low $c_h$ this regain in strength might be less significant for the drilling operation itself. However, soils with higher $c_h$ might experience a varying stiffness during the drilling operation, which could affect the fatigue life of the conductor. Changes in stiffness will alter the distribution of lateral load down the pile. In turn, this may cause fatigue hot spots to migrate upward or downward, smearing the damage. Such hotspot migration is not captured in fatigue assessments that consider only a single lateral stiffness that does not change throughout the drilling campaign.

![Graph showing normalised secant stiffness vs prototype time for episodes of 200 cycles and 10000 cycles of amplitude 0.08d (mid-pile level) separated by periods of consolidation.]

**Figure 3-9.** Normalised secant stiffness vs prototype time for episodes of 200 cycles and 10000 cycles of amplitude 0.08d (mid-pile level) separated by periods of consolidation.

Also shown on **Figure 3-9** are the results from Type 6 testing. It is interesting to note that the initial stiffness after dissipation is broadly consistent with the long-term trend in Type 7, but that this value degrades with cycling.

### 3.1.5. Summary and conclusions

For undrained cyclic loading at given amplitude, there is a minimum (or fully degraded) stiffness that is typically reached after a few hundred cycles, provided that the soil has not experienced higher strains. However, this stiffness does not remain “steady” with time, even when cycling continuously. Accordingly, changes in stiffness may occur during drilling operations, which could affect the fatigue life of the conductor – and should be considered.
Similarities between the shear modulus behaviour from soil element tests and the undrained normalised secant stiffness of \( p-y \) curves are shown in Figure 3-5, suggesting these may be linked for design purposes - although more tests are required to confirm this hypothesis.

Experimental testing has shown that the undrained stiffness \( K \) depends on the maximum cyclic amplitude the soil has experienced, and the number of cycles that are applied. Periods of consolidation also strongly influence conductor stiffness.

Current approaches and guidelines do not take the ‘whole life’ load history into account when determining the soil-conductor behaviour for fatigue analysis, yet this may impact on the fatigue life estimation. This paper has shown that load history has an effect on the stiffness of conductor \( p-y \) curves, through degradation and consolidation effects. This work is part of a broader research project conducted at UWA which aims to understand better the ‘whole life’ \( p-y \) behaviour and the resulting impact on conductor fatigue life estimation.

**Acknowledgements**

This work was supported by the ARC Industrial Transformation Research Hub for Offshore Floating Facilities which is funded by the Australian Research Council, Shell Australia Woodside Energy, Bureau Veritas and Lloyds Register Group (IH140100012). The third and fourth authors also acknowledge support from Shell Australia via the Shell Chair in Offshore Engineering at UWA.

**References**


3.2. Evolving soil-conductor stiffness due to multiple-episode cyclic loading

Abstract

Offshore drilling operations typically involve a wellhead/casing system connected to a floating vessel, with metocean forcing on the vessel translating to lateral movement at the wellhead. The conductor is the outer-most casing and must be checked against fatigue due to repeated cycling, which can create ‘hot-spots’ of high accumulated damage. Assessing this involves soil-structure interaction analysis, in which the soil-conductor lateral behaviour is modelled as non-linear springs called $p$-$y$ curves.

Current industry practice often only accounts for stiffness degradation due to cyclic loading, excluding load history effects and/or potential regain in stiffness resulting from pore-pressure dissipation after cycling.

This paper compares results from centrifuge testing of a rigid length of conductor installed in reconstituted samples of carbonate silt and kaolin clay, subject to sequences of cyclic lateral displacement. The main aspects studied are the effect of previous cycling (load history), the effect of consolidation between episodes of undrained loading, and the impact of one-way vs two-way loading.

3.2.1. Introduction

Conductors are a critical part of drilling riser operations, and are susceptible to fatigue because of the continuous motion of the system. Safe design depends on the assessment of fatigue utilisation, which is performed with interaction analyses where the soil-conductor behaviour is modelled as a series of non-linear springs, characterised by their secant stiffness. This stiffness will influence the distribution of bending moment along the length of the conductor and the location of the hot-spot, where the risk of fatigue damage is greatest.

Recently proposed approaches for soil-conductor lateral behaviour analysis (Zakeri et al. 2019) assume that the secant stiffness reaches an amplitude-dependent steady state behaviour after a few hundred cycles of loading. This method, which will be referred to as the “steady state approach” in this paper, is based on centrifuge tests on kaolin and Gulf of Mexico Clay.
However, recent centrifuge tests on carbonate silt (Guevara et al. 2020) have shown that the secant stiffness increases with time from an undrained fully remoulded minimum (steady-state), consistent with the behaviour observed previously by multiple authors (Zhang et al. 2011, Zhou et al. 2019). In addition, the undrained fully remoulded stiffness and subsequent increase depend on the highest previous load experienced by the soil (Doherty et al. 2019, Guevara et al. 2020) and the number of cycles applied.

The results presented in this paper aim to expand on previous work by examining fully remoulded stiffness and its load-history dependence. The tests presented here are part of a broader campaign aimed at improving fatigue design of subsea well conductors and casings.

**3.2.2. Centrifuge test description**

The tests were conducted in the 1.8m radius beam centrifuge at the National Geotechnical Centrifuge Facility, University of Western Australia, on samples of reconstituted carbonate silt and kaolin clay. Samples were prepared from slurry that was into a strongbox of internal dimensions of 650 mm long, 390 mm wide and 325 mm height (with a sand base drain). The moisture content ($w$) of the slurries was 140% for the carbonate silt and 130% for the kaolin clay. After positioning it in the centrifuge, the sample was spun for ~ 5 days at an acceleration of 40g until fully consolidated.

A miniature T-bar penetrometer (5 mm in diameter and 20 mm in length (Stewart and Randolph 1991)) was used to track changes in soil strength with time. T-bar tests were performed at a penetration rate of 3 mm/s and included a cyclic stage to investigate remoulding. Typical measured strength gradients were 1.65 and 1.05 kPa/m, with sensitivities of 3.3 and 2.40, for the carbonate silt and kaolin clay, respectively. Values of consolidation coefficient have been reported by other authors as $c_v = 1 \text{ m}^2\text{/year}$ (Chow et al. 2019) for the carbonate silt and 3.3 $\text{ m}^2\text{/year}$ (Richardson et al. 2009) for the kaolin clay ($\sigma'_v = 15\text{kPa}$).

It is important to note that, while the carbonate silt was recovered from an offshore site, its response does not necessarily replicate in-situ soil behaviour. This stems from the reconstitution process used in the laboratory, which cannot replicate the natural deposition process – often resulting in lower sensitivity in the model than in the field. However, it is noted that the strength and sensitivity of the carbonate silt are similar to properties seen in
deep water Gulf of Mexico clay, hence making these results directly relevant for conductors in such soils.

A schematic of the test setup used is shown in Guevara et al. (2020), and briefly described here. The stainless-steel model pile had an outer diameter ($D$) of 19.5 mm and a wall thickness ($t_w$) of 0.95 mm ($D/t_w = 20.5$), representing a prototype conductor of 780 mm (30 inch) and 38 mm (1.5 inch) in diameter and wall thickness, respectively. The horizontal load $F_h$ applied was measured using a bending leg, with two independent lasers measuring the displacement and rotation of the pile. Although the pile was practically rigid for the range of loads applied, there was a very small amount of compliance in the connection of the pile and the bending leg. This was accounted for by developing a relationship between the applied load and pile rotation, which was then used to determine the pile lateral displacement at 50% of the penetration depth.

The piles were installed in flight to an embedment depth of 4.5 $D$ or 6 $D$ (depending on the test) into 8 different locations within the sample. The test locations were separated by at least 6 $d$ from each other and from the walls of the box to avoid interaction effects. After installation, the lateral loading sequence imposed on the pile was preceded by a stationary period of 2.15 hours to allow the dissipation of the pore pressures generated during the installation process.

The tests discussed in this paper comprised four episodes of displacement controlled cyclic motion at the actuator level, followed by a stationary period to allow for pore pressure dissipation. The duration of the stationary period was assessed based on the solution proposed by Osman and Randolph (2012) for consolidation around a laterally loaded pile, and set to achieve approximately 75% dissipation. This solution was also applied to determine the time frame to consider an episode of cycling as (effectively) undrained, by restricting its duration to the time that would result in no more than 25% consolidation. Each episode consisted of an arrangement of five packets of increasing and decreasing lateral displacement amplitude ($\Delta y$), shown in Table 3-2, and were applied to both carbonate silt and kaolin clay. In addition to the tests in Table 3-2, a monotonic test was performed to determine the lateral capacity ($F_{h,u\text{lt}}$) for each soil, for comparison to cyclic testing.
Table 3-2. Test type and description.

<table>
<thead>
<tr>
<th></th>
<th>Test A: Amplitude of each 100 cycles packet</th>
<th>(Δy/D)</th>
<th></th>
<th>Test B: Amplitude of each 200 cycles packet</th>
<th>(Δy/D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS</td>
<td>0.04</td>
<td>0.08</td>
<td>0.16</td>
<td>0.08</td>
<td>0.04</td>
</tr>
<tr>
<td>KC</td>
<td>0.05</td>
<td>0.10</td>
<td>0.20</td>
<td>0.10</td>
<td>0.05</td>
</tr>
</tbody>
</table>

3.2.3. Results and discussion

The results are reported in terms of a normalised secant stiffness ($K$), which has been defined in this paper as:

$$K = \frac{\Delta F_h / F_{h,ult}}{\Delta y / d}$$  \[3-2\]

Where $\Delta F_h$ is the cyclic range in horizontal load for a particular cycle, representing the integrated response of the soil pressure along the pile length; $F_{h,ult}$ is the measured lateral load capacity from the monotonic test; $\Delta y$ is the difference between the peak lateral displacement and the trough lateral displacement at mid embedment depth of the conductor per cycle (range of displacement), and $d$ is its diameter.

A plot of the horizontal load and displacement at the actuator level, for the first two episodes of test type 1 in the carbonate silt with 4.5 D embedment, is shown in Figure 3-10a. The first and last cycle of packets 2, 3 and 4 are highlighted in red, green and magenta, respectively, for the first episode of the test. The load against displacement at mid embedment depth of the pile is shown in Figure 3-10b for the highlighted cycles, with an indication of how $\Delta F_h$ and $\Delta y$ are obtained for equation [3-2], for the first cycle of the 3rd packet (green continuous line on the plot).
Figure 3-10. Data from test Type 1 at 4.5 D embedment depth: a) Head load and displacement at the actuator level vs time (model scale); b) Head load vs displacement at mid-embedment level for the start and end cycle of the 2nd, 3rd and 4th packets of loading, and the monotonic push test (model scale).

3.2.3.1. The effect of previous cycling on the stiffness during an undrained event.

The secant stiffness $K$ vs cycle number for all the packets of episode 1, 2, 3 and 4, from test type A (two-way loading) and type B (one-way loading) at 6D embedment depth, can be observed in yellow, red, green and blue, respectively, in Figure 3-11, for both carbonate silt and kaolin clay. As described by Guevara et al. (2020), previous cycles of high amplitude displacement have an effect on the subsequent lesser amplitude cycles within an undrained episode of loading. At the end of the third packet of the first episode on the carbonate silt, and second packet for the kaolin clay, the subsequent stiffness for smaller amplitudes was fully remoulded and exhibited a value comparable to that of a large amplitude cycling ($\sim 0.1D$). Despite higher sensitivity, it appears from these plots that the carbonate silt shows a more resilient behaviour than the kaolin clay: the stiffness fully degraded for the silt when cycled at $\Delta y/D = 0.16$, whereas the stiffness of the kaolin fully degraded when cycled at $\Delta y/D = 0.10$.

3.2.3.2. One-way cycling vs two-way cycling.

A comparison of the secant stiffness from the final cycle of each packet of episode 1 and episode 4 for both test type A and test type B is plotted against half the cyclic amplitude range ($\Delta y/2D$) in Figure 3-12 for both soils. Not much difference is observed between one-way (offset) loading and two-way loading tests on both soils, except for very small displacement amplitudes ($\Delta y$) during the increasing phase of episode 1 of the tests –
although it is difficult to conclude on with certainty whether this is a real effect, due to challenges in measuring the response at such small movements.

**Figure 3-11.** Secant Stiffness against cycle number for test type A (two-way cycling) and test type B (one-way cycling) on carbonate silt and kaolin clay.

**Figure 3-12.** Secant stiffness of the final cycle in each packet against episode amplitude for test type A (two-way) and test type B (one-way) on carbonate silt and kaolin clay, for first and final episodes.
3.2.3.3. Regain in strength after undrained cycling episodes and consolidation periods.

The undrained secant stiffness ($K$) of the final cycle of each packet during the increasing amplitude phase of all the episodes for test type A (two-way) on carbonate silt and kaolin clay can be observed in Figure 3-13a, plotted against the amplitude of cycling. After the first loading episode in the carbonate silt, the stiffness of the subsequent episodes follows a distinctive path, in which the stiffness values are approximately half of those in the first episode. However, the stiffness path for the kaolin clay progressively increases with each episode of cycling and consolidation after the fully remoulding process of the first episode.

![Figure 3-13](image)

**Figure 3-13.** Secant stiffness against amplitude per episode of undrained cyclic loading of two-way cyclic amplitude tests on carbonate silt and kaolin clay on a) The increasing cyclic amplitude phase, and b) the decreasing cyclic amplitude phase.

The undrained secant stiffness ($K$) against amplitude per episode of undrained cyclic loading for the decreasing amplitude phase of test type A on carbonate silt and kaolin clay is plotted in Figure 3-13b against the amplitude of cycling. It is observed that, after the first episode of loading, the stiffness decreases further on the second episode and shows a recovery at the last episode, in which the stiffness value is 1.5 times the minimum value. Conversely, the kaolin clay shows an increase in stiffness with the decrease in cyclic...
amplitude, and a progressive recovery after each episode. It is also observed that the gradient of the path of the stiffness becomes steeper for each subsequent episode.

Interestingly, the kaolin clay exhibits more degradation than the silt on the largest amplitude packet of the first episode of cycling, but recovers up to 2 times its minimum value.

### 3.2.4. Conclusions

1. It was verified that one-way and two-way cycling do not yield significantly different secant stiffness values, which agrees with results reported from previous studies. Differences were observed for very small displacements on the increasing amplitude phase of the tests, but more tests are needed to confirm if this is a real phenomenon. This means that a small offset in displacements (<0.1 $D$), produced by drifting of the vessel, could be analysed as if the vessel was positioned on top of the conductor, which is common practice when performing well-head systems fatigue life assessments.

2. The soil-conductor stiffness on the decreasing amplitude phase of each event, is lower than on the increasing amplitude phase. Instead of returning to its steady state value from the increasing amplitude phase, $K$ remained constant as the amplitude reduced for the carbonate silt, being set by the previous maximum sustained amplitude. For the kaolin clay on the decreasing amplitude phase, $K$ approximately doubled as the amplitude reduced by a factor of 10, but remained about half the value of $K$ from a steady state approach. This means that, if the conductor has experienced sufficient cycles of a larger amplitude loading, its stiffness will be considerably reduced and the hot-spot will migrate down the conductor, influencing the fatigue life estimation.

3. A recovery in the undrained fully remoulded secant stiffness was observed after each period of pore pressure dissipation, where $K$ increased to 1.5 times its minimum value for the carbonate silt and 2 times for the kaolin clay, on the decreasing amplitude phase of each episode. This stiffening effect of the consolidation after cycling partially removes the reduction in stiffness of unloading relative to loading, and implies that assuming a post-large-amplitude secant stiffness for smaller cycling amplitudes is not appropriate. An adequate soil-conductor modelling has to include both the effect of previous remoulding and pore pressure dissipation.
Acknowledgements

This work was supported by the ARC Industrial Transformation Research Hub for Offshore Floating Facilities which is funded by the Australian Research Council, Shell Australia, Woodside Energy, Bureau Veritas and Lloyd’s Register (IH140100012). The third and fourth authors also acknowledge support from Shell Australia via the Shell Chair in Offshore Engineering at UWA.

References


3.3. A comparison study of soil-conductor stiffness from centrifuge and laboratory testing

Abstract

Conductors are the outermost casing in an offshore well, providing axial and lateral support to the inner strings and mitigating hole collapse during drilling at shallow depth. Recent studies have investigated the geotechnical response of a conductor subject to cyclic lateral loading induced by environmental conditions. Originally based on a series of centrifuge tests reported by BP, and subsequently extended through the development of a laboratory scale testing apparatus (operated by NGI), the result is a growing body of test data in this area. To extend and verify this experience for other soil types and a wider range of riser motion histories, a new research project was launched at The University of Western Australia, which explores the life cycle behaviour of conductors. The objective of this paper is to present a comparison between results from recent centrifuge testing and companion tests performed by NGI using the laboratory (p-y) apparatus. The tests were performed on reconstituted samples of kaolin clay and carbonate silt. A parallel suite of simple shear tests at different strain rates was also performed. These are presented in a way that allows direct comparison to the centrifuge and p-y apparatus data. Where differences are observed, these are discussed and explanations proposed. The data obtained is also plotted against the recommendations in a recently published approach, to be used as the basis of a draft industry standard – with an emphasis on identifying potential refinements of the method to capture soil-specific characteristics.

3.3.1. Introduction

Offshore drilling for hydrocarbons first involves the installation of large steel pipe piles called conductors, which are required to provide axial support for the well and drilling components, and to prevent the walls of the drill hole from collapsing. Connected to the vessel through a riser system, environmental loads acting on the vessel and riser system translate into lateral displacements at the top of the conductor – which are typically less than 10% of the conductor diameter. Fatigue is a major design consideration for conductors, where analysis of the conductor often considers the soil as Winkler springs called p-y curves. A commonly used industry approach (API 2014) for modelling this behaviour leads to curves with smaller stiffness than is expected in practice, and observed in, for instance,
centrifuge experimental tests (Jeanjean 2009). In particular, the stiffness of the API (2014) curves does not adequately capture the small-strain response, which is the range of displacements in which the conductor will be moving. Several recent methods have been proposed as alternative design approaches, including Zakeri et al. (2019) and Komolafe and Aubeny (2020). The former using data from both centrifuge testing and a novel “p-y testing apparatus” (Zakeri et al. 2017) to propose updated curves representing what the paper defines as the “steady state” stiffness for deep water clays.

More recent work (Guevara et al. 2020) suggests that there is no true “steady state” response, but rather a “fully softened” response which is transient in nature, and may recover depending on the drainage properties of the soil and load history. Further, it is suggested that it may be possible to link the “fully softened” stiffness to results from conventional soil element testing (such as resonant column and simple shear), in line with previous studies on scaling of laboratory testing to monotonic p-y curves (Zhang and Andersen 2017).

This paper compares data from centrifuge and p-y apparatus testing in two soil types – kaolin clay and carbonate silt. The results are compared to soil element tests, and the findings used to postulate a potential future design approach.

3.3.2. Description of tests

3.3.2.1. Centrifuge tests

The tests were conducted in the 1.8 m radius beam centrifuge at the National Geotechnical Centrifuge Facility, University of Western Australia on reconstituted samples of kaolin clay and carbonate silt. Samples were prepared from slurry that was placed in a strongbox of internal dimensions of 650 mm long, 390 mm wide and 325 mm height (with a sand base drain). The moisture content (w) of the kaolin clay and carbonate silt slurries were 130% and 140%, respectively. After positioning the strongbox in the centrifuge, each sample was spun for ~ 5 days at an acceleration of 40g until fully consolidated.

A miniature T-bar penetrometer (5 mm in diameter and 20 mm in length per Stewart and Randolph, 1991) was used to track changes in soil strength with time. T-bar tests were performed at a penetration rate of 3 mm/s and included a cyclic stage to investigate remoulding. The typical T-bar strength gradient ($k_{su}$) and measured sensitivity ($S_t$) were
1.05 kPa/m and 2.4, respectively, for the kaolin clay, and 1.65 kPa/m and 3.3 for the carbonate silt, determined with a $N_{T-bar} = 10.5$.

A rigid hollow steel pile was installed in different locations of the soil box, instrumented with a device to measure the shear force at the head of the pile, and two lasers to monitor its rotation and displacement. A detailed description of the test setup used can be found in Guevara et al. (2020). The results presented here are part of a broader study on load history impact on soil-conductor behaviour. In this paper, only the increasing amplitude phase of the tests is discussed (each test consisted of episodes of increasing and decreasing amplitude cycling with pore pressure dissipation periods in between). The tests are displacement controlled to best represent the nature of the loading from a riser-floating facility system. The fragment of each test sequence analysed here is detailed in Table 3-3 for the tests in kaolin clay. Although the sequences were similar for the tests performed in carbonate silt, the amplitudes of each packet differ - details of testing in the carbonate silt can be found in Guevara et al. (2020).

**Table 3-3. Centrifuge tests type and description (kaolin clay).**

<table>
<thead>
<tr>
<th>Test number</th>
<th>Amplitude, $\Delta y/2D$ (-)</th>
<th># of cycles (N) per packet of amplitude</th>
<th>Illustration</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Monotonic push</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>2-way loading 0.02 – 0.04 – 0.07 – 0.14 – 0.16</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>2-way loading 0.05 – 0.10 – 0.20</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1-way loading 0.05 – 0.10 – 0.20</td>
<td>200</td>
<td></td>
</tr>
</tbody>
</table>

3.3.2.2. P-y apparatus tests

Tests were conducted in the BP-developed $p$-$y$ apparatus operated by NGI (Zakeri et al. 2017). The apparatus comprises a cylindrical chamber that houses the sample. Once the sample is installed, a 10 mm diameter rod is inserted through a pre-augered hole in the soil and the specimen is consolidated via pressure applied at both ends of the cylinder. After the soil is consolidated, the rod (representing a conductor) is moved laterally. The soil chamber has an inner diameter of 68 mm, and samples are trimmed to a height to diameter
ratio between 1:1 and 2:1. The samples used in this project were consolidated in tubes from slurry, with the samples prepared in Australia and transported to Norway. The unit weight profile of the kaolin clay and carbonate silt centrifuge samples (required to determine the vertical pressure), were obtained with moisture content cores extracted from undisturbed locations in order to determine the vertical stress that would produce a sample with strength \( s_u \) of 10 kPa (needed for handling / placing samples in the p-y apparatus). The final pressures used in the p-y apparatus were marginally higher than the (tube) pre-consolidation pressures, at 68 kPa and 33 kPa for the kaolin clay and carbonate silt, respectively.

Each test involved episodes of cyclic displacement-controlled packets of increasing amplitude followed by a symmetrical decreasing amplitude phase. All the tests performed with the p-y apparatus were loaded symmetrically from zero displacement (two-way). In some cases, a consolidation period would then be allowed, enabling pore pressures generated during the cycling to dissipate, after which the cyclic episode was repeated. The objective of this was to track the impact of large amplitude cycling on subsequent smaller amplitude cycling, via changes in stiffness. In this paper, only the increasing amplitude phase of the first episode of each tests will be discussed. The fragment of each sequences analysed here is detailed in Table 3-4, and was consistent for both kaolin clay (KC) and carbonate silt (CS).

<table>
<thead>
<tr>
<th>Test number</th>
<th>Amplitude, ( \Delta y/2D ) (-)</th>
<th># of cycles (N) per packet of amplitude</th>
<th>Illustration</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.04 - 0.08</td>
<td>250 – 25</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.04 - 0.08</td>
<td>250 – 250</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>0.35</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>0.0004 – 0.002 – 0.009 – 0.04</td>
<td>100</td>
<td></td>
</tr>
</tbody>
</table>

3.3.2.3. Laboratory tests

Laboratory soil element tests were performed to complement the centrifuge and p-y apparatus data, using soil samples consolidated in tubes. Simple shear tests at UWA were performed with both the Berkeley style apparatus (flexible sample membrane and radial
confinement pressure e.g. Villet et al. 1985) and the stacked ring apparatus (no radial
confinement; pressure is imposed as the rings prevent horizontal deformation). Direct
simple shear tests at NGI were performed with the Geonor apparatus, utilising a wire
reinforced membrane (Bjerrum and Landva 1966). The tests were performed at the same
vertical pressures as the p-y apparatus tests, and sheared at different rates: ~0.01 to 0.1
mm/min for slow rate tests and 2.5 mm/min for fast rate tests. The radial pressure used in
the Berkeley style apparatus was determined from $K_0 = 0.6$-0.8 for the kaolin clay, and $K_0 = 0.6$ for the carbonate silt. Additional simple shear (using a Berkeley style apparatus) and
resonant column tests were performed by an independent testing laboratory (GTI Perth).
The resonant column tests were performed at mean stresses of 50 kPa and 30 kPa for the
kaolin clay and carbonate silt, respectively.

Table 3-5 compares the shear rate for each test, with strains from the simple shear
converted to normalised lateral displacements ($y/D$) using $\zeta_2 = 1.6$, as recommended by
Jeanjean et al. (2017).

<table>
<thead>
<tr>
<th>Test</th>
<th>Rate, %/t (%)/s</th>
<th>Rate, y/t (mm/s)</th>
<th>Pile diameter, D (mm)</th>
<th>Norm. rate, ($y/D$)/t (-/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS UWA/GTI (slow)</td>
<td>0.006</td>
<td>1 x 10^{-4}</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS UWA/GTI (fast)</td>
<td>0.15</td>
<td>2 x 10^{-3}</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS NGI (slow)</td>
<td>0.001</td>
<td>2 x 10^{-5}</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS NGI (fast)</td>
<td>0.25</td>
<td>4 x 10^{-3}</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Centrifuge (Monotonic)</td>
<td>1.0</td>
<td>19.5</td>
<td></td>
<td>5 x 10^{-2}</td>
</tr>
<tr>
<td>$p$-$y$ testing (Monotonic)</td>
<td>0.47</td>
<td>10.0</td>
<td></td>
<td>5 x 10^{-2}</td>
</tr>
</tbody>
</table>
3.3.3. Results and discussion

3.3.3.1. Laboratory tests results

Shear stress ratio against strain from simple shear testing performed at all three laboratories is shown in Figure 3-14 for the kaolin clay and Figure 3-15 for the carbonate silt. Testing on the kaolin clay suggests modest rate effect (~9% per log cycle), which is captured by the Berkeley and stacked-rings apparatus, but not observed in tests using a reinforced membrane. Testing on the carbonate silt shows higher rate effects (~18% per log cycle), which was again not evident in tests using a reinforced membrane. The reason for this is being investigated further, as rate effects would typically be observed in tests using this type of apparatus (e.g. Lunne and Andersen 2007).

Figure 3-14. Simple shear tests results on kaolin clay.

Figure 3-15. Simple shear tests results on carbonate silt.
Differences are observed in the stress-strain behaviour of the two soils – with the kaolin clay showing a more ductile response with gradually increasing mobilised strength. The carbonate silt has a much stiffer initial response, reaching a peak at $\gamma < 2\%$.

3.3.3.2. *Comparison of centrifuge and p-y apparatus results*

**Monotonic push**

In order to compare the monotonic response from centrifuge and $p-y$ apparatus, the measured lateral capacity is normalised by $s_u$. An average shear stress ratio was selected from the slow simple shear tests – $s_u/\sigma'_v = 0.22$ and $0.32$ for kaolin clay and carbonate silt, respectively. For the case of the $p-y$ apparatus tests, the vertical stress ($\sigma'_v$) was imposed by the apparatus, whereas for the centrifuge tests the vertical stress was calculated at mid-depth of the pile, using an appropriate soil unit weight profile.

For the centrifuge tests, the average pressure over the embedment depth was determined by dividing the measured shear force at the pile head by the projected lateral area of the pile, while the displacement was taken at mid-embedment depth (calculated from laser measurements). Due to their shallow depth, lateral loading in the centrifuge mobilises a combination of wedge and flow around failure mechanisms, whereas the $p-y$ apparatus is intended to model only a flow around failure mechanism.

The normalised monotonic response for testing in kaolin clay and carbonate silt is shown in Figure 3-16 and Figure 3-17. On the vertical axis of both figures, the lateral capacity is normalised by the undrained shear strength from the slow simple shear tests. On the right axis, the measured capacity is normalised by the undrained shear strength scaled for rate effects – using multipliers of $1.27$ and $1.47$ for kaolin clay and carbonate silt respectively.

The following observations are made:

- The centrifuge tests on kaolin clay suggest $N_p$ in the range 12-14 using the slow simple shear soil strength, with the $4.5d$ test giving lower resistance and requiring greater displacement to mobilise the full capacity. The range of $N_p$ is consistent with (but slightly higher than) values proposed in the literature, and both tests show softening behaviour.

- In contrast, the $p-y$ apparatus (performed at the same normalised rate as the centrifuge tests) measures higher values of $N_p$ even at low displacement. The reason for this is not fully understood, but may be due to boundary effects.
Figure 3-16. Measured $N_p$ from tests in kaolin clay.

Figure 3-17. Measured $N_p$ from tests in carbonate silt.

- In the carbonate silt, the response of the $p-y$ apparatus appears to be broadly similar to the centrifuge tests at low displacement levels, with both apparatus measuring a peak at $< 0.05 \, d$ before softening. However, with further displacement the $p-y$ apparatus increases in resistance – possibly as boundary effects dominate relative to the localised failure mechanism that produced the initial peak.

- The different stress-strain response in the simple shear tests between the two soils may explain why an initial peak is seen in the carbonate silt but not in the kaolin
clay. Overall, the higher monotonic values (in the $p$-$y$ apparatus) are important to consider when normalising cyclic test results.

**Cyclic tests**

A comparison of the cyclic $p$-$y$ apparatus and centrifuge tests is achieved by tracking changes in secant stiffness ($k_{p-p}$), defined as the cyclic range of horizontal pressure ($\Delta P$) divided by the range in horizontal displacement ($\Delta y$) normalised by the pile diameter ($D$), as shown in Equation [3-3]:

$$k_{p-p} = \frac{\Delta P}{\Delta y / D}$$  

[3-3]

Zakeri et al. (2016) suggest that after a number of cycles at a given amplitude a “steady state” stiffness is reached, beyond which no further changes in stiffness occur. In reality, this “steady-state” is transient and depends on the cyclic strain imposed and the drainage properties of the soil (Guevara et al. 2020). Accordingly, the term “fully softened” stiffness will be used for the state of initial (maximum) softening. Examples of where this value is selected are shown in Figure 3-18 and Figure 3-19 for two of the $p$-$y$ apparatus tests.

**Figure 3-18.** Selected fully softened stiffness for $p$-$y$ apparatus Test 2.
Figure 3.19. Selected fully softened stiffness for p-y apparatus Test 4.

The measured values of $k_{p-p}$ are subsequently normalised by an ultimate pressure ($P_u$), giving the non-dimensional $K$, as indicated by equation [3-4]:

$$K = \frac{k_{p-p}}{P_u}$$  \[3-4\]

Note that $K$ is the same term defined in Guevara et al. (2020).

Since the monotonic centrifuge tests indicated $N_p \sim 12$ based on slow simple shear tests, this has been used (with the appropriate soil strength) to normalise the $k_{p-p}$ for both kaolin clay and carbonate silt centrifuge tests. In comparison, two different approaches were used to normalise measured $k_{p-p}$ from p-y apparatus testing – with $k_{p-p}$ normalised either using $N_p = 12$ (and the slow simple shear strength) or the maximum measured $P_u$ observed in the monotonic test. The same approach was used for p-y apparatus tests in both soil types, with the results shown in Figure 3-20 (kaolin clay) and Figure 3-21 (carbonate silt). For comparison purposes, the normalised steady state stiffness proposed by Zakeri et al. (2019) for a spring-only system with soil $s_u < 40$ kPa is also plotted. The following is observed:
When comparing centrifuge and p-y apparatus results, there appears to be better agreement when using the higher (measured) values of $N_p$ at small displacement levels, and the theoretical value of $N_p = 12$ at higher displacement levels.

The relationship proposed by Zakeri et al. (2019) overpredicts the observed stiffness, including for low strains where a ‘cut off’ maximum value was proposed.

Figure 3-22 (kaolin clay) and Figure 3-23 (carbonate silt) compare the normalised fully softened stiffness ($K$) with results from simple shear and resonant column testing, with the p-y apparatus results normalised only by the maximum measured $P_u$. On the right axis, the results of $G/G_{\text{max}}$ from resonant column and simple shear tests are plotted, with strains scaled to lateral displacement ($y$) using the approach proposed by Zhang and Andersen (2017). Values of $\xi_1 = 2.8$ and $\xi_2 = 1.6$ were assumed, consistent with a rough pile, for scaling the elastic and plastic strains, respectively, with the elastic strain calculated based on the resonant column data of $G_{\text{max}}$. Measured values of $G_{\text{max}}$ in kaolin clay and carbonate silt were 16.6 MPa and 16.0 MPa respectively.
**Figure 3-21.** Normalised fully softened secant stiffness (carbonate silt).

**Figure 3-22.** Normalised fully softened secant stiffness from p-y apparatus and centrifuge tests, and G/Gmax from resonant column and simple shear tests (kaolin clay).

The plots show similarities between the behaviour of the normalised undrained fully softened stiffness and the normalised secant modulus from resonant column and simple shear testing – with a limiting value of $K = 375$ for kaolin clay and $K = 350$ for carbonate...
silt bringing the data broadly into alignment. The ratio of limiting $K$ and $G/G_{\max} = 1$ is expected to be linked to the rigidity ratio of the soil. Elastic solutions for lateral pile displacement (e.g. Baguelin et al. 1977) suggest that $K_{p,p} \approx 4G$ – with $K = 375$ and $K = 350$ resulting in $G_{\max}$ values that are broadly consistent with the resonant column data for the two soils. This suggests it may be possible to develop a link between the normalised fully softened stiffness and soil element tests readily performed in a laboratory, although more work is required to confirm this.

![Graph showing carbonate silt data](image)

**Figure 3.23.** Normalised fully softened secant stiffness from p-y apparatus and centrifuge tests, and $G/G_{\max}$ from resonant column and simple shear tests (carbonate silt).

### 3.3.4. Conclusions

This paper presents a comparison between centrifuge, $p$-$y$ apparatus and soil element testing in kaolin clay and carbonate silt. From this comparison the following observations are made:

1. The $p$-$y$ apparatus fully softened secant stiffness results agree with those from centrifuge tests results when normalised by the measured monotonic capacity. This capacity was observed to be significantly higher than the theoretical capacity in the kaolin clay. For the carbonate silt, the $p$-$y$ apparatus showed an initial peak that was comparable to the centrifuge results, before increasing to values in excess of the theoretical capacity. This is different to previous experience with this apparatus testing mostly intact soils from offshore, where the back-calculated capacity values were generally closer to the theoretical values.
2. The different stress-strain response from the simple shear tests on kaolin clay and carbonate silt could explain the different $p$-$y$ responses – with the carbonate silt showing a stiff initial response, and the kaolin clay a more ductile response. Overall, the higher measured resistances may reflect an effect from the boundary in the $p$-$y$ apparatus. However, as this has not been observed in earlier testing using this apparatus (on mostly intact soils), further investigation of this aspect is warranted.

3. The relationship proposed by Zakeri et al. (2019) overpredicts the observed stiffness for the soils studied in this paper.

4. Similarities were observed between the normalised fully softened stiffness ($K$) vs displacement behaviour, and the $G/G_{\text{max}}$ vs shear strain behaviour. A design curve could potentially be constructed by scaling simple shear and resonant column tests, although more tests are required to confirm this hypothesis.

This work is part of a broader research project which aims to understand better the ‘whole life’ $p$-$y$ behaviour and the resulting impact on conductor fatigue life estimation.

Acknowledgements

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References


CHAPTER 4 - Cyclic $p$-$y$ model to account for load history in soil-structure interaction analysis

Prologue

This chapter (Chapter 4) presents a $p$-$y$ model that is able to replicate the observations from experimental data outlined in Chapter 3:

1. After large amplitude cycling, the soil-conductor lateral stiffness does not return to its initial fully remoulded state at smaller amplitude cycling, but remains impacted by the previous loading history.
2. The cyclic behaviour of the soil-conductor lateral stiffness is material-dependent: different soil types will experience different behaviour based on characteristics such as sensitivity and permeability.
3. The degradation of stiffness and strength of soil-conductor load-displacement behaviour due to pore pressures generated when cycling, occurs in parallel to the dissipation of these pore pressures.
4. The pore pressure dissipation leads to a regain in strength and stiffness, and a hardening of the response with subsequent loss of sensitivity.

This Chapter comprises two journal papers:

In the first paper was published in Computers and Geotechnics and is titled: “A cyclic $p$-$y$ model for the whole-life response of piles in soft clay” (White et al., 2021). It presents a $p$-$y$ model (PICSI) developed to capture the features observed from the tests. The model is validated against other tests available in the literature and against one of the tests presented in Chapter 3. PICSI has the capability to accurately predict the changes in strength and stiffness with different loading sequences accounting for load-history and pore pressure dissipation.

The second paper has been prepared for submission to Computers and Geotechnics and is titled: “A methodology to calibrate the PICSI cyclic $p$-$y$ model using experimental results and optimisation”. It presents an experimental methodology for the calibration of the PICSI framework based on model testing in reconstituted carbonate silt and kaolin clay. The procedure uses numerical optimisation, where an error function was computed as the average normalised difference between the measured and calculated secant stiffness values.
The calibrated parameters were then validated against results from an independent set of centrifuge tests on carbonate silt using a flexible pile of similar dimensions to a drilling conductor.
4.1. A Cyclic $p$-$y$ model for the whole-life response of piles in soft clay

Abstract

It is evident from model testing, field studies and theoretical considerations that the strength of a soft clay can reduce and then recover – potentially to above the initial value – as a result of cyclic loading followed by consolidation. For piled foundations and well conductors, these changes in soil strength and the resulting lateral resistance affect their stiffness, capacity and fatigue. This paper introduces a new model for the cyclic lateral ‘$p$-$y$’ response of a pile in soft clay, using concepts from critical state soil mechanics, combined with a parallel Iwan model to capture the hysteretic response. Example analyses show that the model can capture the general forms of behaviour observed in model tests, and is rapid and simple to implement. The model provides a new basis for whole life modelling of piles and well conductors, allowing changes in stiffness and capacity to be simulated, as well as improved modelling of fatigue accumulation. This approach allows more reliable design, quantifying the benefits and risks associated with evolving soil strength.

4.1.1. Introduction

Pile foundations and oil and gas well conductors rely on lateral support from the soil to resist horizontal loads. The part of a well conductor immediately below the seafloor acts as a laterally-loaded pile that provides restraint for the well sections below and above. It is conventional for the soil reaction on piles and conductors to be reduced to a single degree of freedom ‘$p$-$y$’ non-linear spring, which describes the lateral resistance offered by the integrated effect of the soil around the pile. In soft clay, theoretical solutions exist to link the elastic stiffness and plastic strength of the soil to the initial $p$-$y$ stiffness and the limiting resistance, $p_u$, on the pile, respectively (Baguelin et al. 1977, Randolph and Houlsby 1984). Empirical approaches, calibrated to field tests, have been used to define the full $p$-$y$ load-displacement response (e.g. Matlock 1970).

It has long been recognised that cyclic loading causes softening of clay due to pore pressure generation, and methods exist to estimate the resulting cyclic strength for design calculations (e.g. Andersen et al. 1988). For laterally-loaded piles, modifications to the monotonic $p$-$y$ response have been proposed to allow for the effects of cyclic loading, which generally involve a factoring down of the static lateral resistance (e.g. Doyle et al. 2004).
A further adjustment for the strain rate during cyclic loading compared to static laboratory tests may compensate for this reduction. Also, more sophisticated methods exist to convert a history of cyclic loading into a specific p-y response (Erbrich et al. 2010, Zhang et al. 2017, Komolafe and Aubeny 2020).

Recent studies have also highlighted that dissipation of the pore pressure generated by undrained cyclic loading leads to reconsolidation and recovery of the soil strength. Model tests of ‘episodic’ cyclic loading – i.e. with sets of cycles interspersed by periods of consolidation – by Zhang et al. (2011) showed the pile head lateral stiffness fall to 40% of the initial value during an initial cyclic episode, but then rise by a factor of 2 following subsequent consolidation periods. Further results presented by Doherty et al. (2019), Lai et al. (2020) and Guevara et al. (2020) show similar trends in more detail, illustrating the effects of cyclic amplitude, pile length and different patterns of cycling and waiting periods. The observed changes in p-y stiffness are significant and affect the stability, stiffness and fatigue rate of piles and well conductors.

The same process of cyclic softening followed by consolidation and strengthening has been recognised in the behaviour of pipelines, foundation and anchors on soft clay seabeds, and has been captured by simple design-focused models based on critical state soil mechanics (e.g. Boukpeti and White 2017, Cocjin et al. 2017, DNV-GL 2019, Zhou et al. 2020). The purpose of this paper is to outline a model for the lateral p-y response of piles that captures these same underlying mechanisms.

The proposed p-y model consists of two components:

- A critical state-inspired (CSI) model for hardening and softening of the p-y response
- A parallel Iwan (PI) model (Iwan 1966) for the hysteretic non-linearity of the p-y response

Referred to as the PICS (parallel Iwan critical state-inspired) model, it tracks the softening caused by lateral pile movement and straining of the surrounding soil, as well as the hardening caused by consolidation over time. The net effect of the softening and hardening varies with time, and is used to scale the strength and stiffness within the PI model of the lateral pile response.
4.1.2. CSI model for hardening and softening

4.1.2.1. Overview

The PICSI model uses an analogue of the voids ratio – strength relationship that underpins critical state soil mechanics, augmented by a minimum of additional features to replicate model test observations. Voids ratio is replaced with a hardening index, $H$ ($0 < H < 1$), and the mean effective stress is replaced by the undrained strength, $s_u$ (normalised by an initial value, $s_{u,i}$), as illustrated in Figure 4-1. $H = 0$ is the initial condition and $H = 1$ is the ultimate limiting condition.

The model represents the behaviour of soil that is initially on the ‘wet side’ of the critical state, meaning that the soil has a tendency to densify on shearing, eventually reaching a higher undrained strength. Continuous cycling from the initial state results in excess pore pressure that temporarily reduces the undrained strength. The pore pressure effect is captured by a proxy parameter called the damage index, $D$ ($0 < D < 1$). Consolidation causes densification and hardening through the dissipation of pore pressure. This is captured in the model by a time-dependent reduction in damage index concurrent with an increase in hardening index, which is analogous to consolidation following an unload-reload path – we therefore define the slope of this path using $\kappa^*$. In the initial state ($H = 0$), the minimum strength is $s_{u,f} = s_{u,i}/S_0$ where $S_0$ is the initial sensitivity. As the soil progressively densifies, the sensitivity reduces to unity. The behaviour therefore converges towards a maximum or final strength, $s_{u,f}$, which is related to $s_{u,i}$ via the parameter $\lambda^*$. This parameter is analogous to the slope of the critical state line, which is also linked to the potential change in soil strength from densification, as illustrated by interpretation of the Atterberg limit tests (Wroth and Wood 1978).

In conventional critical state models, there is a single unique (critical state) strength for a given hardening level. In this present model, the soil strength for a particular hardening value is instead bracketed by initial and remoulded values of strengths (an approach proposed previously by White & Hodder (2010), and other subsequent publications). This feature provides a range of potential strengths for any hardening value, rather than a single critical state value, which allows the model to exhibit remoulding and recovery of strength.

The interaction between damage, consolidation and hardening is illustrated in Figure 4-1. The three paths labelled A-C represent continuous cyclic loading at different rates relative
to the consolidation process are identified to show the potential ways that the undrained strength can evolve. The ‘fast’ case A involves negligible consolidation, so the strength simply falls from the initial to the remoulded value, as observed in cyclic T-bar tests and cyclic lateral pile tests in which negligible consolidation occurs (e.g. Stewart and Randolph 1991, Doyle et al. 2004). The ‘slow’ case C involves a high level of consolidation between each cycle or shearing stage, so the effect of consolidation and hardening eclipses the generation of pore pressure and softening. As a result, the strength rises with every cycle, converging towards the limit. The same behaviour has been observed in axial pipe-seabed sliding tests (Smith and White 2014), interface shear box tests (Boukpeti and White 2017) and episodic T-bar penetrometer tests (Cocjin et al. 2014). However, we are not aware of any lateral pile tests in soft clay that have been conducted with sufficient consolidation between cycles to match this trend. Case B represents continuous cycling at a rate that is intermediate between A and C.

Case E represents episodic cycling, in which packets of fast cycles are interspersed with periods of consolidation. In this case, the softening during each cyclic period is followed by hardening, as the effect of pore pressure dissipation eclipses the effect of generation. The net result is a rise in strength and stiffness, and a reduction in the sensitivity observed in each packet. This trend matches published results from various model testing studies of lateral pile behaviour (Zhang et al. 2011, Doherty et al. 2019, Guevara et al. 2020, Lai et al. 2020) as well as analogous studies of cyclic T-bar penetrometer tests (White & Hodder 2010) or plate anchor loading (Zhou et al. 2020).

**Figure 4.1.** Illustration of model notation and parameters.
4.1.2.2. Governing equations

Current strength
The current normalised strength, \( s_u/s_{u,i} \), depends on the current hardening, \( H \), and damage, \( D \). An equilibrated strength, \( s_{u,e} \) is defined as the strength at the current \( H \) when \( D = 0 \):

\[
\frac{s_{u,e}}{s_{u,i}} = 1 + \frac{H}{\lambda^*} \quad [4-1]
\]

The current strength, \( s_u \), is therefore:

\[
\frac{s_u}{s_{u,e}} = 1 - D \left( 1 - \frac{1}{S_t} \right) \quad [4-2]
\]

A general form of the geometry of the model (Figure 4-1) allows the sensitivity to fall with the hardening, from \( S_t = S_{t0} \) at \( H = 0 \) to \( S_t = 1 \) at \( H = 1 \), at a rate set by the power, \( q \):

\[
S_t = 1 + (S_{t0} - 1)(1 - H)^q \quad [4-3]
\]

This tendency for the sensitivity of a sample to diminish through repeated episodes of cyclic shearing and consolidation has been observed in T-bar penetrometer tests (Hodder et al. 2009) – which are analogous to large-amplitude lateral pile motion – and also during cyclic lateral loading of piles (Zhang et al. 2011). This trend of \( S_t \) can be interpreted as there being no tendency for pore pressure to be generated once \( H = 1 \), because the soil is at a critical state under the equilibrated effective stress (i.e. when \( D = 0 \)). The sensitivity therefore falls to 1 and the damage and hardening processes stop.

In an initial burst of cyclic loading, taking place over a short time so that consolidation and hardening is minimal (i.e. \( H = 0 \)), the strength changes from an initial value \( (s_u = s_{u,i}) \) as damage accumulates \( (D \to 1) \) so that \( s_u \to s_{u,f} \). In the long term, as consolidation dominates, \( H \to 1 \) so \( s_u \to s_{u,f} \), so long as there is cyclic motion that leads to damage, from which consolidation can create the gain in strength (see Figure 4-1 for the strength notation).

Generation of damage
The damage is caused by shearing of the surrounding soil during changes in normalised lateral pile position, \( y/d \) (where \( d \) is the pile diameter), and cannot exceed \( D = 1 \). A simple function to describe this behaviour is:
\[ \delta D = d_r (1 - D)^{d_p} \left| \frac{y}{y_{ref}} \right|^{d_a} \frac{\delta y}{d} \]  

[4-4]

where \( d_r \) and \( d_p \) are dimensionless constants representing the damage rate and power coefficients, while \( \delta y/d \) is a normalised displacement increment. The term \( \left| \frac{y}{y_{ref}} \right|^{d_a} \) allows the rate of damage to depend on the amplitude of displacement, and not just the cumulative displacement, controlled by the exponent \( d_a \). The parameter \( y_{ref} \) is a reference displacement introduced to make this term dimensionless and should be defined as a fraction of the pile diameter, so that consistent parameters can be used across a range of pile sizes (Subsequent examples in this paper assume \( d_a = 0 \), so the parameter \( y_{ref} \) is not used in this paper.). For the case of \( d_a = 0 \), Equation [4-4] can be integrated to show that, for a given \( d_p \), the damage is a function of the scaled cumulative normalised lateral displacement \( S d_r \), where \( S = \Sigma |\delta y/d| \). By including \( d_r \) in this scaled displacement, responses for soils with different damage rates (or levels of ‘brittleness’) will coincide. This behaviour is shown in Figure 4-2, for a range of values of the damage rate parameter, \( d_p \).

Two practical illustrations are used to highlight this approach:

Firstly, the degradation in lateral stiffness observed by (Zhang et al. 2011) during lateral undrained cycles of a model pile in kaolin clay is plotted on Figure 4-2. These stiffness values are the secant peak-to-peak stiffness within a cycle, \( K_{p-p} = \Delta F_h / y^{pp} \) where \( \Delta F_h \) is the difference between the peak values of horizontal force at each cyclic limit and \( y^{pp} \) is the pile displacement between these peak values of \( F_h \). The values of \( K_{p-p} \) are scaled so that initial and the steady final stiffnesses correspond to \( D = 0 \) and 1 respectively. This trend corresponds to \( d_r \sim 2 \) for \( d_r = 1 \).

Cyclic T-bar penetrometer tests provide a second comparison, being analogous to cyclic lateral pile loading. The cycle-by-cycle degradation of steady penetration resistance observed in cyclic T-bar penetrometer tests in soft clay agrees well with the damage function for \( d_p \sim 1-2 \) for \( d_r \sim 1 \), as shown on Figure 4-2. This comparison uses the numerical simulations of Zhou and Randolph (2009), which showed that the failure mechanism around the T-bar has an extent in the direction of movement of approximately two diameters. On this basis, a single pass of the T-bar causes strain in the soil that is equivalent to two diameters of T-bar or pile movement. During the first pass, the average accumulated
strain corresponds to one diameter, and during the return pass the average strain therefore
corresponds to three diameters. The response marked on Figure 4-2 is based on a strain of
10-20 being required for 95% of full remoulding, which originates from interpretation of

![Figure 4-2](image)

**Figure 4-2.** Damage as a function of the scaled cumulative displacement (for \(d_a = 0\)).

This formulation does not recognise cyclic loading as being more damaging than
monotonic loading to the same accumulated deformation, which is a limitation that is
tolerated in other practical models for soil softening (e.g. Whyte et al. 2020). More complex
alternatives could be used in place of Equation [4-4], based on other established models for
pore pressure build up during cyclic loading.

**Dissipation of damage**

The damage decays with time due to pore pressure dissipation, leading to consolidation.
The time rate of this decay follows the usual scaling of consolidation, being proportional
to \(c_r/d^2\):

\[
\frac{\delta D}{\delta t} = -c_r \left( \frac{c_v}{d^2} \right) D^{c_p} \tag{4-5}
\]

where \(c_r\) and \(c_p\) are dimensionless parameters controlling the rate and power of this
consolidation effect respectively. **Figure 4-3** plots \(D\), obtained by integrating equation [4-5]
from an initial \( D = 1 \), against dimensionless time \( T \), where \( T = c_r t / d^2 \) for a range of \( c_p \) values. The parameters give the solution flexibility to be scaled to match analytical solutions for the dissipation of pore pressures around a laterally loaded pile.

The resulting consolidation process is compared in Figure 4-3 with the dissipation solution by Osman and Randolph (2012), using equation [4-5] integrated with \( c_r = 5 \), for a range of \( c_p \) values. It can be seen that \( c_p = 3 \) provides a reasonable match. However, this comparison relates to consolidation in response to sustained monotonic loading.

![Figure 4-3](image)

**Figure 4-3.** Damage recovery as a function of the dimensionless time for \( c_r = 5 \), compared with the consolidation solution of Osman and Randolph (2011).

Combining equations [4-4] and [4-5], the general variation of damage is:

\[
\delta D = d_r (1 - D)^d_p \left[ \frac{\delta y}{y_r} \right] \left[ \frac{y_r}{y_r} \right] \frac{c_r}{c_p} D^{c_p} \delta t
\]  

[4-6]

Comparing Figure 4-2 and Figure 4-3 it is evident that the accumulation of damage from \( D = 0 \) and the decay of damage from \( D = 1 \) are controlled by the scaled dimensionless cumulative displacement \( (Sd_r) \) and the scaled dimensionless time \( (Tc_r) \) in exactly the same way. Therefore, for a fixed set of constants, the combined response in equation [4-6] is a
function of the scaled dimensionless pile velocity, which controls the distance travelled per unit time, and therefore the relative rates of damage and hardening:

\[ V = \frac{S d_r}{T c_r} \]  

[4-7]

This relationship is demonstrated in Figure 4-4, which shows the development of damage with scaled distance (or time) for \( d_r = 1, \ c_r = 5 \) and \( d_p = c_p = 3 \). All responses stabilise at a constant damage that increases with scaled dimensionless pile velocity. This reflects that at higher velocities there is greater generation, and so a higher damage level is sustained in balance with the dissipation.

**Figure 4-4.** Damage accumulation during steady motion at different rates.

*Evolution of hardening*

Consolidation leads to hardening, following a path in the \( H - \delta u_s/\delta t \) plane dependent on \( \lambda^* \) (see Figure 4-1). When the pile is stationary (i.e., \( \delta u_s/\delta t = 0 \)), \( \frac{\delta H}{\delta D} = -\kappa^*(1 - H)^{h_p} \). This formulation means that \( \kappa^* \) sets the initial slope of the hardening response at \( H = 0 \), and lies in the range 0 to 1. The power coefficient \( h_p \) sets how this slope changes as the limit of \( H = 1 \) is approached. Modifying Equation [4-5], the hardening evolution can be written as:
\[
\frac{\delta H}{\delta t} = c_f (1 - H)^{h_p} \left( \frac{\kappa^* c_v}{d^2} \right) D c_p
\]  

[4-8]

As for the damage model, a minimal number of parameters are used.

To illustrate the hardening model, the variation of \(H\) and \(s_{\text{u}}/s_{\text{u,i}}\) with time is shown in Figure 4-5 using the same range of parameters as used in Figure 4-4 (\(\lambda^* = 0.5, \kappa^* = 0.5, h_p = 2\) and \(S_{\text{a0}} = 5\)). In all cases, the strength evolves towards the limit of \(s_{\text{u,f}}\), but for movement that is more rapid relative to dissipation, there is a fall in strength associated with undrained cyclic loading and pore pressure generation.

![Figure 4-5. Hardening and strength changes during steady motion.](image)

**Illustration of typical model responses**

To illustrate the model response, we firstly present a simulation of the changing strength around a pile during episodic loading (Figure 4-6). In this example, a 1m diameter pile is first subject to 50 cycles of normalised amplitude \(\Delta P/d = \pm 0.1\), thereby moving by a scaled distance of \(S_d = 20\) diameters with \(d_r = 1\). Adopting a 10 second cyclic period, the corresponding dimensionless velocity is \(V > 25,000\) and the 50 cycles take a dimensionless time of \(T < 2 \times 10^4\). The pile is then stationary for a period of \(T = 3.17\), while consolidation occurs, corresponding to \(10^7\) seconds (or 116 days) in soil with \(c_v = 10\) m\(^2\)/year. The
sequence of cycling and recovery is repeated 5 times. The other model parameters adopted are the same as in Figure 4-5.

The time histories of progressive hardening and repeated damage and consolidation are shown in Figure 4-6a to Figure 4-6c. The hardening-strength path is shown in Figure 4-6d, bounded by the limits of $D = 0$ and $D = 1$. The resulting evolution of soil strength is shown in Figure 4-6e.

This example is a highly idealised representation of the changing excitation that a pile or well conductor might experience. However, it shows that the model can capture general patterns of changing strength, associated with arbitrary sequences of movement and damage coupled with ongoing consolidation.

In a second example (Figure 4-7), the model is compared with the strength response from an episodic cyclic T-bar penetrometer test (White & Hodder 2010), with the cylindrical T-bar being comparable to an element of pile. The soil disturbance from each passing of the T-bar is represented by two diameters of pile movement. After each packet of 20 cycles, the T-bar is held stationary and consolidation occurs. The model replicates the test data well, and also matches closely the CSI cycle-by-cycle strength model presented in the same study, which has a similar basis. A distinction between the two models is that in the White & Hodder (2010) model the smallest increment of damage corresponds to a T-bar cycle, whereas the present model is formulated in terms of pile (or T-bar) displacement, and therefore can be applied to general patterns of movement via the PI model, which is described in the next section.
Figure 4-6. Example of episodic cyclic motion: Time histories of (a) damage, (b) hardening history and (c) strength; (d) hardening-strength path and (e) cyclic evolution of strength.
Figure 4-7. Comparison with episodic cyclic strength response in penetrometer test.

4.1.3. Parallel Iwan (PI) model for non-linear cyclic p-y response

4.1.3.1. Overview

The PICS model tracks the softening and hardening behaviour of the soil surrounding the pile, and then uses this to scale the p-y response. This response is formed of parallel Iwan elements (Iwan 1966). These elements allow general forms of cyclic behaviour to be captured, including features such as the non-linearity of the monotonic and cyclic responses with high stiffness at reversal points and a progressive reduction in tangent stiffness as the limiting resistance is approached. A parallel Iwan (PI) model consists of a number of spring-slider elements, each carrying force $f_i$, with different slider capacities ($s_i$) and spring stiffness values ($k_i$) Figure 4-8. PI models have been used previously to describe the p-y response of piles (e.g. Einav 2005, Benckelaers 2015) and the contribution of this paper is to couple the PI and CSI models to capture softening and hardening observed in cyclic loading of piles. In this paper $p$ represents the lateral force per unit length of pile, which has typical units of kN/m, such that $p/d$ is the net lateral pressure on the pile.
Figure 4-8. Parallel Iwan (PI) model and full pile system.

4.1.3.2. Conversion of monotonic backbone curve to PI model parameters

For a monotonic loading event, the PICSi model should reproduce a specified monotonic "backbone" curve. It is therefore convenient to work backwards from a monotonic $p$-$y$ curve to derive the stiffness of each of the parallel springs ($k_i$) and the capacity of the sliders ($s_i$). To do this a backbone curve is first discretised as shown in Figure 4-9.

Values for $p_0$ to $p_n$ and $y_0$ to $y_n$ are then known. The tangent stiffness of each segment ($E_i$) of the backbone curve can be computed as:

$$E_i = \frac{p_i - p_{i-1}}{y_i - y_{i-1}} \quad i = 1 \ldots n \quad [4-9]$$

The tangent stiffness can also be expressed as the sum of all the active parallel springs, which is:

$$E_i = \sum_{j=i}^{n} k_j \quad [4-10]$$
Figure 4-9. Discretisation of a p-y backbone curve.

and can be written in matrix form as:

\[
\{ E \} = [A]\{ k \} \tag{4-11}
\]

where \( A \) is an \( n \) by \( n \) transformation matrix with 1s on and above the diagonal and zeros below. Values for \( k \) can then be found:

\[
\{ k \} = [A]^{-1}\{ E \} \tag{4-12}
\]

The following relationship can then be used to find the slider capacities:

\[
p_i = \sum_{j=1}^{i} s_j + y_i \sum_{j=i+1}^{n} k_j \tag{4-13}
\]

A second \( n \) by \( n \) transformation matrix \( B \) can be introduced containing 1s above the diagonal and zeros on and below the diagonal. The above equation can be written in terms of the two matrices:
where the dot multiplication is used to indicate element by element multiplication of vectors, rather than vector multiplication. This equation can be rearranged to solve for slider capacities:

\[ \{s\} = [[A^T]^{-1}}\{\{p\} - \{y\}.[B][k]\} \]

With the PI parameters \((k_n \text{ and } s_n)\) derived from the monotonic backbone curve, the model can be implemented by noting that the force in any spring \((f_i)\) is the product of the elastic displacement \((u_e)\) and the spring stiffness \((k_i)\):

\[ f_i = k_i u_e^p \]

The elastic displacement is the difference between the total displacement and the plastic displacement \((u_i^p)\).

\[ f_i = k_i (y - u_i^p) \]

If

\[ \text{abs}(f_i) > s_i \]

the capacity of the slider has been exceeded and the value of the plastic displacement must be incremented by the change in total displacement. Noting that the maximum elastic displacement in the spring is \(s_i/k_i\), then the plastic displacement is:

\[ u_i^p = y \pm \frac{s_i}{k_i} \]

As an example, the **Figure 4-10** shows the API soft clay \(p-y\) curve (API 2011, ISO 2016) discretised with 5 points and the corresponding PI model with 5 springs/slider elements subject to 2-way cyclic loading. There is no cyclic degradation in the base PI model and the CSI model is used to represent the degradation and recovery responses observed in model tests.
4.1.3.3. Combining the CSI and PI models

Houlsby et al. (2017) derived a PI ratcheting model within the hyperplastic framework and demonstrated that values of $s_n$ may be varied as functions of the state of the material, without affecting the model formulation. Similarly, to accommodate the changes in strength, and consequently stiffness, the CSI model was linked to the PI response by scaling in the initial slider and spring capacities by the ratio of the current strength to the initial strength:

$$\{s\} = \{s_i\} \frac{s_u}{s_{u,i}} \quad \{k\} = \{k_i\} \frac{s_u}{s_{u,i}}$$  \[4-20\]

This approach assumes that changes in stiffness mirror changes in strength, with both being adjusted from their initial values by the same proportion. However, the two may not be equally affected for all amplitudes of cyclic loading, depending on the underlying micromechanical phenomenon. The model could be extended to have independent scaling approaches to stiffness and strength, if further observations indicate that this is required.
However, as shown by later examples, this approach is successful based on the data presented in this paper.

### 4.1.4. Example application of PICSI model

A centrifuge model test is used to illustrate the capability of the PICSI model to represent typical observations of lateral pile behaviour. Guevara et al. (2020) present centrifuge test results from a rigid length of pile (or conductor) installed in reconstituted carbonate silt. The pile had a diameter $d = 19.5$ mm and an embedded length $4.5d$, with testing taking place at a $g$-level of 40. The pile was fixed against rotation and practically rigid for the range of loads applied.

Two tests are considered. In the first test, the pile was subjected to a one-way undrained monotonic push to the ultimate capacity, as shown in Figure 4-11. The rigid lateral translation of a pile can be represented with a single PI spring in an $F_h-y$ model, where $F_h$ is the total horizontal force (i.e. the integral of $p$ down the length of the pile). To apply the PI model, the measured monotonic lateral response of the pile was discretised and used to evaluate spring stiffness and slider capacity values for the PI model, using the method described above. As shown in Figure 4-11, this approach provides an accurate representation of the monotonic behaviour.

The reconstituted carbonate silt used during the centrifuge tests had a measured strength profile of $s_u = 1.65z$ kPa/m, where $z$ is the depth below the soil surface. Using the approach presented by Jeanjean et al. (2017), and averaging the lateral bearing factor, $N_p$, over the embedded length of the pile (so that $N_p = 11.8$, following Jeanjean et al. 2017), the predicted ultimate capacity is 61.5 N. The load reached in the monotonic test is 65.5 N, which is within 7% of the predicted ultimate capacity.
Figure 4-11. Centrifuge test comparison: Discretisation of monotonic backbone data.

In the second test, the pile was subjected to cycles of displacement-controlled lateral movement with an approximate normalised amplitude of $\delta y/d = \pm 0.04$, at a frequency of $f = 0.625$ Hz. The measured displacement is shown in Figure 4-12, and was used as an input into the PICSI analysis, from which the resulting resistance is calculated. During the initial 500 cycles the stiffness fell, with a minimum peak-to-peak secant stiffness ($K_{p-p} = \Delta F_h/\delta y^{pp}$) of around 7% of the initial peak-to-peak secant stiffness ($K_{p-p0}$). By the end of the test, after 10000 cycles, the stiffness had increased by a factor of two from this softened minimum value.

To simulate this cyclic test, the PICSI model is overlain on the monotonic PI response of Figure 4-11, to capture the changing soil strength and stiffness. The parameter values are listed in Table 4-1 and are based on the theoretical consideration given earlier in the paper, with minor modifications to improve the match with the experimental data. The adopted initial sensitivity, $S_{p0}$, is from cyclic T-bar testing of carbonate silt reported by Zhou et al. (2020). The predicted evolution of cyclic secant stiffness is shown in Figure 4-13 and the full cyclic response is shown in Figure 4-14, compared with the monotonic backbone.
curve, with key cycles are highlighted. The predicted secant stiffness variation and the overall cyclic response agree well with the experimental data.

![Graph showing pile lateral displacement](image)

**Figure 4-12.** Measured lateral displacement of model pile.
Table 4.1. Model and prototype dimensions.

<table>
<thead>
<tr>
<th>Model feature</th>
<th>Parameter Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monotonic response</td>
<td>PI element fitted to monotonic test</td>
<td></td>
</tr>
<tr>
<td>Cyclic response:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength limits</td>
<td>Initial sensitivity</td>
<td>$s_o$ 5</td>
</tr>
<tr>
<td>(Equations [4-1], [4-2])</td>
<td>Slope of $D - 0$ line</td>
<td>$\lambda^*$ 0.5</td>
</tr>
<tr>
<td></td>
<td>Effect of hardening on sensitivity</td>
<td>$q$ 1</td>
</tr>
<tr>
<td>Damage generation</td>
<td>Rate constant</td>
<td>$d_r$ 1.1</td>
</tr>
<tr>
<td>(Equation [4-4])</td>
<td>Power constant</td>
<td>$d_p$ 2.3</td>
</tr>
<tr>
<td></td>
<td>Effect of amplitude</td>
<td>$d_n$ 0</td>
</tr>
<tr>
<td>Consolidation</td>
<td>Consolidation coefficient</td>
<td>$c_v$ 1 m(^2)/year</td>
</tr>
<tr>
<td>(Equation [4-5])</td>
<td>Rate constant</td>
<td>$c_r$ 0.2</td>
</tr>
<tr>
<td></td>
<td>Power constant</td>
<td>$c_p$ 3</td>
</tr>
<tr>
<td>Hardening (Equation [4-8])</td>
<td>Slope of hardening path (Fig. 1)</td>
<td>$\kappa^*$ 0.5</td>
</tr>
<tr>
<td></td>
<td>Variation of hardening slope</td>
<td>$h_p$ 1</td>
</tr>
</tbody>
</table>

Figure 4.13. Centrifuge test comparison: evolution of measured and computed stiffness vs a) dimensionless time $T$, b) Number of cycles.

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Figure 4-14. Centrifuge test comparison: a) the calculated lateral response using PICSI b) measured response.

4.1.5. Discussion

The PICSI model provides a first attempt to capture the complex patterns of changing strength around a pile within a model that can be integrated into a pile response analysis. PI models are increasingly used for cyclic modelling of piles and conductors (e.g. Whyte et al. 2020), and the PICSI model provides an overlay that captures soil softening and consolidation. This opens up the possibility of efficiently modelling the full ‘whole life’ history of changing soil support. This may unlock beneficial effects such as ‘smearing’ of fatigue damage as hot spots migrate along the pile, or a gain in pile capacity for life extensions. It may also provide a basis to interpret changes in system natural period that are observed as a result of evolving pile head stiffness. The model can be implemented within finite element lateral pile analysis software, and the changing strength (CSI) aspect could equally be overlain on other p-y models.

Some model parameters can be derived from conventional soil properties, although others may require alternative methods of calibration. Methods exist to scale the soil stress:strain response seen in direct simple shear (DSS) tests directly to the monotonic p-y response (Zhang and Andersen 2017), and extensions to cyclic behaviour have been proposed (Erbrich et al. 2010, Zhang et al. 2017). The same approach could be used to calibrate the PICSI model parameters from DSS responses, or alternatively the p-y element test
described by (Zakeri et al. 2017) could provide a more direct calibration. The combined effects of softening and consolidation are not usually considered in a single soil element test or a conventional in situ test. However, ‘episodic’ versions of both the T-bar (White and Hodder 2010, Zhou et al. 2020) and the DSS test (Yasuhash & Andersen 1991, Truong et al. 2019, Laham et al. 2020) are possible, and offer the potential to provide model parameters by measuring responses akin to Figure 4-6.

The ICSI model has been developed with soft soils in mind, which lie initially on the ‘wet’ side of the critical state line and therefore show a tendency to contract under loading, generating positive excess pore pressure. Under cyclic loading, even soils that have been over-consolidated to an over-consolidation ratio (OCR) value that places them slightly on the ‘dry’ side of the CSL tend to generate positive pore pressure under cyclic loading (e.g. Andersen 2015), so the model may work well even for these higher OCR soils. It is also possible to conceive a reflected version of the model framework (Figure 4-1) in which the soil moves up the vertical axis, softening rather than hardening, which could represent soils that are initially dilatant and generate negative excess pore pressure, which leads to swelling and softening after dissipation. However, at present, we have not attempted to consider such high OCR cases, and we have focused instead on soft soils, where the opposing effects of cyclic softening and consolidation hardening are topical and offer potential design optimisation in relation to ‘whole life design’ (e.g. Lai et al. 2020, Laham et al. 2021, Gourvenec 2020, Guevara et al. 2020).

4.1.6. Conclusions

This paper sets out a new p-y model for the long term ‘whole life’ behaviour of laterally-loaded piles in soft clay. It addresses an emerging requirement to capture the progressive changes in soil support that occur in soft soils around piles and well conductors, which influence the capacity, stiffness and fatigue of these systems.

The model is inspired by model testing observations and theoretical solutions for each element of the behaviour, and combines a parallel-Iwan (PI) non-linear spring with a critical state-inspired overlay for the changing strength and stiffness. It allows the general responses observed in model tests to be replicated, with many parameters being fixed based on theoretical considerations. A new efficient methodology for defining the PI sub-springs is set out.
This new basis for whole life modelling of piles and well conductors, allowing changes in stiffness and capacity to be simulated, may lead to beneficial improvements in predictions of system stiffness, fatigue and late-life capacity.

**Acknowledgements**

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**References**


4.2. A methodology to calibrate the PICSi cyclic p-y model using experimental results and optimisation

Abstract

Fatigue life estimation of the components of a drilling conductor system is often a critical design consideration. The analysis of fatigue life is usually performed using a coupled system, with the soil-conductor interaction modelled using p-y springs that remain constant throughout the analysis. This means that shifts in bending moment profile that occur due to degradation and recovery of the soil during or after cycling are not accurately modelled. While the PICSi framework (White et al. 2022) can model changes in stiffness and strength of p-y curves due to cycling and pore-pressure dissipation, guidance has not yet been provided on how to calibrate the framework parameters. This paper presents an experimental methodology for this calibration process based on centrifuge and p-y model testing in reconstituted carbonate silt and kaolin clay. The procedure uses numerical optimisation, where an error function is computed as the average normalised difference between the measured and calculated secant p-y stiffness values. The calibrated parameters are validated against results from an independent set of centrifuge tests on carbonate silt using a flexible pile of similar dimensions to a drilling conductor. It is found that the calibrated model is able to match the changes in cyclic bending moment through a sequence of different packets of cycles with increasing and decreasing amplitudes, and with intermittent pause periods. This calibration procedure provides an objective approach for more accurate modelling of conductor fatigue.

4.2.1. Introduction

Conductors are the outer-most casing of a drilling system. They prevent the walls of the hole from collapsing and provide axial support for the heavy safety components and inner casings of the system. They are connected to the lower packages (BOP, LMRP) which are in turn connected to the riser system, which is connected to the drill floor, as shown in Figure 4-15. When the drilling vessel is impacted by environmental loads, the whole system sways laterally and the embedded riser interacts with the surrounding soil. The behaviour of the soil therefore affects the system response.
Among the various checks that must be performed to ensure a safe drilling operation, fatigue life estimation of the components of the system is often a critical design consideration. Fatigue occurs when repeated stress cycles at specific locations (or hot-spots) produce cracks that could progress to major damage (or failure) of the system. Hot-spots often occur where stress is concentrated, such as discontinuities in the geometry, welds, changes in diameter or thickness, or changes in stiffness such as those produced at the top of cement between the conductor and casing (DNV GL 2018). Although most hot-spots occur on the wellhead and upper portions of the conductor, some can be located below mudline. Accurate calculation of stress along the length of the conductor and casing string is therefore important for determining the fatigue life of individual system components.

Traditionally, fatigue life is determined by running coupled time-domain analyses of the whole system including the vessel, riser, lower components, conductor, casing and soil. These analyses are performed for a range of seastates that the system will operate in, and the accumulated fatigue from damage in each condition is found to determine the fatigue life. The soil is often modelled using non-linear Winkler springs (using p-y curves) that maintain a constant load-displacement response throughout the analysis. While more refined analysis can be performed by decoupling the system at the lower flex joint (DNV GL 2018) and modelling the soil as a continuum in a finite element (FE) software, recent experimental studies (Guevara et al. 2022a) have shown that p-y curves can provide an acceptable prediction of bending moment profile.
For monotonic loading, the current API RP 2GEO guidelines (API 2014) recommend p-y curves that are widely acknowledged to under-predict the lateral stiffness at small displacements (Jeanjean 2009). Typical operational displacements for conductors are below 10% of the conductor diameter (d) and are therefore in the range where the API curves do not predict the stiffness accurately. More recent studies (Jeanjean et al. 2017) have proposed p-y curves that reflect the observed results from centrifuge tests and databases of field experience. These studies are being incorporated in the draft ISO 19901-4 (ISO/DIS 2021) with certain modifications to the p-y curves. Other authors have also proposed methods of constructing monotonic p-y curves based on scaling from laboratory simple shear test results (Bransby 1999; Zhang and Andersen 2017).

Since the environmental loading on a drilling conductor system is cyclic, monotonic p-y curves are unlikely to adequately capture the changes that occur in the soil-conductor stiffness over the operational life of the system, and other (recent) approaches have therefore been proposed to model this scenario. One approach relies on the accumulation of pore pressures from different amplitude cycling loading to determine a factor by which the monotonic curve must be modified (Zhang et al. 2017). Other authors (Zakeri et al. 2019) propose methods for estimating a characteristic “steady-state” (cyclic) stiffness that is assumed to be reached after a several hundred cycles, with this method originally developed based on observations from centrifuge testing in kaolin clay and Gulf of Mexico clay (Zakeri et al. 2016a; b), as well as p-y apparatus tests (Zakeri et al. 2017) on other types of clay. A recent method (Komolafe and Aubeny 2020) extends the “steady state” stiffness approach by modelling the transition from undisturbed stiffness to the fully remoulded state, for the range of displacements expected to be experienced by drilling conductors. However, none of these approaches can model the impact of the entire load history on soil-conductor stiffness.

Previous studies for steel catenary risers (Hodder et al. 2009; Sahdi 2013) and pipelines (Hou 2020) have shown the impact of load history on soil-pipe stiffness through episodes of cycling and consolidation, whereby an initial loss of stiffness is then regained (after cycling stops) due to pore pressure dissipation. Furthermore, these studies have also demonstrated that a regain in stiffness and strength can be achieved while cycling, provided the rate of pore pressure dissipation and recovery exceeds the rate of damage induced by remoulding. Similar changes in stiffness have been observed in laterally loaded short piles.
from experimental studies in centrifuge. Here, the regain in stiffness and strength due to episodic loading followed by consolidation periods was documented for laterally loaded piles in kaolin clay by Zhang et al. (2011), and in reconstituted carbonate silt by Doherty et al. (2019). Moreover, an increase in soil-pile stiffness was found to occur cycling by Guevara et al. (2020) for tests with large numbers of cycles (N > 10000). Guevara et al. (2020) also report a higher regain in stiffness from continuous cyclic loading when compared to multiple (short) episodes of loading with consolidation periods in between, to achieve the same overall timeframe. This observation is comparable to that documented by Hou (2020) for steel catenary risers, suggesting that in soft soils, the more pore pressure that is generated through remoulding, the higher the subsequent densification that can occur – leading to a greater potential regain in stiffness. Furthermore, based on centrifuge tests of rigid piles in reconstituted carbonate silt and kaolin clay, Guevara et al. (2020, 2021b) demonstrate that the sequencing of cyclic loading is important, as large amplitude cycling can have a significant impact on subsequent smaller amplitude response.

Aside from model testing, the effects of load-history have been observed through multiple episodic simple shear tests on kaolin clay (Laham et al. 2021) and multiple amplitude partially drained cyclic triaxial tests in sand (Jostad et al. 2021).

Frameworks have previously been proposed to model the effect of episodes of cyclic loading and consolidation on soil strength and stiffness for different applications (White and Hodder 2010; Zhou 2019). These model the effects of remoulding and consolidation as separate processes and capture the corresponding changes in bearing capacity. Building on these models, White et al. (2022) recently proposed a method (PICS I) to model the p-y response of laterally-loaded piles, with the p-y spring being continuously updated based on the effects of load history, inspired by traditional critical state soil mechanics (Wood 1990), and with additional features observed from centrifuge tests. This method allows for remoulding and consolidation to occur in parallel, with the p-y spring updated accordingly.

While some examples of its application were given, no guidance was provided on how the PICS I model could be calibrated. Accordingly, this paper describes a method for calibrating the model based on experimental (centrifuge and p-y apparatus) test results. It is acknowledged that these tests are not routinely performed, and an alternative strategy would be to develop methods to derive these parameters from simple shear or triaxial tests, which are more commonly performed as part of site characterisation activities. In addition
to the calibration method, this paper presents a modification to the original PICSI formulation, which led to a reduction in the number of model parameters to calibrate.

4.2.2. The PICSI framework

The Parallel-Iwan Critical-State Inspired (PICSI) model proposed by White et al. (2022) tracks soil softening produced by lateral displacement of a pile, as well as hardening caused by the dissipation of the excess pore pressure. The $p$-$y$ model consists of two components:

- A parallel Iwan (PI) model (Iwan 1966) for the hysteretic non-linear $p$-$y$ response, which consists of parallel springs and sliders (Figure 4-16).
- A critical state-inspired (CSI) model for hardening and softening of the $p$-$y$ response.

The net effect of the softening (damage) and hardening varies with time and is used to scale the springs stiffness ($k_i$) and sliders capacities ($s_i$) within the Parallel-Iwan model (Figure 4-16).

The model uses an analogue of the voids ratio – strength relationship that underlies critical state soil mechanics, with additional features to replicate observations from model tests. The voids ratio is replaced with a hardening index $H$, and the mean effective stress is replaced by the undrained shear strength normalised by the initial strength, $s_u / s_{u,i}$. Figure 4-17a shows envelopes of the attainable states of strength and hardening. The shape of the lower portion of the envelope depends on the equation chosen to describe it. Detailed explanation is provided later in the paper.
Figure 4.16. Parallel-Iwan arrangement of springs and sliders of stiffness $k_i$ and capacity $s_i$.

Figure 4.17. Model strength envelopes: a) original formulation, and b) modified formulation.

The damage, $D$, is caused by shearing of the surrounding soil when the pile is displaced laterally, $y$, and cannot exceed $D = 1$, which is shown in Figure 4.17 as the lower-left envelope and is analogous to the critical state line CSL (Wood 1990). Damage recovers with time due to pore pressure dissipation (consolidation). The function that describes the incremental change of damage $\delta D$ for a normalised displacement $(\delta y / d)$ and time increment $\delta t$ is:
\[ \delta D = d_r (1 - D)^{d_p} \frac{y}{y_{ref}} \left| \frac{\delta y}{d} \right| - c_r \left( \frac{c_v}{d^2} \right) D^{c_p} \delta t \]  

[4-21]

The equation consists of two parts:

- The first part corresponds to the increase in damage produced by a given normalised displacement increment (\(\delta y/d\)). Damage also depends on the absolute displacement (\(y\)) normalised by the peak displacement (\(y_{ref}\)), as well as the current accumulated damage (\(D\)). Fitting parameters \(d_r\), \(d_p\) and \(d_a\) enable the relative contribution of these components to be adjusted in the calibration process.

- The second part of the equation corresponds to recovery from damage through the dissipation of excess pore pressures, which is dependent on the coefficient of consolidation (\(c_v\)), the conductor diameter, and parameters that control the rate of recovery \(c_r\) and \(c_p\).

The densification of soil due to the dissipation of the pore pressures generated during cycling is modelled by the hardening index, \(H\). The variation in hardening with time can be described with the following equation:

\[ \frac{\delta H}{\delta t} = c_r (1 - H)^{h_p} \left( \frac{\kappa^* c_v}{d^2} \right) D^{c_p} \]  

[4-22]

The current level of hardening will depend on accumulated damage, the consolidation parameters \(c_r\) and \(c_p\), the coefficient of consolidation of the soil (\(c_v\)), and the hardening parameters \(h_p\) and \(\kappa^*\).

The current normalised strength (\(s_u/s_{u,i}\)) depends on the current levels of hardening (\(H\)) and damage (\(D\)). An equilibrated strength (\(s_{u,e}\)) is defined as the strength at the current \(H\) when \(D = 0\):

\[ \frac{s_{u,e}}{s_{u,i}} = 1 + \frac{H}{\lambda^*} \]  

[4-23]

The current strength (\(s_u\)) is therefore:
\[
\frac{S_u}{S_{u,e}} = 1 - D \left(1 - \frac{1}{S_t}\right)
\]  

[4-24]

A general form of the model (Figure 4-17a) allows soil sensitivity to vary with hardening, from \( S_t = S_{t0} \) at \( H = 0 \) to \( S_t = 1 \) at \( H = 1 \), at a rate set by the power, \( q \):

\[
S_t = 1 + (S_{t0} - 1)(1 - H)^q
\]  

[4-25]

In this paper, a simplification is proposed for calculating the soil sensitivity by assuming the \( D = 1 \) envelope varies linearly with hardening, plotted as the dashed line in Figure 4-17b. This can be achieved by replacing equation (5) with the following equation, with model then independent of the parameter \( q \):

\[
S_t = \frac{S_{t0}(H + \lambda^*)}{\lambda^*(1 - H) + S_{t0}H(\lambda^* + 1)}
\]  

[4-26]

As will be shown in following sections, this simplification provides acceptable agreement with the data, while reducing the number of parameters needed to be calibrated. It is important to note that although some tests reached a large number of cycles (\( N \approx 20000 \)), the strength appears to continue increasing rather than reaching a plateau (as it would when \( H \to 1 \)). This implies that more cycles would be needed to fully calibrate the model, and as such the simplification proposed in this paper (as well as the calibrated parameter \( \lambda^* \)) represents only a first approximation – which may be updated in future using tests where \( H = 1 \) (i.e. \( s_{u,r} = s_u \)).

Another important aspect to note in the PICSI model is that by formulating the recovery from damage as per White et al. (2022), the strength will always be higher than the initial strength once pore pressures induced by cyclic loading have fully dissipated. This is a good approximation for materials for which soil structure, ageing and cementation between particles do not play a large role in the sensitivity of the soil. For soils where these aspects are important, it may be possible to model these by introducing a ‘non-recoverable strength’ term, such as through modification of equations [4-23] to [4-25] or [4-26]. This is the subject of further work, requiring testing of undisturbed natural soils.

Implementation of the model is performed via a sequential analytical process, documented more fully in White et al. (2022). Figure 4-18 shows the procedure at a spring level when subjected to a time-history of displacements \( y(t) \) – and it is observed that at each time-
increment ($\delta t$), the strength and stiffness of the parallel-Iwan system is affected by both the current level of damage $D$ and hardening $H$ produced by the time history of displacement.

**Figure 4-18.** Framework procedure at a p-y spring level.

### 4.2.3. Experimental data

Physical modelling was performed to observe key aspects influencing the stiffness of soil-conductor lateral behaviour, and to calibrate the PICS1 model parameters. Details on the
observations derived from the centrifuge and p-y apparatus tests can be found in Guevara et al. (2020, 2021b; c). From these tests the horizontal forces/pressures and displacements with time were extracted to calibrate the model by tracking changes in normalised cyclic secant stiffness ($K$), defined as:

$$ K = \frac{\Delta P / P_u}{\Delta y / d} $$

[4-27]

where $\Delta p$ is the cyclic range of horizontal pressure; $P_u$ is the measured lateral capacity from a monotonic test; $d$ is the pile diameter; and $\Delta y$ is the range of lateral displacement (peak to peak).

4.2.3.1. Soil characterisation

Two different reconstituted samples were used for the tests described in this paper: a carbonate silt recovered from offshore Australia (Chow et al. 2019) and UWA kaolin clay (Stewart 1990). The properties of carbonate silt and kaolin clay were obtained through laboratory classification and element tests in accordance with relevant Australian and ASTM standards, and are summarised in Table 4-2.

<table>
<thead>
<tr>
<th>Description</th>
<th>Parameter</th>
<th>Carbonate Silt</th>
<th>Kaolin Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undrained shear strength gradient</td>
<td>$s_u / \sigma_\phi'$</td>
<td>0.32</td>
<td>0.22</td>
</tr>
<tr>
<td>(from CK0U SS tests sheared at 22% / h)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sensitivity (from cyclic T-bar tests)</td>
<td>$S_t$</td>
<td>3.3</td>
<td>2.4</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>$G_s$</td>
<td>2.76</td>
<td>2.6</td>
</tr>
<tr>
<td>(Boukpeti et al. 2012)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Increase in strength due to rate effects</td>
<td></td>
<td>18% per log cycle</td>
<td>9% per log cycle</td>
</tr>
<tr>
<td>Operative coefficient of consolidation</td>
<td>$c_v$</td>
<td>(consistent with Chow et al. 2019)</td>
<td>(consistent with Richardson et al. 2009)</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>$LL$</td>
<td>63</td>
<td>61 (Stewart 1990)</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>$LP$</td>
<td>42</td>
<td>27 (Stewart 1990)</td>
</tr>
<tr>
<td>T-bar strength gradient with depth</td>
<td>$k_{T-bar}$</td>
<td>1.65 kPa/m</td>
<td>1.05 kPa/m</td>
</tr>
</tbody>
</table>

Samples for element testing were reconstituted with moisture content ($w$) of 140% for the carbonate silt and 130% for the kaolin clay, roughly twice the liquid limit. The slurry was
then consolidated in tubes under a vertical pressure representing an embedment depth of 6.0 m for the carbonate silt (30 kPa) and 11.0 m for the kaolin clay (65 kPa). The vertical stress was selected to ensure that a sample with strength ($s_u$) of 10 kPa would be produced, as this was needed for handling and placing the samples in the $p$-$y$ apparatus. The soil strength gradient ($s_u/\sigma'_v$) and increase in strength due to rate effects were determined via CK$_0$U simple shear tests at slow and fast rates. Cyclic T-bar penetrometer tests were also performed on the centrifuge samples to measure soil strength gradient with depth ($k_{T-bar}$) and sensitivity, derived using a capacity factor $N_{T-bar}$=10.5 (Stewart and Randolph 1991). Profiles of moisture content, unit weight, undrained shear strength from T-bars and monotonic simple shear tests are presented in Figure 4-19.

![Figure 4-19](image)

**Figure 4-19.** Moisture content, effective unit weight and undrained shear strength profiles for a) Carbonate silt and b) Kaolin clay.

It is observed from Figure 4-19a and b that the extrapolation of the T-bar strength gradient with depth coincides with the slow simple shear tests, despite the T-bars being performed at higher strain rate than the slow simple shear tests. This is thought to relate to partial soil softening induced during initial penetration of the T-bar offsetting the increase due to the loading rate (Einav and Randolph 2005; Zhou and Randolph 2009a; b).
4.2.3.2. Centrifuge tests

Centrifuge testing involved the laterally translation of a rigid pile installed in reconstituted normally consolidated samples of kaolin clay and carbonate silt at 40g. The moisture content \( (w) \) of each slurry was the same as for samples consolidated for element testing, as described in the previous section. The tests were conducted in the 1.8m radius beam centrifuge at the National Geotechnical Centrifuge Facility, University of Western Australia. The experimental setup is shown in Figure 4-20 and more details can be found in (Guevara et al. 2021).

![Diagram of test setup](image)

**Figure 4-20.** Diagram of test setup: (a) First campaign setup; (b) Second campaign setup. From (Guevara et al. 2020)

The model pile was 3D printed in stainless steel to represent a prototype conductor with a diameter of 780 mm (30 inch) and a wall thickness of 38 mm (1.5 inch). The pile was connected to a bending leg, which was used to determine the lateral load \( (F_N) \) applied to the conductor. The displacement and rotation of the pile were measured using two lasers and a flat plate bracket as a target. The hollow pile was installed in flight by monotonic jacking. Test locations were separated by at least 6 pile diameters from each other and the walls of the box to avoid interaction between tests.

The tests were displacement controlled at the actuator level to best represent the nature of the loading from a floating drilling vessel, and the pile can be considered rigid for the range of loading applied. The test sequences are described in Table 4-3 and Table 4-4 for the
carbonate silt and the kaolin clay, respectively. The amplitudes at mid-depth are expressed in terms of half the range of normalised lateral displacement (peak to peak amplitude) \( \Delta y/2d \).

**Table 4-3. Centrifuge test sequences – carbonate silt.**

<table>
<thead>
<tr>
<th>Test number</th>
<th>Embedment Depth</th>
<th>Amplitude at mid-depth, ( \Delta y/2d ) (-)</th>
<th># of cycles (N) per packet of amplitude</th>
<th>Number of episodes</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS1</td>
<td>4.5d and 6.0d</td>
<td>Monotonic push</td>
<td>-</td>
<td>1</td>
</tr>
<tr>
<td>CS2</td>
<td>4.5d and 6.0d</td>
<td>Packet 1: 0.0006</td>
<td>Packet 2: 0.0009</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 3: 0.022</td>
<td>Packet 4: 0.0059</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 5: 0.0144</td>
<td>Packet 6: 0.0284</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 7: 0.037 – PF</td>
<td>Packet 8: 0.038 – PF</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 9: 0.039</td>
<td>Packets 10 to 18: symmetrically “unloaded” via matching cyclic packets.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>150</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>CS3</td>
<td>4.5d and 6.0d</td>
<td>Packet 1: 0.0035</td>
<td>Packet 2: 0.0113</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 3: 0.0346</td>
<td>Packet 4: 0.0186</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 5: 0.0087</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>100</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>CS4</td>
<td>4.5d and 6.0d</td>
<td>Packet 1: 0.005</td>
<td>Packet 2: 0.0183</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 3: 0.0669</td>
<td>Packet 4: 0.0320</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 5: 0.0130</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>200</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>CS5.1</td>
<td>4.5d</td>
<td>Packet 1: 0.0205</td>
<td>Packet 2: 0.03</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 3: 0.024</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>200 (small amplitude), 2 (larger amplitude)</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>CS5.2</td>
<td>4.5d</td>
<td>Packet 1: 0.022</td>
<td>Packet 2: 0.033</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 3: 0.023</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>200 (small amplitude), 20 (larger amplitude)</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>CS5.3</td>
<td>4.5d</td>
<td>Packet 1: 0.019</td>
<td>Packet 2: 0.042</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 3: 0.028</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>200 (small amplitude), 200 (larger amplitude)</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>CS6</td>
<td>4.5d</td>
<td>0.05</td>
<td>200</td>
<td></td>
</tr>
<tr>
<td>CS7</td>
<td>4.5d</td>
<td>0.048 to 0.029</td>
<td>10000</td>
<td></td>
</tr>
</tbody>
</table>

PF: Monotonic Push forward 0.10d

Generally, each test comprised episodes of cyclic loading with pore pressure dissipation (pause) periods in between, with each episode comprising a packet of cycles of constant
amplitude. The frequency and number of cycles of loading within an episode was selected to ensure that each episode was effectively undrained (≤ 25% dissipation), which was estimated using the Osman and Randolph (2012) dissipation solution. Using the same approach, the period between episodes was set to ensure ≈ 75% dissipation of the pore pressures generated in the last episode. This percentage was later compared with the data from the pore pressure sensors on the pile and found to be around 76% for the kaolin clay tests, and 65% for the carbonate silt. The packet sequence increases and decreases in amplitude symmetrically with respect to the largest amplitude packet.

Table 4-4. Centrifuge test sequences - kaolin clay.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Embedment Depth</th>
<th>Amplitude at mid-depth, Δy/2d (-)</th>
<th># of cycles (N) per packet of amplitude</th>
<th>Number of episodes</th>
</tr>
</thead>
<tbody>
<tr>
<td>KC1</td>
<td>4.5d and 6.0d</td>
<td>Monotonic push</td>
<td>-</td>
<td>1</td>
</tr>
<tr>
<td>KC2 (2-way loading)</td>
<td>4.5d and 6.0d</td>
<td>Packet 1 and 2: 0.0009</td>
<td>Packet 5: 0.0217</td>
<td>150</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 3 and 4: 0.0091</td>
<td>Packet 6: 0.035 – PF</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 7: 0.035 – PF</td>
<td>Packet 8: 0.035</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packets 9 to 15: symmetrically “unloaded” via matching cyclic packets.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>KC3 (2-way loading)</td>
<td>4.5d and 6.0d</td>
<td>Packet 1: 0.0057</td>
<td>Packet 2: 0.0209</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 3: 0.0459</td>
<td>Packet 4: 0.0220</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 5: 0.0115</td>
<td></td>
<td></td>
</tr>
<tr>
<td>KC4 (1-way loading)</td>
<td>6.0d</td>
<td>Packet 1: 0.0033</td>
<td>Packet 2: 0.0210</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 3: 0.0687</td>
<td>Packet 4: 0.0296</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Packet 5: 0.0111</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

PF: Monotonic Push forward 0.10d

Although aiming for pure lateral translation, the presence of the load cell and connection pieces created a very small compliance in the loading system. This was accounted for by developing a relationship between the applied load and the pile rotation (measured with the two lasers), which was then used to determine the lateral displacement at 50% of the pile tip depth. The integrated response of the distributed soil pressure (p) along the length of the pile was determined via the measured lateral load (Fh) and the projected embedded portion of the pile (i.e Fh/dL). The lateral displacement at mid-embedment depth (Δy/d) and the
pressure measured on each cyclic test ($\Delta P$), along with the ultimate pressure measured from the monotonic push ($P_u$) were then used to determine the normalised secant stiffness ($K$) during each cycle. It should be noted that the measured lateral load includes contributions from both (near surface) wedge and (deeper) flow around mechanisms. A contribution also occurs due to shearing at the base of the pile, although this is small and can be ignored. A discussion is presented in the results section on how this might impact the calibration of the model.

### 4.2.3.3. p-y apparatus tests

The p-y apparatus tests were performed in Norway using equipment designed by BP and operated by NGI (Zakeri et al. 2017), using samples consolidated from slurry in tubes and with an internal diameter of 72 mm. The consolidation pressures applied in preparing the samples were the same as those used for element testing. The p-y apparatus uses a 10 mm diameter rod inserted in a pre-augered hole in a sample with height-to-width of between 1:1 and 2:1. The sample was consolidated with end plates on both ends from which the inserted rod protrudes. Once the sample is consolidated in the apparatus, the rod is moved laterally while maintaining constant volume/height with the end plates. The consolidation pressures adopted for testing were 3 kPa greater than the pressures used during sample preparation, in order to minimise potential disturbance of the prepared samples due to transportation from Perth to Norway. The test sequences applied in the p-y apparatus for the carbonate silt and kaolin clay samples are shown in Table 4-5.

#### Table 4-5. p-y apparatus test sequences – carbonate silt and kaolin clay.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Amplitude, $\Delta y/2d$ (-)</th>
<th># of cycles (N) per packet of amplitude</th>
<th>Number of episodes</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS1_py and</td>
<td>0.04 – 0.08 – 0.04</td>
<td>250 (small amplitude), 25 (larger amplitude)</td>
<td>3</td>
</tr>
<tr>
<td>KC1_py (2-way loading)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CS2_py and</td>
<td>0.04 – 0.08 – 0.04</td>
<td>250 (small amplitude), 250 (larger amplitude)</td>
<td>3</td>
</tr>
<tr>
<td>KC2_py (2-way loading)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CS3_py and</td>
<td>0.35</td>
<td>100</td>
<td>1</td>
</tr>
<tr>
<td>KC3_py (2-way loading)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CS4_py and</td>
<td>0.0004 – 0.002 – 0.009 – 0.04</td>
<td>100</td>
<td>1</td>
</tr>
<tr>
<td>KC4_py (2-way loading)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The applied displacement ($\Delta y/2d$) and measured pressure ($\Delta P$) on the rod, along with the ultimate pressure ($P_u$) from the respective large amplitude monotonic tests (CS3_py and
KC3_py), were then used to determine the normalised secant stiffness \( K \) during each cycle (Equation [4-27]). The pressure from the first peak of the monotonic push test is used to normalise the stiffness results from cyclic testing, which is the most straightforward choice of ultimate pressure for normalising the p-y apparatus tests results. However, as discussed in Guevara et al. (2022b) this may be influenced by rate and boundary effects.

4.2.4. Calibration

The calibration method proposed in this paper requires both monotonic and cyclic tests, with the latter exploring differences in cyclic amplitude, load history and time for pore pressure dissipation. For this calibration exercise, three tests have been used with either a large number of constant amplitude cycles or multiple episodes of cycling, and three tests with increasing amplitude cycling. The tests were taken from two-way loading tests, although one-way loading tests were also performed in the centrifuge campaigns.

4.2.4.1. Calibration of the parallel-Iwan model

For each soil type, embedment depth and test type (centrifuge or p-y apparatus) a monotonic push was performed. Each of these tests was used for the calibration of spring stiffness and slider capacity in the parallel-Iwan model, to then be used in its corresponding set of cyclic tests. Figure 4-21 shows the measured monotonic backbone curves and the discretised PICS backbone curves that were transformed into a parallel-Iwan system for each monotonic test performed in the centrifuge. Figure 4-22 shows the same set of curves for the tests performed in the p-y apparatus. The p-y curve is modelled up to the first peak, and any post-peak softening is not taken into account. Displacements beyond the peak at which the monotonic curve is modelled will mobilise the maximum pressure in the backbone curve minus the damage produced by the pore pressures accumulated up to that displacement. A current limitation of the model is that it does not include rate effects in its formulation, which is overcome in this study by performing all tests at a similar rate (roughly 1 - 3 mm/s at model scale).
Figure 4-21. Monotonic backbone curves from centrifuge tests discretised for PICS.

Figure 4-22. Monotonic backbone curves from p-y apparatus discretised for PICS.

4.2.4.2 Calibration of hardening and pore pressure dissipation parameters

The calibration procedure uses numerical optimisation, where an error function \( E \) was computed as the average normalised difference between the measured \( K_{test} \) and calculated
(\(K_{\text{PICSI}}\)) secant stiffness values for each of the \(N\) cycles in a test (see Eq. [4-28]). To avoid an overweighting by tests with large number of cycles, the error was divided by the number of cycles \((N)\) in the respective test. The normalised errors were then summed up for all the tests used in the optimisation.

\[
E = \left( \sum_{i=1}^{N} \frac{K_{\text{test},i} - K_{\text{PICSI},i}}{K_{\text{test},i}} \right) \frac{1}{N}
\]  

[4-28]

The PatternSearch optimisation algorithm in Matlab (Mathworks 2021) was used. The algorithm terminates when any of the following are satisfied:

- The change in the function value is less than a defined tolerance;
- The mesh size (or variation in the input parameters) is less than a defined tolerance;

or

- The number of function evaluations reaches a defined number.

**Fig. 4-23** illustrates of the optimisation process.

The list of PICSI parameters is given in **Table 4-6**. It was found to be more efficient to carry out parameter optimisation in two stages:

- In the first stage, the hardening parameters were assigned upper and lower bounds within plausible ranges, while the values of damage parameters were fixed to values from a previous manual fitting. Once the values for the consolidation and hardening parameters were selected, these were then constrained and the damage parameters optimised in the second stage.

- A second iteration was performed by fixing the optimised damage parameters and optimising the values for the consolidation and hardening parameters, then fixing the optimised consolidation and hardening parameters and optimising the damage ones. In total, five iterations were performed until the difference on the value of the parameters between iterations was negligible. The difference in the error function value between the first and last iteration was less than 1%, and the average error value on the last iteration for all the tests analysed was 21%.

The parameters \(\lambda^*, \kappa^*, c_p, c_r\) and \(h_p\) control strength and stiffness gain from hardening and consolidation, and can only be evaluated using tests where sufficient time was allowed to dissipate pore pressures. For the carbonate silt these parameters were therefore calibrated
using tests CS5.2 and CS6 (which both have a large cyclic loading episode and consolidation periods), as well as test CS7 (which comprised a large single episode of 20000 cycles). For the kaolin clay, test KC3 at both embedment depths and test KC4 were used to calibrate the hardening and consolidation parameters.

![Flowchart](image)

**Fig. 4.23. Optimisation process flowchart.**

4.2.4.3. Damage parameters

The damage parameters \((d_p, d_r, d_\alpha)\) were calibrated using only the first episode of each test, during which the influence of pore pressure dissipation is minimal. Regarding the centrifuge tests, only the increasing amplitude phase of the first event of each test was used. As observed by Guevara et al. (2020, 2021a), once sufficient cycles at a large amplitude have occurred, the stiffness of subsequent smaller amplitude cycles may be impacted. This finding is clear in the centrifuge tests, where both shallow wedge and flow around mechanisms are mobilised; but is less evident in the p-y apparatus, where the stiffness of the subsequent smaller amplitude cycles appears to recover to the fully remoulded state observed before the large amplitude cycling occurred. This is thought to relate to
differences in confinement of the upper portion of the soil in the p-y apparatus and the centrifuge tests - for the centrifuge tests, the near seabed the soil is free to deform in the vertical direction (without constraint from the soil above); whereas in the p-y apparatus, the soil is fully constrained above resulting in a deep flow around mechanism.

The tests selected for optimising the damage parameters for the carbonate silt were CS2 at $4.5d$ and $6.0d$ embedment and CS4_py, and the algorithm described in the previous section was used. For the kaolin clay, the tests used to optimise the damage parameters were KC2 and KC3 at $4.5d$ embedment depth, and KC4_py.

### 4.2.4.4. Calibration results

Table 4-6 shows the optimised parameters for carbonate silt and kaolin clay obtained from the calibration procedure, which were then used to model the complete range of test loading sequence and compared directly to test results. Select results for carbonate silt are shown in Fig. 4-24, Figure 4-25 and Figure 4-26, while select results for kaolin clay are shown in Figure 4-27.

<table>
<thead>
<tr>
<th>Model feature</th>
<th>Description</th>
<th>Parameter</th>
<th>Carbonate silt</th>
<th>Kaolin clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength limits</td>
<td>Slope of Damage = 0 line</td>
<td>$\lambda^*$</td>
<td>8.69</td>
<td>0.33</td>
</tr>
<tr>
<td>Damage generation</td>
<td>Rate constant</td>
<td>$d_r$</td>
<td>2.39</td>
<td>32.33</td>
</tr>
<tr>
<td></td>
<td>Power constant</td>
<td>$d_p$</td>
<td>1.68</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td>Effect of amplitude</td>
<td>$d_a$</td>
<td>1.00</td>
<td>0.99</td>
</tr>
<tr>
<td>Pore pressure dissipation</td>
<td>Rate constant</td>
<td>$c_r$</td>
<td>0.30</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>Power constant</td>
<td>$c_p$</td>
<td>9.65</td>
<td>5.00</td>
</tr>
<tr>
<td>Hardening</td>
<td>Slope of hardening path</td>
<td>$\kappa^*$</td>
<td>0.31</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>Variation of hardening slope</td>
<td>$h_p$</td>
<td>0.002</td>
<td>0.010</td>
</tr>
</tbody>
</table>

Fig. 4-24 shows results of the two 10000-cycles episodes single amplitude test CS7 (Table 4-3) alongside the PICSI model. The regain in secant stiffness from the model shows good agreement with the experimental data, including the sharp jump in stiffness after the pause period. However, the shape of the loading-unloading-reloading p-y loops is not accurately matched – the model does not yet include additional damping as part of its formulation, and therefore the shape of each loop is directly dependent on the monotonic response used to calibrate the parallel-Iwan spring system.
**Fig. 4-24.** Two-way centrifuge test on carbonate silt at 4.5D embedment depth: two episodes of 10000 cycles single amplitude packets.

The ability of the model to predict multi-amplitude cyclic loading is shown in **Figure 4-25a**, where results of a multi-amplitude test (CS2) at 4.5d embedment are presented alongside the PICSI results. The secant stiffness from the PICSI model shows good agreement with the experimental data prior to the peak cyclic load, while the post peak behaviour is not accurately modelled – as discussed previously. In contrast, **Figure 4-25b** shows that stiffness measured with the $p-y$ apparatus (CS4.py) does return to a higher value for smaller amplitude cycling after a large amplitude cycling loading – and this is captured reasonably well by the PICSI model.

**Figure 4-26** presents the comparison for centrifuge test CS4 (embedded at 4.5d) which comprised one-way multi-amplitude cyclic episodes with pore pressure dissipation periods.
As observed in test CS2 (Figure 4-25a), the post-peak stiffness does not return to its pre-peak value – although the PICSi model provides a good match up to this point.

The current formulation of PICSi does not capture scenarios where the pile translates into non-remoulded material, which is reflected in the experimental data (Figure 4-26) as a tendency for the strength to return to the backbone curve when subjected to displacements larger than those experienced during cycling. While potentially important when designing for pile capacity, this is not expected to be significant for assessment of conductor fatigue as these analyses are conventionally performed considering no drift (i.e. 2-way loading).
Figure 4-25. Results of a) two-way centrifuge test on carbonate silt at 4.5d embedment depth: single episode of packets of increasing and decreasing amplitude, and b) two-way p-y apparatus test on carbonate silt: single episode of packets of increasing and decreasing amplitude.
Figure 4-26. One-way centrifuge test on carbonate silt at 4.5d embedment depth: four episode of packets of increasing and decreasing amplitude, with dissipation periods.

For the kaolin clay, the PICS model shows good agreement with the experimental p-y test data from test KC4_p_y from Figure 4-27a, in terms of the changing normalised secant stiffness with cycles. The exception is for cycles 500 to 700, which are at very small displacements and the model does not seem to capture the return to a higher stiffness post-peak. The opposite is observed in Figure 4-27b, where results of a multi-amplitude two-way test (KC3) at 4.5d embedment are presented alongside the PICS prediction. The secant stiffness from the PICS model shows good agreement with the experimental data prior to the peak cyclic load, while the post peak behaviour is not accurately modelled. Nevertheless, the increase in stiffness due to cycling and reconsolidation prior to the peak in each cyclic event are observed in both the experimental data and the PICS model. Although some aspects of the behaviour are not properly captured in these comparisons, it is evident that the PICS model captures the general observed trends of changing stiffness.
due to the competing processes of damage and reconsolidation, which is not possible using other p-y spring models.

\[\text{Figure 4-27. Results of a) two-way p-y apparatus test on kaolin clay: single episode of packets of increasing and decreasing amplitude, and b) two-way centrifuge test on kaolin clay at 4.5d embedment depth: four episode of packets of increasing and decreasing amplitude, with dissipation periods.}\]

\[\text{4.2.5. Independent validation}\]

To validate the calibrated parameters for carbonate silt, results from a separate test performed in the centrifuge using a flexible pile were modelled using PICS. Details of the experimental setup are given in Guevara et al. (2022), with a summary of the prototype pile
dimensions given in Table 4-7. The pile was instrumented with strain gauges along its length to measure changes in bending moments resulting from cyclic displacements imposed at the pile head. The imposed displacement sequence shown in Figure 4-28.

**Table 4-7. Test setup dimensions**

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Prototype at 80 g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outer Diameter (Aluminium pile)</td>
<td>0.96 m</td>
</tr>
<tr>
<td>Wall thickness (Aluminium pile)</td>
<td>0.036 m</td>
</tr>
<tr>
<td>Wall coating (Aluminium pile + Epoxy coating)</td>
<td>0.0768 m</td>
</tr>
<tr>
<td>Total Outer Diameter</td>
<td>1.114 m</td>
</tr>
<tr>
<td>Embedment</td>
<td>18.24 m</td>
</tr>
<tr>
<td>Height of imposed displacement above mudline</td>
<td>3.36 m</td>
</tr>
</tbody>
</table>

![Graph showing displacements](image)

**Figure 4-28.** Measured displacements at the pile head.

The pile was modelled using the software LAP (Doherty 2017). The pile-soil interface was assumed to be partially rough ($\alpha = 0.5$), with pile stiffness set equal to the measured value of 889 MN.m² for the model. In the model, the soil strength gradient with depth was set at $k_{su} = 1.66$ kPa/m – determined by slow rate CK₀U simple shear tests, and consistent with T-bar tests performed in the sample. Measured sensitivity (from T-bar testing) was $St = 5$ in the centrifuge samples, with this value used in the PICS model. The ultimate lateral soil pressure with depth ($N_p \cdot s_u$) was determined using Jeanjean et al. (2017) assuming a no-gapping condition. The monotonic $p$-$y$ curves were determined from simple shear tests.
using the scaling method proposed by Zhang and Andersen (2017), with scaling factors $\zeta_1 = 2.8$ and $\zeta_2 = 2.0$. The chosen value $\zeta_2 = 2.0$ is larger than the value recommended by Zhang and Andersen (2017), but this was selected in order to reduce the stiffness of the p-y curves at small strains, resulting in bending moment profiles that better match the first cycle of the test.

The monotonic backbone curve at each pile node was then transformed into a parallel-Iwan spring system. The cyclic pile head displacement sequence shown in Figure 4-28 was then imposed at the pile head, with PICSI used to model the response of the soil at each time-step in the analysis. The results, in terms of head load throughout the test are shown in Figure 4-29, and in terms of bending moment profiles for the first and last cycle of each packet of different amplitude of the test in Figure 4-30. For comparison, the bending moment that would have been predicted using the monotonic curves that were input to PICSI is shown for each cyclic packet as the continuous grey line in Figure 4-30. The results demonstrate that the calibrated PICSI model is able to capture changes in bending moment profile, including the load history effects on the stiffness. Key observations are as follows:

- The bending moment profile is relatively unchanged from cycle ($N = 1$ to 400), which is attributed to the cyclic amplitude being sufficiently small to avoid ‘damage’ to soil stiffness. However, once the amplitude increased ($N = 401$ to 600) there is a clear downward shift in the bending moment profile reflecting the loss of soil support close to the surface, which is well modelled by PICSI.

- The impact of the first large amplitude packet can be observed in the subsequent small amplitude sequence ($N = 601$ to 700), as evident through a lower (and deeper) peak bending moment when compared to the previous low amplitude packet ($N = 1$ to 400). As noted earlier, at shallow depths PICSI is expected to overestimate the recovery of stiffness after larger amplitude cycling. The fact that the PICSI model matches the experimental data suggests that the level of non-recoverable softening in the soil wedge zone (where soil strength is lowest), as created by higher amplitude cycling between $N = 401$ and 600, was insufficient to impact the bending moment profile.

- The shallow bending moment profile for $N = 701$ – the first cycle in the largest amplitude packet – is slightly underpredicted by PICSI soil springs, perhaps...
because the conductor is moving into fresh (undamaged) soil, which would result in an increase in resistance not captured by PICS. However, the PICS prediction accurately captures the peak BM, and is better than the estimate using the monotonic unsofterned profile (also shown). By $N = 800$ the experimental data and model prediction are in closer agreement.

- For $N = 801$ to $900$ the bending moment profile predicted by the PICS model is noticeably stiffer than suggested by the experimental data. This is consistent with PICS overpredicting the recovery of stiffness (after high amplitude cycling) in the upper soil.

![Figure 4-29. Measured and modelled head load vs time.](image-url)
**Figure 4.30.** Measured and modelled bending moment vs depth below mudline.

### 4.2.6. Discussion

Fig. 4.24 to Figure 4.27 show that the model accurately captures the secant stiffness degradation and regain due to consolidation and hardening (except in post peak cycling and one-way loading conditions which will be discussed below), but does not accurately model the shape of the load-displacement loops. The area inside the loops (or damping) and how it changes with cycling could be significant for fatigue life estimation of conductors. Planned future work includes developing an approach to modify the Parallel-Iwan spring and slide capacities to better capture the shape of the degraded p-y curve.

It is observed that the post-peak secant stiffness cyclic degradation in the tests that had symmetric increasing and decreasing amplitude cycling (Figure 4.25a, Figure 4.26 and Figure 4.27b) is not accurately modelled by PICS. The post-peak behaviour shown by
PICSI exhibits some recovery, consistent with that seen in the p-y apparatus test results shown in **Figure 4-25b** and **Figure 4-27a**. It is postulated that the upper soil surrounding the pile, where a wedge mechanism forms (like the rigid pile tests), degrades differently to the lower soil that flows around the pile (like the p-y apparatus tests). The reasons for this could be the lower confinement pressure near the soil surface, which allows the deformation with less constraint, or water entrainment during large amplitude cycling. A difference in degradation between wedge/flow failure mechanisms could explain why PICSI does not accurately predict the bending moment profile in the final cyclic packet from the flexible pile test (N=801-900). In practice, this could be captured by using different PICSI model parameters for the deep failure mechanism (flow-around) and the shallow wedge, to reflect the different rates of softening and dissipation. Further refinement in the calibration could explore the possibility of obtaining two sets of parameters: one for the shallow wedge mechanism and another for the deep flow-around mechanism.

The current formulation of PICSI does not capture scenarios where the pile translates into non-remoulded material, which is reflected in the one-way test shown in **Figure 4-26**, and in the first cycle of the large amplitude packet (N=701) from the flexible pile test (**Figure 4-30**). In the latter, the soil has experienced some damage due to the second cyclic amplitude packet (N=401-600), before incursion into non-remoulded material in the first cycle of the fourth packet (N=701 - 800). The bending moment profile of cycle 701 modelled by PICSI shows some degradation that is not observed in the experimental data. Nevertheless, by the end of the packet (N=800), after the soil has remoulded, the PICSI model shows better agreement with the experimental data.

The advantage of PICSI is that it is able to model the changes in bending moments along the pile with different amplitude cycling. This is relevant for fatigue analysis of conductors, as the degradation of stiffness due to cycling will progressively shift the bending moments down the pile, inducing stresses in the lower components. Conversely, consolidation and hardening will shift the bending moments upwards, reducing the stresses in the lower components and increasing stresses in the upper sections of the conductor. The use of constant p-y curves throughout fatigue analysis are unable to model these shifts in stresses and if too stiff, may overestimate stress in the upper components of the conductor and underestimate them in the lower components (and vice versa if too soft).
4.2.7. Conclusions

This paper documents a calibration procedure for the Parallel-Iwan Critical State Inspired (PICS) $p-y$ model using data from rigid conductor physical model tests in the centrifuge and $p-y$ apparatus. The method utilised an optimisation routine to determine the model parameters, rather than user judgement. Calibration of pore pressure dissipation (hardening) parameters were separated from calibration of the damage parameters.

As part of the calibration process, a simplification to the PICS model is introduced that reduces the number of parameters to calibrate, that is achieved by assuming a linear envelope for the fully remoulded strength line ($D = 1$). This resulted in estimates that agree well with the experimental data.

The calibrated parameters were first compared to the complete set of rigid conductor tests, before being independently validated against experimental data using a flexible pile test. The later used a model based on PICS generated springs distributed along the pile, which provided a good match to the cyclically induced changes in bending moment in the experiment. Future applications for fatigue assessment of conductors could implement either a decoupled model, such as the one presented in this paper, or introduce coded subroutines that support a coupled finite element model via updated springs at every timestep.

The rigid pile centrifuge tests used for the calibration represent mostly a shallow wedge failure mechanism, whereas the $p-y$ apparatus tests represent a deep flow around failure mechanism. The combination of both for the calibration process provided a good match for the flexible pile data. Further refinement of the calibration could explore the possibility of obtaining two sets of parameters: one for the shallow wedge mechanism and another for the deep flow-around mechanism. Nevertheless, centrifuge tests are thought to be the preferred method to calibrate the model, since the $p-y$ apparatus test stiffness values could be influenced by boundary effects, although more research is needed to confirm this. Future work could also explore applying the method proposed in this paper to calibrate the parameters for the PICS model from multiple episodic simple shear tests, either directly or by scaling the stress-strain response to $p-y$ curves.
Acknowledgements

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CHAPTER 5 - AN EVALUATION OF SOIL-CONDUCTOR P-Y CURVES FOR FATIGUE ANALYSIS OF OFFSHORE DRILLING SYSTEMS

Prologue

Chapter 5 comprises a paper prepared for submission to Canadian Geotechnical Journal titled: “An evaluation of soil-conductor p-y curves for fatigue analysis of offshore drilling systems”.

The chapter reviews three different methods for deriving p-y curves for use in fatigue analysis of drilling conductors:

- The forthcoming (draft) ISO 19901-4 recommendations.
- The PICS framework with the calibrated parameters from Chapter 4.
- An alternative simplified method for deriving fully remoulded (steady-state) p-y curves presented in this chapter.

The bending moments obtained from a pile-soil finite element analysis using the three methods are compared to experimental centrifuge data from a flexible pile test. The results presented in this chapter show how the bending moment profile is affected by the adopted p-y curves. Potential impact on the fatigue life assessment of drilling conductor systems is discussed and recommendations for design provided.
5.1. An evaluation of soil-conductor p-y curves for fatigue analysis of offshore drilling systems

Abstract

The paper reviews three different methods for deriving p-y curves for fatigue analysis of drilling conductors, providing an assessment of the forthcoming (draft) ISO 19901-4 recommendations (expected to be published in 2023). An alternative simplified method for deriving fully remoulded (steady-state) p-y curves is proposed, based on centrifuge tests and laboratory element measures of small strain stiffness, linking maximum shear modulus to soil strength ratio and sensitivity. It is shown that this new approach provides a more accurate prediction of the previously-published data on which the ISO guidelines are based. In addition, a third p-y method is examined – the PICSI framework (White et al. 2022)– which can model changes in the stiffness and strength of p-y curves associated with cyclic loading and pore-pressure dissipation. The bending moments obtained from a pile-soil finite element analysis using the three methods are compared to data from flexible piles tested in a centrifuge. This test demonstrated the load history effect in which the bending moment profile for a given cyclic amplitude is altered by prior packets of larger cycles. The resulting evolution of the bending moment profile showed migration of the fatigue hotspot down the pile, which is beneficial for the fatigue life. The forthcoming ISO fatigue p-y curves over-predicted the observed stiffness. The new alternative simplified p-y method generally predicted the bending moments to ±25% accuracy, while the PICSI framework gave the most accurate bending moment predictions across the full test sequence. These methods are recommended for design use in preference to the new ISO fatigue approach.

5.1.1. Introduction

Estimating the fatigue life of a drilling conductor system is a critical design consideration among the various checks that must be performed to ensure safe operations. In addition to providing axial support to the components of the system and preventing the walls of the hole from collapsing, conductors transfer the continuous lateral displacements from the upper parts of the system to the soil. Due to the system being interconnected through the riser and tensioned from the vessel (Figure 5-1), the environmental displacements experienced by the vessel on top of the system are transferred down to the conductor as cyclic displacements.
Repeated stresses produced by the cyclic movement of the conductor can produce fatigue failure at certain locations. In design, stresses are tracked at the locations considered most at risk of fatigue failure – these are called hotspots and include locations where there are changes in geometry, discontinuities, connectors or welds.

Conductors are typically pipe piles with diameters \( d \) between 30” and 36”, wall thickness around 1” and length \( L \) to diameter \( L/d \) ratios > 20. Due to the large \( L/d \) ratios, conductors can be accurately represented as Euler-Bernoulli beams, and the soil-conductor lateral load-displacement relationship modelled as Winkler springs. The most widely used type of Winkler spring for lateral behaviour of conductors is the \( p-y \) curve, which is a non-linear relationship between soil pressure \( P \) and lateral displacement \( y \). The secant stiffness \( k \) of a \( p-y \) curve at any given point is defined as:

\[
k = \frac{P}{y} \tag{5-1}
\]

Current ISO 19901-4 guidelines (ISO 2016) recommend \( p-y \) curves based on monotonic loading that are known to underestimate stiffness (Jeanjean 2009), particularly at small displacements. For drilling operations, displacements at the conductor head are typically below 10% of the diameter – and hence an accurate determination of stiffness at small displacements is important for reliable fatigue life estimation.
The forthcoming edition of ISO 19901-4 (ISO/DIS, 2021 – currently under review) proposes a new framework that covers both monotonic and fatigue lateral loading and aims to overcome limitations in the previous guidelines. The ISO framework proposes different p-y curves for three different applications: monotonic, cyclic and fatigue:

- The recommended p-y curves for monotonic loading in clay (originally from Jeanjean et al. 2017) can be derived by either scaling from simple shear laboratory tests or by using default normalised curves.
- The p-y curves for fatigue analysis of conductors are intended to represent “steady state” conditions, which assume that there is a unique unload-reload stiffness for a given displacement, reached after a few hundred cycles, and for which the stiffness and damping will stabilise. The method disregards the transient phase before reaching this state, as well as the impact of any load history previously experienced, and any potential regain in stiffness due to pore pressure dissipation (consolidation). The method is based on Zakeri et al. (2019), which relies on centrifuge tests performed in Gulf of Mexico and kaolin clay (Zakeri et al. 2016), as well as p-y apparatus tests (Zakeri et al. 2017) on a range of marine clays.

Recent studies (Doherty et al. 2019; Guevara et al. 2020, 2021; Zhang et al. 2011) have demonstrated the impact of periodic cyclic loading and pore pressure dissipation on soil-pile stiffness of short rigid piles in reconstituted carbonate silt and kaolin clay. Such changes in stiffness affect stresses in a conductor by progressively shifting the location of peak bending moments down and up the pile as the soil remoulds (softens) and then recovers.

White et al. (2022) propose a framework to model the effect of load history on p-y curves, by simultaneously accounting for softening due to cyclic remoulding and recovery due to dissipation of pore pressures. The framework is inspired by critical state soil mechanics (Wood 1990), and incorporates additional features observed in experiments. It was developed for soils that tend to densify with shearing, such as clays or silts with an initial state on the ‘wet’ side of the critical state line.

For fatigue analysis the bending moment at hot-spots is critical in design, so this paper aims to assess the performance of the forthcoming ISO guidelines when used to compute the bending moments from centrifuge tests on a flexible pile. In addition, two other methods
are also used to compute the bending moments. In summary, three approaches for defining \( p-y \) curves for fatigue analysis in soft soils will be discussed:

- The forthcoming ISO/DIS (2021) method for fatigue \( p-y \) curves;
- A new simplified fully remoulded “steady-state” method proposed in this paper, based on centrifuge data and laboratory element tests; and
- The PICSI method (White et al. 2022).

The impact of the choice of \( p-y \) curve selected on bending moment profile will be discussed and recommendations for fatigue design of conductors in soft soils provided.

### 5.1.2. \( p-y \) curves recommended in the forthcoming ISO guidelines

#### 5.1.2.1. Lateral ultimate soil resistance

The overall method proposed to analyse piles subject to lateral loading in the forthcoming ISO guideline harmonises various methods for lateral capacity (Murff and Hamilton 1993; Randolph and Houlsby 1984; Yu et al. 2015; Zhang et al. 2016). The method accounts for roughness of the pile, gap formation behind the pile, and the combination of deep flow-around and shallow wedge failure mechanisms. The ultimate lateral soil reaction \( (P_u = N_p s_u) \) of an embedded pile with depth is used as an input for developing the monotonic, fatigue and cyclic \( p-y \) curves. It is important to note that the proposed method was calibrated with stress-strain behaviour measured in direct simple shear tests at a strain rate of 5%/hour, and therefore the guidelines recommend that undrained shear strength \( (s_u) \) determined from DSS testing is used.

#### 5.1.2.2. Monotonic \( p-y \) curves

Two approaches are proposed for determining the shape of the \( p-y \) curves: 1) default normalised \( p/p_u \) vs \( y/d \) curves that depend on plasticity index and OCR, and 2) a scaling procedure from simple shear tests. The scaling procedure is a modification of the approach proposed by Zhang and Andersen (2017), based on similarities in the stress-strain response of soil with the \( p-y \) curves as previously demonstrated by Bransby (1999). While these \( p-y \) curves are proposed for monotonic conditions where the piles are expected to experience large displacement, their suitability for conductor fatigue life estimation will be discussed in this paper.
5.1.2.3. Fatigue p-y curves

The method for constructing p-y curves for fatigue analysis in the forthcoming ISO guideline is based on experiments performed in kaolin clay and Gulf of Mexico clay, and validated with data from p-y apparatus tests on a range of marine clays. The test data was used to define a normalised “steady-state” secant stiffness \( K \), which is defined as the peak-to-peak secant stiffness within a loop \( k_{p-p} \) normalised by the lateral soil reaction pressure \( P_u \):

\[
K = \frac{k_{p-p}}{P_u} \quad [5-2]
\]

and where

\[
k_{p-p} = \frac{\Delta P}{\Delta y/d} \quad [5-3]
\]

with \( \Delta P \) being the range of lateral soil reaction and \( \Delta y \) being the lateral peak-to-peak movement amplitude.

The ISO/DIS guideline proposes two options for p-y curves used in fatigue analysis: a spring-only method and a spring-dashpot method. In this paper we will discuss the spring-only method, which consists of a unique normalised soil resistance for fatigue actions \( p_{fa} / p_u \) against \( y/d \) curve, represented by:

\[
p_{fa} = p_u A_s \left( \frac{y}{d} \right)^{-B_s} \quad [5-4]
\]

and where \( A_s = 0.45 \) if \( s_u < 40 \) kPa (and \( A_s = 0.19 \) for other conditions) and \( B_s = 0.05 \) for all conditions are the recommended parameters for soft to very hard clay.

The form of Equation [5-4] is shown in Figure 5-2 and does not pass through the \((y = 0, p = 0)\) origin. This is addressed by specifying a point at which the curve given by Equation [5-4] is joined to the origin, as illustrated by the dotted line in Figure 5-2.
When implemented in a soil-pile system, all $p$-$y$ springs have to “climb” through this arbitrarily defined portion of the $p$-$y$ curve to find convergence for the imposed loads at each timestep. In most practical well conductor simulations, the mobilised pressure will remain close to the peak, and therefore the response will be defined largely by the discretisation of the $p$-$y$ curve rather than the data points used to establish the curve. It is likely that this portion of the curve may be too stiff, which can be demonstrated through the following approximate calculation (and noting the gradient of the initial portion of the curve, i.e. $(P/P_u)/(y/d) = 640$). Adopting $P_u = 12s_u$, the equation for the initial portion of the curve can be expressed as

$$P = 12s_u 640 \left( \frac{y}{d} \right) \quad [5-5]$$

Approximate elastic solutions for lateral pile displacement (e.g. Baguelin et al. 1977) suggest that:

$$P = 4G \left( \frac{y}{d} \right) \quad [5-6]$$

where $G$ is the shear modulus of the soil. Substituting Eq [5-5] into [5-6] leads to a soil rigidity index of $G/s_u = 1900$, which is thought to be unreasonably high for most soils particularly given this portion of the curve operates up to 64% of the unsoftened capacity.

---

**Figure 5-2.** Normalised fatigue $p$-$y$ curve.
5.1.3. Alternative simplified method

The alternative simplified method provides steady-state p-y stiffness values tied to laboratory-based measurements of limiting small-strain soil stiffness $G_{\text{max}}$, with the stiffness decaying with amplitude according to a conic form of p-y curve calibrated to model tests. Recent studies (Guevara et al. 2020, 2022b) have identified similarities between the variation of $K$ with $y/d$ and the variation of the shear modulus ($G$) with shear strain ($\gamma$). Figure 5-3 shows data from Guevara et al. (2022b), from centrifuge tests on carbonate silt and kaolin clay using a short rigid hollow pile, and tests performed using the NGI operated p-y apparatus (Zakeri et al. 2017).

![Figure 5-3](image)

**Figure 5-3.** Normalised fully softened secant stiffness from p-y apparatus and centrifuge tests, from (Guevara et al. 2022b).

The normalised peak secant stiffness at small displacements ($K_{\text{max}}$) could be related to the maximum shear stiffness ($G_{\text{max}}$) by combining Equations [5-2], [5-3] and [5-6]:

$$K_{\text{max}} = \frac{4G_{\text{max}}}{P_u}$$  \[5-7\]

For the carbonate silt, bender element test data shown in Figure 5-4, performed on samples that are representative of the centrifuge test sample, suggest a linear trend for $G_{\text{max}}$ with mean effective stress. From this figure, a value of $G_{\text{max}} = 4.72$ MPa can be interpolated for the mean stress level at 1.8 m depth, which when combined with a $s_n = 2.72$ kPa (measured
in SS at slow rate) and \( N_p = 14 \) (increased for rate effects in centrifuge tests) gives a value of \( K_{\text{max}} = 495 \). This is consistent with the data in Figure 5-3, allowing for uncertainty in Eq. [5-6] from factors such as relative soil pile rigidity, length to width ratio of the pile, and head fixing and loading conditions (Baguelin et al. 1977). Based on that study, values of \( K_{\text{max}} \) between 370 and 620 are realistic, which is consistent with the data from Guevara et al. (2022b) in Figure 5-3, showing that soil element test stiffness data from bender elements can be linked with limiting p-y stiffnesses from model tests.

![Figure 5-4](image)

**Figure 5-4.** Measured \( G_{\text{max}} \) from bender element tests on reconstituted carbonate silt.

It is possible to fit a curve to the tests data from Figure 5-3, using a modified version of the four-parameter conic equation proposed by Burd et al. (2019), and then calculate the resulting normalised secant stiffness:

\[
\frac{P_F}{P_U} = \frac{P_{\text{rem}}}{P_{\text{u}}} \frac{2c}{-b + \sqrt{b^2 - 4ac}} \quad ; \quad \frac{y}{d} \leq \frac{y_{\text{rem}}}{d} \tag{5-8}
\]

where:

\[
a = 1 - 2n \tag{5-9}
\]

\[
b = 2n \frac{y/d}{y_{\text{rem}}/d} - (1 - n) \left[ 1 + \frac{K_{\text{max}} y/d}{P_{\text{rem}}/P_{\text{u}}} \right] \tag{5-10}
\]

\[
c = \frac{K_{\text{max}} y/d}{y_{\text{rem}}/d} (1 - n) - n \left( \frac{y/d}{y_{\text{rem}}/d} \right)^2 \tag{5-11}
\]

and where:
\( P_f \) is the lateral soil pressure for fatigue actions

\( \frac{P_{rem}}{P_u} \) is the normalised fully remoulded soil pressure, which would be equal to \( \frac{1}{S_t} \)

\( \frac{y_{rem}}{d} \) is the normalised displacement at which \( \frac{P_{rem}}{P_u} \) is reached

\( n \) is a parameter that determines the shape of the curve

When the displacements exceed \( y_{rem} (y/d > \frac{y_{rem}}{d}) \) then:

\[
P_f = P_{rem}
\]  \hspace{1cm} \text{[5-12]}

Considering a sensitivity \( S_t = 3.3 \) for the carbonate silt (Guevara et al. 2022b) and a maximum normalised secant stiffness \( K_{max} = 495 \) as calculated above, the parameters \( y_{rem} \) and \( n \) can be adjusted to fit the data in Figure 5-3. The fitted \( p-y \) curve can be observed in \( p-y \) space and \( K-y \) space in Figure 5-5a and Figure 5-5b, respectively, with values of \( n = 0.4 \) and \( \frac{y_{rem}}{d} = 0.05 \) yielding an acceptable fit to the experimental data. For comparison, the fatigue \( p-y \) curve recommended by the forthcoming ISO 19901-4 guideline is also plotted in both figures.
Figure 5-5. Normalised fully softened secant stiffness from p-y apparatus and centrifuge tests in a range of clays. Data from Guevara et al. (2022b) and Zakeri et al. (2019).

Published normalised secant stiffness data from Zakeri et al. (2019) and Guevara et al. (2022b) are shown in Figure 5-5, along with the conic equation curve proposed above and the fatigue p-y curve recommended in the forthcoming ISO 19901-4 guidelines. The data from both sources broadly agree, even though the data from Zakeri et al. (2019) was normalised by $12 s_u$ instead of an apparatus specific measurement of $P_u$. As was shown in Guevara et al. (2022c), higher values of $P_u$ are possible in p-y apparatus testing, which if used would bring the data further into alignment.

Figure 5-5 demonstrates how the perception of data correlation with a fitted curve changes depending on whether a logarithmic axis is applied. At small displacements the fatigue p-y curve recommended in the forthcoming ISO 19901-4 draft overpredicts the normalised secant stiffness, in some cases by a factor $>2$ (Figure 5-5a). However, this discrepancy is
difficult to distinguish when plotting the data on joint logarithmic axes (Figure 5-5d), and even harder to note when only the secant stiffness axis is logarithmic (Figure 5-5c).

5.1.4. The PICSI method
The Parallel-Iwan Critical State Inspired framework, PICSI (White et al. 2022), was developed to model changes in a p-γ spring due to cyclically induced remoulding and pore pressure dissipation. The method consists of a Parallel-Iwan system of springs and sliders to model the non-linear hysteretic behaviour of the p-γ curve, with the strength and stiffness affected by the load history imposed. The approach is applicable for soils that lie on the ‘wet’ side of the critical state line and tend to densify with shearing and consolidation. Unlike pore pressure accumulation procedures, the system is affected at each timestep by the last imposed increment of displacement and time interval. The process of damage, reconsolidation and densification (or hardening) occur in parallel within the framework.

The procedure involves first transforming a p-γ curve into a Parallel-Iwan (PI) system, applying the first increment of displacement for a given interval of time (δy, δt), and calculating the new strength and stiffness of the PI system based on the soil parameters introduced in the model. This process is repeated for the entire time-displacement history, with the full method described by White et al. (2022).

5.1.5. Experimental data and modelling
Results from an independent flexible pile centrifuge test on soft soil were modelled using the methods discussed in the previous section, in order to discuss their impact on the predicted bending moment down the conductor, and the accuracy of these predictions. The soil used was a reconstituted carbonate silt of low plasticity (PI =22%), normally consolidated in-flight at an acceleration of 80g. The prototype external diameter, embedment depth and height of imposed displacement above the mudline were 1.114 m, 18.24 m and 3.36 m, respectively. Tube samples were prepared with the same slurry used for the centrifuge test, and consolidated in tubes under a vertical pressure of 30 kPa, which represents an embedment depth of around 6.0 m. From CK0U simple shear tests, the measured soil strength gradient with depth was $k_{su} = 1.66$ kPa/m and the sensitivity (measured with an inflight T-bar) $S_t = 5$. The experimental setup and the LAP (Doherty 2017) model are shown in Figure 5-6 and more details of the soil and setup can be found in Guevara et al. (2022a).
Bending moments at each strain gauge location were obtained at a logging frequency of 16 Hz. The displacement imposed at the hinge level for the test discussed in this section followed the sequence shown in Figure 5-7, and was measured with a pair of lasers pointing at a bracket connected to the pile. The sequence consisted of five packets of different amplitude triangular cyclic loading without consolidation periods in between. The sequence was designed to demonstrate load-history effects on the bending moment profile by revealing differences in the response during the first, third and fifth packets of cycles, which all had the same displacement amplitude. The time shown in Figure 5-7 is scaled for consolidation effects, that is, the model time in the centrifuge was multiplied by \( n_g^2 \) (Garnier et al. 2007), with \( n_g \) being the radial acceleration of the centrifuge \( (n_g = 80 \text{ for this test})\).
The pile was modelled using the software LAP (Doherty 2017) and the $p$-$y$ methods presented in the previous section. The same ultimate lateral soil reaction was determined for all cases, following the method recommended by the forthcoming ISO 19901-4. A no-gapping condition was assumed, with undrained strength with depth calculated using the gradient ($k_{an}$) indicated above. The ultimate lateral soil reaction and the $p$-$y$ curves for each method were determined at 1 m intervals, down to 20 m below the mudline. The pile flexural stiffness, based on the scaled dimensions, was set as 889 MN.m$^2$ and the pile-soil interface was assumed to be partially rough (Guevara et al. 2022a). The sequence executed in the tests (as recorded from the laser sensors) was imposed at the hinge (top of pile in LAP model) and the response of the soil was modelled at each timestep of the analysis.

For the PICSi method, an initial set of monotonic $p$-$y$ curves were determined by scaling from simple shear tests as per procedures defined in Zhang and Andersen (2017). The scaling factors $\xi_1=2.8$ and $\xi_2=2.0$ were used to convert the strain from the simple shear tests to normalised displacement ($y/d$) for the $p$-$y$ curves. The factor $\xi_2=2.0$ is slightly larger than the value of 1.6 recommended by Zhang and Andersen (2017) and was selected as a calibration parameter to reduce the stiffness of the $p$-$y$ curves at small strains, resulting in bending moment profiles that better match the first cycle of the test, as shown in Figure 5-8.

![Figure 5-8. Bending moment monotonic push.](image)
The monotonic $p$-$y$ curve was used as the starting point for the PICSI model, and transformed into a Parallel-Iwan system of springs and sliders at each pile node. The modification proposed by Guevara et al. (2022c) for the fully remoulded envelope of the model (analogous to the critical state line) was used. The input parameters for PICSI are shown in Table 5-1, and come from the calibration process described by Guevara et al. (2022c). The results obtained with this method are labelled in the plots in the following section as PICSI. It is worth mentioning that the test duration was not enough to dissipate most of the pore pressures generated by the cyclic sequence imposed, hence, only the damage modelling capabilities of PICSI can be observed when modelling this test.

**Table 5-1.** PICSI model parameters for carbonate silt (from Guevara et al. 2022c).

<table>
<thead>
<tr>
<th>Model feature</th>
<th>Description</th>
<th>Parameter</th>
<th>Carbonate silt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength limits</td>
<td>Slope of Damage = 0 line</td>
<td>$\lambda^*$</td>
<td>8.69</td>
</tr>
<tr>
<td>Damage generation</td>
<td>Rate constant</td>
<td>$d_r$</td>
<td>2.39</td>
</tr>
<tr>
<td></td>
<td>Power constant</td>
<td>$d_p$</td>
<td>1.68</td>
</tr>
<tr>
<td></td>
<td>Effect of amplitude</td>
<td>$d_a$</td>
<td>1.00</td>
</tr>
<tr>
<td>Pore pressure dissipation</td>
<td>Rate constant</td>
<td>$c_r$</td>
<td>0.30</td>
</tr>
<tr>
<td></td>
<td>Power constant</td>
<td>$c_p$</td>
<td>9.65</td>
</tr>
<tr>
<td>Hardening</td>
<td>Slope of hardening path</td>
<td>$\kappa^*$</td>
<td>0.31</td>
</tr>
<tr>
<td></td>
<td>Variation of hardening slope</td>
<td>$h_p$</td>
<td>0.002</td>
</tr>
</tbody>
</table>

The forthcoming ISO 19901-4 $p$-$y$ curves for fatigue were derived using Eq. [5-4] and the calculated ultimate lateral soil reaction. The parameters $A_s$ and $B_s$ were assigned values of 0.45 and 0.05, respectively, as suggested in Zakeri et al. (2019) for clays with $s_u < 40$ kPa. Pairs of soil pressure and normalised displacement were generated at 1 m intervals. Since the undrained shear strength of the soil profile linearly increases with depth, and the software interpolates between adjacent $p$-$y$ curves for intermediate nodes, a 1-metre spacing was deemed sufficient. The results obtained with this method are labelled in the plots as ISO fatigue in the following section.

The $p$-$y$ curves from the alternative method proposed in this paper were generated using Equations [5-8] to [5-11], with a maximum normalised secant stiffness $K_{max} = 495$, as
determined from Equation [5-7]. Parameters controlling the shape of the curve were set as \( n = 0.4 \) and \( y_{rem}/d = 0.05 \), as determined from the fit to the experimental data from rigid pile centrifuge testing. The measured sensitivity of the carbonate silt used in centrifuge tests with the rigid pile setup \( (S_t = 3.3) \) is slightly different to that observed in the flexible pile tests \( (S_t = 5) \), while the measured soil strength gradient with depth \( k_{su} \approx 1.65 \text{kPa/m} \) was the same for both samples. For consistency with the measured soil parameters of the flexible pile test, a sensitivity \( S_t = 5 \) was selected to determine the normalised remoulded pressure \( P_{rem}/P_u \) for equations [5-8] and [5-10]. The results obtained with this method are labelled in the plots in the following section as alternative method.

### 5.1.6. Comparison of the applied methods

The resulting ISO 19901-4 \( p-y \) curve for fatigue and the \( p-y \) curve generated using the alternative method are shown in Figure 5-9 normalised by the ultimate soil reaction and diameter. For comparison purposes Figure 5-9 also shows the default normalised monotonic \( p-y \) curve proposed in the draft ISO 19901-4 for a soil of \( OCR < 2 \) and plasticity index \( PI < 30 \).
Figure 5-9. ISO draft fatigue p-y curve and proposed simplified p-y curve.

Figure 5-9 shows that the p-y curve for fatigue derived from the forthcoming ISO 19901-4 is the stiffest up to a displacement of 0.001d, with an initial stiffness up to three times the monotonic value at small displacement. It seems unlikely that this can be justified based on damping effects, as these effects are directly proportional to the strain level and are thus small for small displacements.

The p-y curve from the alternative method proposed in this paper is similar to the monotonic p-y curve in the forthcoming ISO 19901-4 for displacements up to ≈0.001d, beyond which the curve plateaus at a $P/P_u$ value = $1/S_t$. Since the draft ISO monotonic p-y curve discretisation jumps from $P/P_u = 5\%$ ($y/d=0.0001$) to $P/P_u = 20\%$ ($y/d=0.001$), it is not
possible to see if the curves would match for the range of displacements in between. Hence, a curve has been fitted to the draft ISO monotonic p-y curve between these displacements to infer if it would match some of the points of the alternative method proposed. The fitted curve is shown as the dotted line in Figure 5-9, and it can be observed that it reasonably matches both the ISO monotonic p-y curve and the proposed alternative method. This is encouraging since the p-y curve from the alternative method was derived independently from the forthcoming ISO 19901-4 recommendations, and is based on the maximum shear modulus, soil strength and sensitivity – but is reasonable since significant damage is not expected at small displacements, where the soil behaves almost elastically.

The bending moment profiles at the peak positive displacement for the cycles at the start and end of each packet of the sequence from Figure 5-7, for all studied cases and experimental data are shown in Figure 5-10 (N = 1 - 700) and Figure 5-11 (N = 701 - 900).

At N = 1 the bending moment profile generated with the forthcoming ISO 19901-4 fatigue p-y curve appears overly stiff, resulting in higher pile head lateral force, and therefore an overprediction of bending moments in the first few metres above and below the mudline, and underpredicts the bending moments below a normalised elevation z/d = -5. This trend is also observed for all other cycles in both Figure 5-10 and Figure 5-11, even at the start of larger amplitude packets where the soil has not reached “steady-state” (and hence should be stiffer than predicted by the fatigue curves).

The alternative method proposed in this paper underpredicts the bending moments at the start of each larger amplitude packet (N=1, 401, 701), which is consistent with the method being developed for a fully remoulded state. The method predicts the bending moments for cycles 400, 600, and 800 better, which represent “fully remoulded” cycles for packets 1, 2 and 4. For N=601, 700, 801 and 900 – which are influenced by previous larger amplitude cycles – the method also accurately predicts the bending moment profile. The PICSI model accurately tracks the bending moment profiles from cycles 1 to 800, but appears to overpredict the magnitude of the bending moments for cycles 801 and 900.
Figure 5-10. Bending moment profile for N=1 to 700.
Figure 5-11. Bending moment profile for N=701 to 900.

The accuracy of the bending moment prediction from each method is summarised in Figure 5-12, discretised into shallow, peak and deep zones. The different zones were determined based on their position relative to the depth where the peak bending moment is observed, \((z/d)_{peak}\). Since this depth varies depending on the head displacement, it was nominally selected as the elevation of the strain gauge that reported the highest bending moment for the largest amplitude cycle (N=800), with a value \((z/d)_{peak} \approx -5.9\). The shallow zone groups observed bending moment for depths above 0.25\((z/d)_{peak}\), the peak zone groups bending moment for depths between 0.25\((z/d)_{peak}\) and 1.5\((z/d)_{peak}\), while the deep zone groups all bending moment observations below the peak zone.

For the fully remoulded “steady-state” approaches (ISO fatigue and alternative method proposed in this paper), only the last cycle of each amplitude packet is plotted, whereas for the PICS method the start and end cycles of each packet is plotted – with the hollow
markers indicating a bending moment measured at the end cycle of a packet, and filled markers indicating a bending moment measured at the start of a packet. As a visual aid, a range of ±25% is shown in Figure 5-12.

![Graphs showing bending moments comparison](image)

**Figure 5-12.** Calculated vs measured bending moments.
For the shallow zone, the calculated bending moments using the forthcoming ISO 19901-4 fatigue $p$-$y$ curves overpredict all measured bending moments by more than 25%. In contrast, calculations using the alternative method proposed in this paper consistently lies within the 25% band. However, there is a weak trend for the bending moment to be underpredicted as the magnitudes increase. The PICSI method leads to predictions that lie closest to the observations throughout.

For the peak bending moment zone, the ISO fatigue method consistently overpredicts the measured values, particularly for the higher bending moments, with some scatter seen at low bending moments. Both the PICSI and alternative methods accurately predict the bending moments in this zone.

For the deep zone, broadly opposite observations to the shallow zone can be made. Importantly, the bending moments calculated using the forthcoming ISO fatigue $p$-$y$ curve underpredict the measured bending moments in the deep zone – for modest bending moments (which are expected to occur most of the time during drilling operations) the underprediction is by a factor of up to four.

5.1.7. Changing bending moment profile due to load history

As previously mentioned, when assessing the fatigue life of the wellhead-conductor system, the different stresses at the hotspots during different levels of riser movement are assessed, and the accumulated damage (and fatigue life) on each of these hotspots is calculated by summing across the different operating conditions. The lowest fatigue life of any of the analysed components limits the design and determines the fatigue life of the system.

Any change of the shape of the bending moment profile (shifting upwards or downwards) under the same amplitude of riser movement but at different stages in the operating life, will affect the fatigue of the system. When the upper metres of soil get remoulded due to cycling, the bending moment profile shifts downwards as strength needs to be mobilised by soil at lower depths. This adds stresses on lower components of the system, and would not be accounted for if the same soil spring is assumed to apply throughout the fatigue analysis and the system operating life.

The impact of the load history on the bending moment profile, and hence on the stresses on the different hotspots in the system, is illustrated in Figure 5-13. In this figure the profile
with depth of the normalised bending moment range, defined as per Eq. [5-13], is plotted on a colour scale throughout all cycles in the sequence. The normalised bending moment range is defined as:

$$\frac{\Delta M}{\Delta M_{max}} = \frac{M_{\text{peak, } Nzi} - M_{\text{trough, } Nzi}}{\max (M_{\text{peak, } Nzi} - M_{\text{trough, } Nzi})_z}$$  \hspace{1cm} [5-13]

Where:

- $M_{\text{peak, } Nzi}$ is the bending moment at a specific depth $z_i$, and for the positive peak displacement at specific cycle $N$
- $M_{\text{trough, } Nzi}$ is the bending moment at a specific depth $z_i$, and for negative peak displacement at specific cycle $N$
- $\max (M_{\text{peak, } Nzi} - M_{\text{trough, } Nzi})_z$ is the maximum range of bending moment along the pile length for specific cycle $N$

This choice of normalisation allows the depth of the peak bending moment amplitude to be visualised, without the influence of the different cyclic amplitudes in each packet.
Also highlighted on Figure 5-13 is the depth of the peak bending moment range per cycle, shown as the black continuous line. If the load history did not affect the bending moment profile, the location of the peak bending moment range would not vary for cycles of the same amplitude, such as those from packets 1, 3 and 5. However, Figure 5-13 shows that the largest bending moment range at the end of the third packet occurs at a greater depth than at the end of the first packet. Furthermore, the largest bending moment range at the end of the fifth packet is at an even greater depth. This demonstrates that the bending moment profile progresses downwards as the soil softens due to larger amplitude packets of cyclic loading, and does not return up to the original position when the loading reduces.

This impact can be further observed in Figure 5-14, where profiles of normalised and non-normalised bending moment range for the first and last cycle within each packet of the sequence are presented. The displacement sequence at the hinge level is also shown in
Figure 5.14 with different colours for each packet, which correspond to the colours used to identify the bending moment range profiles. The depth of the largest bending moment range moves downwards during each cycle packet and this effect is partly preserved when the amplitude is reduced after larger packets. This causes the bending moment profiles in packets 1, 3 and 5 to differ due to the intervening cycles.

**Figure 5.14.** Profiles of bending moment range for cycles at the start (solid line) and end (dotted line) of each packet: (a) normalised, (b) non-normalised.

### 5.1.8. Discussion

The results presented in this paper show how the bending moment profile is affected when the adopted $p$-$y$ curves that are too stiff, such as proposed in the forthcoming ISO 19901-4 for fatigue conditions. Such curves may overpredict the bending moment, and therefore stress, on the shallow components of the system, which is where fatigue hotspots due to the
geometric irregularities and discontinuities exist. This effect will lead to lower fatigue life predictions for components above the mudline. In contrast, overly stiff $p$-$y$ curves will produce very low bending moments in deeper parts of the conductor, where important connections between extension and successive joints are located, as well as the top of cement when a shortfall case is analysed. This underprediction could lead to unsafe design, deeming a system component suitable when it could fail during the drilling campaign.

The opposite conclusions are drawn for curves that are too soft. For the alternative method proposed in this paper, slightly lower bending moments are predicted for the upper sections of the system, with slightly higher predictions in the lower sections. A potential reason for this could be that the limit of $1/S_i$ imposed for $P_{rem} / P_u$ in equations [5-8] and [5-10] is too onerous. Despite these observations, the alternative method proposed in this paper provides a better fit overall to the new data in this study, and also to the previously-published data on which the ISO fatigue curves were based. It is therefore our recommendation that the method introduced here (Eq. [5-7] to [5-12]) should be adopted in place of the ISO fatigue model, at least as a secondary design check if not as a full replacement of the ISO approach.

The PICSI model accurately tracks the changing p-$y$ response due to progressive cyclic remoulding and therefore bending moment profile for almost the entire cyclic sequence of the test, except for the last small amplitude cycling packet. As shown in Figure 5-13 and Figure 5-14, load history has an impact on the bending moment profile i.e., large amplitude cycles tend to shift the bending moment downwards. This increases the stresses on the lower components of the system and can only be properly modelled using a spring that accounts for load history in the soil. However, calibrating and implementing the model requires additional work and significant data, which in many cases may not be available. Nonetheless, it may represent the most appropriate approach when dealing with soils of high sensitivity or permeability, or where previous pore pressures due to cycling have been dissipated and the soil has hardened. As mentioned before, the test duration was not enough to dissipate most of the pore pressures generated by the cyclic sequence imposed, hence, only the damage modelling capabilities of PICSI can be observed. However, PICSI is able to model strength and stiffness recovery due to pore pressure dissipation, which could be more impactful for longer cycling periods or soils with higher $cv$. Another application of PICSI could be for reassessing the fatigue life of the well system components when a large amplitude cycling (storm) has occurred and disconnecting the system was not possible.
Based on findings in this paper the alternative method (Eq. [5-7] to [5-12]) is proposed for cases where the ‘fully remoulded’ condition dominates the fatigue life, with PICS I being suitable if the full loading history can be modelled.

5.1.9. Conclusions

This paper presents an alternative p-y curve method for fatigue analysis of conductors that links laboratory-measured soil parameters to pile-soil stiffness using rigorous theoretical concepts. This approach leads to p-y curves that give good agreement with new and previously-published data on the p-y stiffness (including the data from which the ISO fatigue p-y curves were based). A numerical back-analysis of centrifuge model test data shows that it this method predicts the measured bending moments within ±25%.

In comparison, the fatigue p-y curves recommended in the forthcoming ISO 19901-4 are much stiffer, overpredicting the bending moment (and therefore stress) in the upper sections of the conductor, and underpredicting them in the lower sections.

From comparisons with experimental data, it was observed that the PICS I model predicts the bending moments most accurately over the full test sequence, and can capture the movement of the fatigue hotspot that smears the fatigue damage down the pile. However, calibrating and implementing the PICS I model requires additional work and data, which in many cases may not be available.

On the basis of this study, it is recommended that the new simplified model is adopted for fatigue life estimation of drilling conductor systems, and in some situations the PICS I approach may be merited for more detailed analysis.

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CHAPTER 6 – CONCLUSIONS

This thesis presents the outcomes from extensive research into ‘whole life’ changes in soil-conductor lateral stiffness due to motion at seabed imposed by environmental conditions during drilling operations. The findings are based on results from experiments performed in the centrifuge and laboratory on two types of soft soil. This chapter presents the main conclusions from this research and recommendations for future work.

6.1. Outlining the current state of practice

Submissions to a prediction exercise run by the University of Western Australia were analysed to inform the current state of practice of soil-conductor analysis. The problem studied was the response of a model conductor in normally consolidated fine-grained soil, subjected to sequences of monotonic and cyclic loading in a centrifuge.

It was found that $p$-$y$ methods were the most widely used approaches and, in the case of the sequences explored in the tests, outperformed numerical (finite element/finite difference) approaches. While not specifically explored in this research, it is considered likely that numerical approaches are more time consuming than $p$-$y$ analysis. This suggests the latter is a better tool for these design problems, considering that soil-conductor fatigue analysis typically involves the application of thousands of displacement cycles.

Despite being a highly constrained problem, using a uniform and relatively well characterised soil sample, and well controlled installation methodology, the level of scatter in the predictions highlights there is room for improvement – particularly in regard to recommended practice. Further improvements will likely rely on more accurate estimation of $p$-$y$ curves for given materials and design applications.

6.2. Impact of previous load history on the stiffness of $p$-$y$ curves

For undrained cyclic loading at a given amplitude, there is a minimum (or fully remoulded) secant stiffness that is typically reached after a few hundred cycles, provided that the soil has not previously experienced higher strains. However, this stiffness does not remain “steady” with time, even when cycling continuously. Experimental testing shows that the undrained stiffness $K$ at any time depends on the maximum cyclic amplitude the soil has experienced to
date and the number of cycles applied. Periods of consolidation also strongly influence stiffness.

The soil-conductor stiffness on the decreasing amplitude phase of an event is lower than on the increasing amplitude phase. This means that, if the conductor has experienced sufficient cycles of a larger amplitude loading, its stiffness will be considerably reduced (post peak) and the hot-spot will migrate down the conductor, influencing fatigue life estimation.

A recovery in undrained fully remoulded secant stiffness was observed after periods of pore pressure dissipation. The stiffening effect of consolidation after cycling partially removes the reduction in stiffness of unloading relative to loading, and implies that assuming a post-large-amplitude secant stiffness for smaller cycling amplitudes is not appropriate. Ideally, modelling soil-conductor behaviour should include both the effect of previous remoulding and pore pressure dissipation.

It was verified that one-way and two-way cycling do not yield significantly different fully remoulded secant stiffness values, which agrees with results from previous studies. Differences were observed for very small displacements on the increasing amplitude phase of individual tests, although more data is needed to confirm if this is a real phenomenon. This implies that a small offset in displacements at seabed (<0.1 d), produced by vessel drift, could be analysed as if the vessel was positioned on top of the conductor, which is common practice when performing well-head systems fatigue life assessments.

Similarities were observed between trends in normalised fully softened stiffness (K) vs displacement in model tests and G/G_{max} vs shear strain behaviour from resonant column tests. This suggests that design curves could be constructed by scaling resonant column laboratory tests, although more work is required in this area to develop design guidance.

The p-y apparatus fully softened secant stiffness agrees with results from centrifuge tests results when normalised by the measured monotonic capacity. This capacity was observed (in the p-y apparatus) to be significantly higher than the theoretical capacity in kaolin clay. For tests in carbonate silt, the p-y apparatus showed an initial peak that was comparable to the centrifuge results, before increasing to values in excess of the theoretical capacity. Overall, the higher measured resistances may reflect an effect from the boundary conditions of the p-y apparatus.
6.3. *p-y* curves that account for load history

A new *p-y* model for the long term ‘whole life’ behaviour of laterally-loaded piles (PICS1) was developed and calibrated to model soil-conductor interaction in carbonate silt and kaolin clay, and validated against experimental data in carbonate silt. The model addresses an emerging requirement to capture the progressive changes in soil support that occur in soft soils around piles and well conductors, which influence the capacity, stiffness and fatigue of these systems. The model is inspired by model testing observations and theoretical solutions for each element of the behaviour, and combines a parallel-Iwan (PI) non-linear spring with a critical state-inspired overlay for the changing strength and stiffness.

The calibration method utilised an optimisation routine to determine the model parameters. Calibration of pore pressure dissipation (hardening) parameters were separated from calibration of the damage parameters.

A subsequent simplification to the PICS1 model was introduced that reduces the number of parameters, achieved by assuming a linear envelope for the fully remoulded strength line (*D* = 1). This resulted in estimates that agree well with the experimental data.

The calibrated parameters were first compared to the complete set of rigid conductor centrifuge and *p-y* apparatus tests, before being independently validated against centrifuge data using a flexible pile test. The latter used a model based on PICS1 generated springs distributed along the pile, which provided a good match to the cyclically induced changes in bending moment in the experiment.

6.4. Alternative simplified *p-y* curves

An alternative *p-y* method for fatigue analysis of conductors is presented, based on the normalised fully softened secant stiffness from increasing amplitude tests in the centrifuge and *p-y* apparatus. The method links soil elasticity concepts, maximum shear modulus, shear resistance and sensitivity, and broadly agrees with independent monotonic *p-y* curves for small displacements where no degradation is expected to occur.

The method was compared to centrifuge data on a flexible pile test on reconstituted carbonate silt and was found to predict slightly lower bending moment than the experimental data for the upper sections of the system, with slightly higher predictions in the lower sections. A potential reason for this could be that the independently measured sensitivity of the soil was used to limit
the ultimate lateral pressure, which may be too onerous. The alternative method provides a significantly better fit to experimental data (within 25% of measured bending moments) when compared to the recommendation in upcoming industry (ISO) guidelines (which underpredicted the measured bending moments by over 75% in some cases).

6.5. Potential impact of $p$-$y$ curves on fatigue analysis of conductors

Results presented in Chapter 5 show how the bending moment profile is affected when adopting $p$-$y$ curves that are too stiff, which are argued to be the case in the forthcoming ISO 19901-4 for fatigue conditions. Such curves may overpredict the bending moment, and therefore stress, on the shallow components of the system; which is where hotspots due to the geometric irregularities and discontinuities may occur. However, overly stiff $p$-$y$ curves will produce unconservative (low) bending moments in lower parts of the conductor, where important connections between extension and successive joints are located, as well as at the top of cement when a shortfall case is analysed – which could lead to unsafe design.

From comparisons with experimental data it was observed that the PICSI model predicts the bending moments accurately for the majority of the test sequence. Calibrating and implementing this model requires additional work and significant data, which in many cases may not be available. Nonetheless, it represents the most appropriate approach when dealing with soils of high sensitivity or permeability, or where previous pore pressures due to cycling have dissipated and the soil has hardened. Another application of PICSI could be for reassessing the fatigue life of the well system components when a large amplitude cycling (storm) has occurred and disconnecting the system was not possible.

While tempting to bound the analysis by performing the fatigue calculations with both overly stiff and overly soft $p$-$y$ curves, that may lead to an impractically large range in fatigue life. Based on findings outlined in Chapter 5, the alternative method is proposed for cases where only the ‘fully remoulded’ condition is important, with PICSI suitable for cases where the full loading history in important.

6.6. Other applications of the PICSI model

Even though in this research PICSI was developed and calibrated to model soil-conductor interaction in carbonate silt and kaolin clay, in its current formulation the model could be used to model lateral behaviour of piles for fixed offshore platforms or anchor piles for FPSO.
However, soil specific calibrations would have to be performed as well as calibrations to encompass the range of displacements of the piles.

Another application for the model could be as lateral soil reaction curve for monopiles for offshore wind turbines. Given that monopiles have small L/D ratios, the lateral behaviour cannot be modelled only with lateral p-y curves as it would be done for long slender piles, since other reaction components become significant. However, PICS-I could be used to model each of the reaction components (distributed load, distributed moment, base horizontal force, and base moment) independently, if calibrated with the appropriate data. Furthermore, PICS-I could be used as a lateral reaction curve for buckets in hybrid combined foundation types such as the newly proposed pile bucket foundations for offshore monopiles.

6.7. Future research

Future research to complement the findings described in this thesis includes:

- More accurate estimation of p-y curves for a wider range in soil types, as only reconstituted carbonate silt and kaolin clay were studied in this research.
- Introducing a ‘non-recoverable strength’ term to the PICS-I model for soils where structure, ageing and cementation between particles plays a large role in the soil sensitivity. This can be done through modification of select model equations, but requires testing of undisturbed natural soils for calibration and validation.
- The rigid pile centrifuge tests used for the calibration of the PICS-I model represent mostly a shallow wedge failure mechanism, whereas the p-y apparatus tests represent a deep flow around failure mechanism. While the combination of both for the calibration process provided an acceptable match for the flexible pile data, further refinement could explore the possibility of obtaining two sets of parameters: one for the shallow wedge mechanism and another for the deep flow-around mechanism.
- Applying the method proposed in this thesis to calibrate the parameters for the PICS-I model from (more readily available) multiple episodic simple shear tests, either directly or by scaling the stress-strain response measured in laboratory tests to p-y curves.
- Reviewing the performance of the different approaches on the fatigue life of a real drilling conductor system including the heavy packages at the top of the wellhead, riser connection and movements of the vessel. This could be done by implementing either a decoupled model, such as the one presented in this paper, or introduce coded
subroutines that support a coupled finite element model via updated springs at every timestep.