Suction caisson foundations for offshore wind energy installations in layered soils

Fangyuan Zhu
MEng, BEng

This thesis is presented for the degree of Doctor of Philosophy of The University of Western Australia

Centre for Offshore Foundation Systems,
School of Civil, Environmental and Mining Engineering
2018
DECLARATION

I, Fangyuan Zhu certify that:

This thesis has been substantially accomplished during enrolment in the degree.

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ABSTRACT

The monopod suction caisson has been proposed as a cost-effective foundation concept for offshore wind turbines. The suction caisson penetrates the soil primarily by its self-weight and then by suction pressure created through pumping water out of the caisson interior. This process does not require expensive installation equipment, is silent and can be reversed to remove the caisson. However, there is a lack of understanding on the performance of suction caissons to long-term and multidirectional cyclic loading (from wind and waves over the lifetime of an offshore wind turbine). The aim of this thesis is to investigate the changes of accumulated rotation and foundation-soil stiffness under the effects of cyclic loading. These two aspects are important to the design of a turbine system in fatigue and serviceability limit states. The soil profile of interest is layered sand over clay, which commonly occur in offshore wind farm sites in the North Sea of scope.

This thesis starts with a numerical study to investigate the effect of soil layering (sand, clay and sand over clay with various thicknesses of the upper sand layer) on performance of a caisson subjected to monotonic loading, which leads to the selection of soil profiles for the experimental study in the next step that investigates caisson behaviour under cyclic loading. The experimental study comprises two series of physical model tests at single gravity and a series of centrifuge model tests. The first series of single-gravity tests focuses on long-term cyclic loading with each test involving typically one million load cycles. The second series of single-gravity tests focuses on multidirectional loading in which the load direction changed between $0^\circ$ and $135^\circ$ in various patterns. The centrifuge tests consider cyclic loading of a suction caisson under realistic stress levels, drainage condition and caisson installation method. The database from the two sets of single-gravity tests provides trends of caisson performance under relevant loading conditions,
which together with the centrifuge database allow description of caisson behaviour at
prototype scale.

As revealed from this thesis, caisson response (including capacity, rotation and stiffness)
in sand over clay soils approach that measured in sand when the sand-clay interface is
located at or beneath the caisson skirt tip. In general, the change of caisson performance
under cyclic loading is controlled by densification in the sand and consolidation in the
clay. These effects typically lead to a stiffer caisson response (or increase in foundation-
soil stiffness) which needs to be considered when assessing the system dynamics for a
wind turbine system. In sand over clay soils with a sand layer thickness equal to half the
skirt length, one-way cyclic loading appears to result in larger caisson rotation than two-
way cyclic loading, whereas two-way cyclic loading was identified in previous research
as the most onerous loading symmetry in sand. A framework to predict the evolution of
caisson rotation with number of cycles for the specific soil profiles and cyclic loading
conditions (e.g. magnitude, symmetry and direction) has been calibrated on the basis of
the experimental database. The response under multidirectional loading investigated here
was found to be bound by relevant unidirectional response, and can therefore be predicted
using the same framework. The findings of this research are related to the field, including
discussion of the strict limit typically placed on out-of-verticality of offshore wind
turbines and the narrow target eigenfrequency range.
ACKNOWLEDGEMENTS

I would like to take this opportunity to express my sincerely gratitude to my supervisors Dr. Britta Bienen, Dr. Conleth O’Loughlin and Professor Mark Cassidy for their guidance and support. I am grateful for their helpful advice during discussions, timely feedback and constant encouragement when I faced difficulties in the research work. I appreciate they always had time to talk when I showed up at their office door unscheduled and their great patience to help me to improve academic writing in English.

I would like to thank Dr. Neil Morgan at Lloyd’s register EWEA, UK for the support. I appreciate his feedback on the testing conditions considered in this study and review comments on the papers. Thanks also to the geotechnical team in Lloyd’s register EWEA (Dr. Neil Morgan, Dr. David Christie, Dr. Keith Lauder, Dr. Nawras Hamdan and Dr. Guangquan Xu) for all their help making my visit productive and enjoyable. I also thank the BMWi (Bundesministerium fuer Wirtschaft und Energie, Federal Ministry for Economic Affairs and Energy) and the PTJ (Projektträger Jülich, project executing organisation) for providing the Fino database.

I thank the COFS technical team (John Breen, Guido Wager, Khin Thida Seint, Andrew Van Dam, Mike Turner, Manuel Palacios, Kelvin Leong, Adam Stubbs and Dave Jones) for their assistance with the experiments. Without them, the experiments presented in this thesis would not have been possible. I also thank the COFS administration team (Lisa Melvin, Monica Mackman, Rochelle Gunn, Dana Mamnone) for making things easier. Thanks to all colleagues and friends in COFS for their help in many different ways.
I acknowledge the financial support provided through the Scholarship for International Research Fees (SIRF) and Ad Hoc Postgraduate Scholarship. I am also grateful to Lloyd’s Register for funding this research project.

Finally, I would like to express special thanks to my family for their continual support, patience and encouragement from the other side of the world. I would like to reserve my greatest thanks to my boyfriend Wangcheng Zhang, to whom this thesis is dedicated for his unconditional love and heart-warming company.
AUTHORSHIP DECLARATION: CO-AUTHORED

PUBLICATIONS

This thesis contains work that has been published and/or prepared for publication.

Details of the work:
Location in thesis:
Chapter 4
Student contribution to work:
50%
Co-ordinating supervisor signature on behalf of all co-authors and date:
19/12/2017

Details of the work:
Location in thesis:
Chapter 5
Student contribution to work:
60%
Co-ordinating supervisor signature on behalf of all co-authors and date:
19/12/2017
Details of the work:

Location in thesis:
Chapter 6

Student contribution to work:
60%

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19/12/2017

The following publications are not part of this thesis but are relevant to this PhD study.

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I, Britta Bienen, certify that the student statements regarding their contribution to each of the works listed above are correct.

Date: 19/12/2017
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LIST OF SYMBOLS

- \( a \)  
  pore pressure factor

- \( A \)  
  plan area of the caisson

- \( c_v \)  
  vertical coefficient of consolidation

- \( d \)  
  cone penetrometer diameter

- \( D \)  
  caisson diameter

- \( D_i \)  
  caisson inner diameter

- \( D_o \)  
  caisson outer diameter

- \( D_r \)  
  relative density

- \( e \)  
  eccentricity of lateral load

- \( E_e \)  
  effective Young’s modulus

- \( F_{f,clay} \)  
  frictional force in clay

- \( F_{f,sand} \)  
  frictional force in sand

- \( G_{clay} \)  
  shear modulus of clay

- \( G_{sand} \)  
  shear modulus of sand

- \( H \)  
  lateral load

- \( H_{sand} \)  
  sand layer thickness

- \( I_p \)  
  plasticity index

- \( I_r \)  
  rigidity index

- \( k \)  
  unloading stiffness

- \( k_1 \)  
  unloading stiffness in the first cycle

- \( k_N \)  
  unloading stiffness in cycle number \( N \)

- \( k_c \)  
  coefficient of permeability in cellulose ether fluid saturated sand

- \( k_w \)  
  coefficient of permeability in water saturated sand
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<td>$k_f$</td>
<td>friction coefficient</td>
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<tr>
<td>$k_p$</td>
<td>end bearing coefficient</td>
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<tr>
<td>$K$</td>
<td>coefficient of lateral earth pressure</td>
</tr>
<tr>
<td>$L$</td>
<td>skirt length</td>
</tr>
<tr>
<td>$m$</td>
<td>multiple of diameter, $D$, over which vertical stress is enhanced</td>
</tr>
<tr>
<td>$M$</td>
<td>overturning moment</td>
</tr>
<tr>
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<tr>
<td>$M_{\text{max}}$</td>
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<tr>
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<tr>
<td>$q_{\text{seep}}$</td>
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</tr>
<tr>
<td>$R_a$</td>
<td>surface roughness</td>
</tr>
<tr>
<td>$s_u$</td>
<td>undrained shear strength</td>
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<tr>
<td>$t$</td>
<td>skirt thickness</td>
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<tr>
<td>$T$</td>
<td>dimensionless time</td>
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<tr>
<td>$u$</td>
<td>change in excess pore pressure</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
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<td>--------------------------------------------------</td>
</tr>
<tr>
<td>$u_i$</td>
<td>maximum excess pore pressure</td>
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<tr>
<td>$v'$</td>
<td>dimensionless penetration velocity</td>
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<td>$v_c$</td>
<td>viscosity of cellulose ether fluid</td>
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<td>$v_w$</td>
<td>viscosity of water</td>
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<td>skirt tip resistance</td>
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<td>interface friction ratio inside of caisson skirt</td>
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<td>vertical effective stress outside caisson</td>
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<tr>
<td>$\sigma_{v_i}'$</td>
<td>vertical effective stress inside caisson</td>
</tr>
<tr>
<td>$\phi'$</td>
<td>effective friction angle</td>
</tr>
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<td>parameter describing cyclic loading magnitude</td>
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<td>$\zeta_c$</td>
<td>parameter describing cyclic loading symmetry</td>
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<tr>
<td>$\delta$</td>
<td>angle of friction between skirt and soil</td>
</tr>
<tr>
<td>$\theta$</td>
<td>caisson rotation</td>
</tr>
<tr>
<td>$\theta_0$</td>
<td>maximum rotation during preloading to $M_{max}$</td>
</tr>
<tr>
<td>$\theta_{cyc}$</td>
<td>relative caisson rotation</td>
</tr>
</tbody>
</table>
\( \theta_N \)  
maximum rotation in cycle number \( N \)

\( \psi_D \)  
direction of lateral displacements at loading point

\( \psi_L \)  
load direction

\( \Delta \theta (N) \)  
accumulated rotation during cyclic loading

\( \Delta u \)  
lateral displacements at loading point

\( \Delta u_x \)  
component of \( \Delta u \) in the \( x \) direction

\( \Delta u_y \)  
component of \( \Delta u \) in the \( y \) direction
CHAPTER 1. INTRODUCTION

1.1 Background

Offshore wind energy has developed at an accelerated rate over the last decade. By the end of 2016, the global offshore wind capacity had reached 14.4 GW, including 2.2 GW newly installed during 2016 alone (GWEC, 2017). More than 85% of all offshore wind installations were located in European waters, led by the UK (5.2 GW) followed by Germany (4.1 GW) (with Figure 1.1 showing offshore wind farm sites in Europe). The European Wind Energy Association (EWEA) set a target of reaching 40 GW offshore wind energy capacity by 2020 and 150 GW by 2030 (EWEA, 2011).

Figure 1.1. European offshore wind farms sites by 2016 (Source: Planet OS)

The majority of offshore wind farms are located in water depths of 10 m to 30 m at distances of 15 km to 40 km from shore (IWES, 2017). In contrast to land based wind turbines where the majority of cost (71%) is in the turbine itself, in offshore wind projects the turbines only account for 32% of the capital cost while foundations and substructures account for around 30% (Moné et al., 2017). Optimising the design of foundations helps
to reduce the cost of offshore wind farms. A cost-effective foundation concept for offshore wind turbines is the monopod suction caisson structure, which consists of a large tower that is welded to the top of a single large suction caisson (or suction bucket, see Figure 1.2). The monopod suction caisson has been selected as one of four foundation designs that could potentially reduce the cost of offshore wind foundations by 30% in the Carbon Trust’s offshore accelerator programme (Carbon Trust, 2012).

Figure 1.2. (a) Monopod suction caisson (https://www.4coffshore.com) and (b) Offshore wind turbines supported by monopod suction caissons (http://www.harland-wolff.com).

Suction caissons are essentially large upturned steel buckets (with diameters typically 15 m to 25 m for monopods, depending on turbine size and water depth) that are forced into the seabed using the self-weight and suction pressure created through pumping water out of the caisson interior. In comparison to monopiles that require heavy installation
equipment (e.g. driving hammer), suction caissons are eco-friendly and offer the possibility of 1) fast and silent installation with no or limited seabed preparation and 2) ease of removal for recycling of steel materials after decommissioning. Fast foundation installation translates to time and hence costs savings of jack-up vessel hire. Due to the expected installation benefits, monopod suction caissons are increasingly considered as a promising foundation concept for water depth up to 55m.

Suction caissons were first used as a monopod foundation to support a 3MW turbine at Frederikshavn, Denmark in 2002 (Ibsen and Brincker, 2004) and were successfully employed in offshore wind farm sites at Horn Rev 2, Denmark in 2009 and at Dogger Bank, Dudgeon and Hornsea, UK in 2013 and 2014 for Meteorological Masts or trial installations (Tjelta, 2015). In contrast to oil and gas installation foundations where large vertical loads dominate, the loading on offshore wind turbine foundations are typically characterised by relatively small vertical loads in combination with large horizontal and moment loads (Houlsby and Byrne, 2000; Malhotra, 2011). Loads on offshore wind turbines include 1) self-weight of the system, 2) wind and wave loads and 3) operational loads (1P and 3P loading). The self-weight of the system (including turbine, tower and foundation) is the permanent or dead load in the vertical direction. The wind load acts on the rotor and tower of the wind turbine with magnitudes depending predominantly on the wind speeds, whereas the wave load acts on the substructure with magnitudes depending on wave height and wave period (Bhattacharya, 2014), see Figure 1.3. The 1P and 3P loading are inertial loads caused by vibrations due to mass and aerodynamic imbalance of the rotor and blade shadowing effect respectively (Yu et al., 2015) which typically control the design of system natural frequency.

Wind and wave loads are cyclic in nature, the repeated occurrence of which results in $10^7$ - $10^8$ cycles in the service life of the wind turbine (or long-term cyclic loading). The resulting forces from wind and waves can be approximated by a lateral load $H$ acting at
an eccentricity $e$ above seabed level and an overturning moment $M (M = H \cdot e)$ at the foundation, see Figure 1.3. These loads vary in magnitude, cyclic symmetry and direction with time due to the random occurrence of wind and waves. Figure 1.4 presents the wind and wave roses at Dogger Bank, North Sea, where the wind and waves are distributed in different directions (0° is North, 90° is East, 180° is South and 270° is West) inducing multidirectional loads at the foundation. The dominant directions of wind and waves generally coincide well (albeit with some misalignment) as waves are mostly caused by wind. It is worthwhile to note that wind with speeds between 8 m/s to 10 m/s and waves with heights less than 2 m have the highest probability of occurrence. Although the magnitudes are smaller than those occur in extreme conditions (e.g. wind speed > 25 m/s for storms, Salvação and Guedes Soares, 2015), these moderate loads (applied frequently in the long-term) are important for fatigue and serviceability considerations.

Figure 1.3. Loads on offshore wind turbine supported by monopod suction caisson.
Figure 1.4. (a) Wind rose at 100 height and (b) wave rose for wind driven sea at Dogger Bank, based on NORA10 data (Trøen, 2014).

The loads acting on offshore wind turbines are eventually withstood by the foundations. Guided by limit state design philosophy, the design considerations for a monopod suction caisson needs to satisfy the ultimate limit state (ULS), serviceability limit state (SLS) and fatigue limit state (FLS) (Bhattacharya, 2014). The ultimate criteria requires the assessment of ultimate capacity. The serviceability and fatigue criteria necessitates understanding of the evolution of 1) accumulated rotation and 2) foundation-soil stiffness over the typical 25 year lifetime of the turbine (Bhattacharya, 2014). Neither rotation nor foundation stiffness are constant but may change as an effect of soil-foundation interaction under long-term environmental loading (typically cyclic loading from wind and waves). The maximum allowable rotation is typically limited to $0.5^\circ$ (DNV, 2016).
The change of foundation stiffness will have an impact on the natural frequency of the wind turbine system, which is dynamically sensitive. Typically, an offshore wind turbine is designed as a ‘soft-stiff’ system with the system natural frequency falling between the forcing frequencies (i.e., 1P and 3P) as illustrated in Figure 1.5. Resonance may occur if the natural frequency of the system approaches these forcing frequencies. Reliable prediction of the evolution of foundation and system stiffness over the design life of a wind turbine is required to mitigate this risk of resonance.

![Figure 1.5. Relationship between system natural frequency and forcing frequencies (Bhattacharya 2014).](image)

Previous research addressing these two aspects (accumulated rotation and foundation-soil stiffness) was based on small-scale physical model tests at single gravity (Byrne and Houlsby, 2004; Kelly et al., 2006; Zhu et al., 2013; Foglia et al., 2014; Nielsen et al., 2017), centrifuge model tests (Watson and Randolph, 2006; Cox et al., 2014) and reduced scale field tests (Houlsby et al., 2005, 2006). These tests considered the effect of wind and wave loading on performance of monopod suction caissons but were limited to 1)
Chapter 1. Introduction

typically less than 10,000 load cycles (most are < 1,000 cycles) and 2) unidirectional cyclic loading. Besides, the experimental evidence of suction caisson responses from these studies are only relevant to single-layer soils. The soil profiles in many sites considered for offshore wind development, however, are layered. For example in the seabeds of the North Sea (see Figure 1.1) where 85% of connected offshore wind turbines in Europe have been erected (EWEA, 2016), firm, stiff and very stiff clays (strength from 40kPa to >288kPa) are common, covered by medium to high density surface sand layers (De Ruiter and Fox, 1975; Bond et al., 1997; BGS, 2002). Therefore, a suction caisson is likely to be embedded in a sand over clay soil profile; see Figure 1.6 for an example undrained shear strength profile in a North Sea seabed (where the dashes represent in-situ CPT profiles and the symbols are measured data from laboratory tests). There is need, therefore, for further research to understand the suction caisson responses with respect to long-term and multidirectional cyclic loading, in particular in layered soils.
Figure 1.6. An example of sand over clay profiles from an in situ CPT test in the North Sea seabed (Kellezi and Stadagaard, 2012).

1.2 Research objectives

This research aims to develop design considerations for the monopod suction caisson foundation subjected to long-term cyclic loading, with respect to the following

- Rotation accumulation;
- Changes to the foundation-soil stiffness;
- Effects of variation in load direction (or multidirectional cyclic loading).
The focus in this thesis is sand over clay layered soils, such that the findings from this work are relevant to the seabed conditions prevalent in the North Sea.

1.3 Research methodology

This thesis comprises of a preliminary numerical study considering monotonic loading and an extensive experimental study (including both single-gravity and centrifuge model tests) to investigate caisson response with respect to cyclic loading. The numerical study is included to understand the effect of soil layering (on ultimate capacity and deformation mechanisms) and guide the selection of layered soil profiles for the physical model tests. Centrifuge tests, though not economic for a large number of cycles, offer stress-level similitude to the field. Therefore, this approach is used to quantify the effects of stress level, drainage condition and installation method. Single-gravity tests are used to investigate the caisson behaviour with respect to long-term cyclic loading (involving approximately one million load cycles). A second series of single-gravity tests is included to investigate the effect of variation in load direction (or multidirectional cyclic loading). The combined database from single-gravity and centrifuge tests address the aims, providing evidence of suction caisson response in layered soil profiles for the evolution of accumulated rotation and foundation-soil stiffness and the effect of variation in load direction.

1.4 Thesis organisation

This thesis consists of 7 chapters. A brief outline of each chapter is given below:

1) This first chapter is an introduction to the thesis. The background, research objectives, research methodology and thesis organisation are outlined.
2) The second chapter introduces a literature review of research on monopod suction caissons with respect to installation, ultimate capacity and cyclic loading response.

3) The third chapter presents a numerical study on the monotonic loading behaviour of a suction caisson in sand, clay and sand over clay layered soils. The emphasis is this Chapter is to understand the caisson response (e.g. ultimate capacity and deformation mechanism) in layered soils and guide the selection of layered soil profiles for physical model tests.

4) Chapter 4, which has been published as a journal paper, presents results from single-gravity tests that focus on long-term unidirectional cyclic loading (each test involved approximately one million load cycles). The tests were performed in sand, clay and sand over clay layered soils over a range of cyclic loading magnitudes and symmetries. The evolution of caisson rotation and stiffness over the course of cyclic loading are discussed with respect to soil profiles and loading characteristics.

5) Chapter 5, which is a journal paper under review, discusses the centrifuge tests on unidirectional cyclic loading in which a caisson was penetrated into the soil in a suction-assisted installation process. This includes the modification of previous methods (for single-layer soils) to suit the prediction of required suction for caisson installation in sand over clay layered soils and reveals caisson performance at realistic stress levels, drainage conditions and following realistic caisson installation methods. A simple method is provided for quantifying caisson rotation at field scales, based on the combined results from the single-gravity (Chapter 4) and centrifuge tests (Chapter 5).

6) Chapter 6, which is in the format of a journal paper to be submitted, presents the results from single-gravity tests that focus on multidirectional cyclic loading. The
tests consider both individual and successive change in load directions, with effects on caisson rotation, stiffness as well as the directionality of the lateral displacements discussed. A simple quantification of caisson rotation for multidirectional loading is described based on predictions for unidirectional loading.

7) Chapter 7 is a closing chapter that summarises the original contributions, their implications for the design of a prototype suction caisson and recommendations for future work.

1.5 References


CHAPTER 2. LITERATURE REVIEW

2.1 Introduction and scope

This Chapter provides a state-of-art review related to the present study. Section 2.2 presents a brief review of suction caisson installation in layered soils and the existing prediction methods to determine the required suction pressure and seepage flow. Caisson installation, though not the focus of this thesis, is included to improve the understanding of installation responses presented in Chapter 5. Section 2.3 presents a brief background on the ultimate capacity of a monopod suction caisson. Section 2.4 discusses previous findings on the cyclic loading response of suction caissons with respect to accumulated rotation and foundation-soil stiffness. Finally, the gaps in knowledge are summarised in Section 2.5.

Since fewer studies are available investigating responses of monopod suction caissons in comparison to monopiles (with much larger aspect ratio $L/D$), some findings from studies on monopiles are also presented in this section for reference.

2.2 Suction caisson installation

2.2.1 Mechanisms of suction caisson installation in layered soils

The installation of a suction caisson generally consists of two stages: self-weight penetration and suction-assisted penetration. The self-weight is firstly utilised to penetrate a caisson to a certain depth creating a seal between skirt and soil. The caisson is then further installed through suction by pumping water from caisson interior to create a differential water pressure across the caisson lid. In sand or highly permeable soils, the applied suction will generate a seepage flow from the outside to the inside of the caisson, which acts to decrease the soil effective stresses inside the caisson and at the skirt tip,
therefore reducing the internal wall frictional and skirt tip resistance (Tjelta, 1994; Watson et al., 2006). Significant seepage flow, however, will be prevented in soils of low permeability. It is worthwhile to note that full installation of a suction caisson (with the skirt penetrated to full length and the lid invert coming into contact with the soil plug) is not always possible. If the applied suction reaches a critical value that reduces vertical effective stress inside the caisson to zero, the upward hydraulic gradient inside the caisson can induce local piping failure in the sand, which prevents the caisson from further penetration (Houlsby and Byrne, 2005a). The maximum allowable suction for installation in clay is also limited to avoid failure of the soil plug (Houlsby and Byrne, 2005b). Suction caisson installation in layered soils with the presence of both highly permeable and less permeable soil layers (e.g. sand over clay of interest in this study) is more problematic in comparison to single-layer soils.

For caisson penetration in sand approaching a low permeability clay layer, the effective stress may not be sufficiently reduced as seepage flows in the sand layer is restricted by the clay, which increases the penetration resistance and thus the required suction (Cotter, 2009). The higher requirement of suction pressure increases the possibility of piping failure, the occurrence of which could potentially halt the caisson penetration. As noted by Tran (2005), when the caisson is already penetrated into the less permeable layer, piping failure with the formation of pumping preferential flow channels along the skirt wall could also occur. This needs to be avoided to ensure full penetration of the suction caisson. Despite the complexity and the widespread occurrence of layered seabeds, the understanding on suction caisson installation in layered soils is limited to a few studies (e.g. Tran, 2005; Cotter, 2009; Senders, 2008), which needs to be improved.

2.2.2 Prediction of installation resistance

This section briefly reviews the methods for prediction of installation resistance in sand (Houlsby and Byrne, 2005a) and clay (Houlsby and Byrne, 2005b), as this thesis will
modify the two methods to enable the prediction in sand over clay layered soils. The two methods are based on an effective stress approach. An alternative method by Senders and Randolph (2009) that predicts the installation resistance in sand according to the cone penetration resistance (or CPT approach) is also introduced in this section as this method will be included later in this thesis for comparison.

Houlsby and Byrne method for sand

The Houlsby and Byrne (2005a) method calculates the caisson installation resistance in sand as the sum of friction on the outside and the inside of caisson and the end bearing at the skirt tip. The formula for installation resistance without suction \( R \) is

\[
R = \int_0^z \sigma'_vo \, dz (K\tan\delta)\, (\pi D_o) + \int_0^z \sigma'_vi \, dz (K\tan\delta)\, (\pi D_i) + \sigma'_end \, (\pi D_t) \tag{2.1}
\]

where \( \sigma'_vo \), \( \sigma'_vi \), and \( \sigma'_end \) are the effective vertical stresses outside, inside the caisson and at skirt tip, \( D_o \) and \( D_i \) are the outer and inner diameters of the caisson, \( D \) is the mean diameter \( (D = (D_o + D_i)/2) \), \( t \) is the skirt thickness, \( z \) is the penetration depth and \( K\tan\delta \) is the friction coefficient. When a suction pressure \( p \) is applied at the caisson lid, the installation resistance maintains equilibrium with the applied suction force and caisson submerged self-weight \( W \) as

\[
R = W + p\left(\frac{\pi D_i^2}{4}\right) \tag{2.2}
\]

The determination of effective stresses in the method (i.e. \( \sigma'_vo \), \( \sigma'_vi \), and \( \sigma'_end \)) include 1) the enhancement of vertical stress under the downward movement of the caisson and 2) the reduction in the internal wall frictional resistance and skirt tip resistance as the effect of the applied suction. In the special case that the enhancement of vertical stress occurs in a constant zone outside of skirt and uniform stress is assumed within the caisson, the required suction for caisson installation can be determined from
Chapter 2. Literature review

\[ W + p \left( \frac{\pi D_i^2}{4} \right) = \left( \gamma' + \frac{a_s}{z} \right) Z_o^2 \left[ \exp \left( \frac{Z}{Z_o} \right) - 1 - \left( \frac{Z}{Z_o} \right) \right] \times (K\tan\delta)_o (\pi D_o) \]

\[ + \left( \gamma' - \frac{(1 - a)s}{z} \right) Z_i^2 \left[ \exp \left( \frac{Z}{Z_i} \right) - 1 - \left( \frac{Z}{Z_i} \right) \right] \times (K\tan\delta)_i (\pi D_i) \]

\[ + \left[ \left( \gamma' - \frac{(1 - a)s}{z} \right) Z_i \left[ \exp \left( \frac{Z}{Z_i} \right) - 1 \right] N_q + \gamma' t N_f \right] \times (\pi D_t) \]

Equation (2.3)

with

\[ Z_o = \frac{D_o (m^2 - 1)}{4(K\tan\delta)_o}, \quad Z_i = \frac{D_i}{4(K\tan\delta)_i} \]

\[ N_q = \tan^2(45° + \varphi'/2)e^{\pi\tan\varphi'} \]

\[ N_f \approx 2(N_q + 1)\tan\varphi' \]

In Equation (2.3), \( m \) is the multiple of the diameter over which the vertical stress is enhanced and \( a \) is the ratio of excess pore pressure at the tip of the caisson skirt to the lid invert. The value of \( a \) is dependent on the ratio of permeability within to outside the caisson (i.e. \( k_{\text{inside}}/k_{\text{outside}} \)) and its variation with penetration depth \( z \) can be expressed as

\[ a = \frac{a_1 k_{\text{inside}}/k_{\text{outside}}}{(1 - a_1) + a_1 k_{\text{inside}}/k_{\text{outside}}} \]

Equation (2.4)

with

\[ a_1 = 0.45 - 0.36 \left[ 1 - \exp\left( -\frac{Z}{0.48 D} \right) \right] \]

Houlsby and Byrne method for clay

The method Houlsby and Byrne (2005b) method calculates the caisson installation resistance in clay as the sum of frictional resistance on the outside and inside of a caisson and end bearing at the skirt tip. The expression for installation resistance without suction is

\[ R = z\alpha_o \bar{s}_u (\pi D_o) + z\alpha_i \bar{s}_u (\pi D_i) + (\gamma' z N_q + s_u N_c) (\pi D_t) \]

Equation (2.5)
where \( \alpha_o \) and \( \alpha_i \) are interface friction ratios on the outside and inside of the caisson respectively, \( s_u \) is the undrained shear strength at penetration depth \( z \), \( \bar{s}_u \) is the average undrained shear strength between mudline and skirt tip penetration depth, \( N_q \) and \( N_c \) are bearing capacity factors. For undrained analysis \( N_q = 1 \).

For suction-assisted penetration, this method considers the reduction in skirt tip resistance by the applied suction pressure assuming that the flow of soil beneath the skirt occurs entirely inwards. This results in the calculation of required suction pressure \( p \) as

\[
W + p \left( \frac{\pi D_o^2}{4} \right) = z\alpha_o\bar{s}_u(\pi D_o) + z\alpha_i\bar{s}_u(\pi D_i) + (\gamma'z + s_u N_c)(\pi D_t) \tag{2.6}
\]

Senders and Randolph method

In the Senders and Randolph (2009) method using the CPT approach, the caisson installation resistance \( R \) in relation to the cone tip resistance \( q_c \) is expressed as (Senders and Randolph, 2009)

\[
R = \pi D_t k_f \int_0^z q_c(z) \, dz + \pi D_o k_f \int_0^z q_c(z) \, dz + \pi D_t k_p q_c(z) \tag{2.7}
\]

where \( k_f \) is the friction coefficient and \( k_p \) is the end-bearing coefficient. The required suction for caisson installation in sand is estimated assuming 1) the internal wall friction and tip resistance reduce linearly from values calculated from Equation (2.7) at no suction to zero when suction reaches the maximum allowable (or critical) value \( p_{\text{crit}} \), and 2) the external wall friction remains unaffected by the applied suction. These yield the vertical equilibrium

\[
W + p \left( \frac{\pi D_i^2}{4} \right) = \int_0^z q_c(z) \, dz + \left[ \pi D_o k_f \int_0^z q_c(z) \, dz + (\pi D_t) k_p q_c(z) \right] \times \left( 1 - \frac{p}{p_{\text{crit}}} \right) \tag{2.8}
\]

with
Chapter 2. Literature review

\[ p_{\text{crit}} = \gamma'z \left( \pi - \arctan\left(5\left(\frac{z}{D}\right)^{0.85}\right) \left(1 - \frac{2}{\pi}\right) \right) \]

The prediction of suction pressure from Equation (2.8) should not exceed the maximum allowable value, i.e. \( p \leq p_{\text{crit}} \).

2.2.3 Prediction of seepage flow

This thesis will use the method of Houlsby and Byrne (2005a) to predict seepage flow and thus caisson penetration rate during suction installation. In this method, the seepage flow \( q_{\text{seep}} \) under an applied suction \( p \) is determined using Darcy’s law as

\[ q_{\text{seep}} = \frac{kDp}{\gamma_w}F \]

where \( \gamma_w \) is the effective unit weight of water, \( k \) is the soil permeability and \( F \) is a dimensionless factor that depends on the penetration depth \( z/D \) and permeability ratio \( k_{\text{inside}}/k_{\text{outside}} \). Values of \( F \) are determined from numerical analyses in this method and can also be estimated from the simplified expression \( F = (1-a)(k_{\text{inside}}/k_{\text{outside}})\pi/(4z/D) \).

2.3 Ultimate capacity of monopod suction caisson

The ultimate load carrying capacity of foundations is essential for the safe design of offshore wind turbines, which determines the possibility of system failure under extreme loading conditions. The ultimate capacity, related to the design in ultimate limit state (ULS; DNV (2016)), is also relevant to other design limit states (e.g. serviceability and fatigue limit states, SLS and FLS). This thesis will use the ultimate capacity to provide a reference for the cyclic loading magnitude.

Investigations on ultimate capacity of monopod suction caissons mainly considered monotonic loading, i.e. lateral loading with a constant eccentricity \( e = M/H \). Figure 2.1a presents an example suction caisson response (i.e. moment in relation to rotation) under monotonic loading. As there is no clear ultimate, the ‘intersection method’ (or a graphical
conclusion method) may be used to determine the ultimate capacity (as illustrated in Figure 2.1a). This method determined the ultimate capacity as the load corresponding to the intersection point of two tangential lines along the initial and latter proportions of the load-displacement curve.

Monotonic loading in previous studies was considered in reduced scale field tests (e.g. Larsen 2008), physical model tests (e.g. Byrne et al., 2003; Villalobos et al., 2005, 2009; Larsen et al., 2013; Zhu et al., 2011, 2014) and numerical analysis (e.g. Bransby and Yun 2009; Barari and Ibsen, 2012; Achmus et al., 2013a; Deb and Singh, 2016, 2017). These studies addressed ultimate capacity of suction caissons with respect to various factors, e.g. vertical load, loading eccentricity, caisson geometry and installation method, but were restricted to single-layer soils (e.g. sand and clay).

Bearing capacity of skirted foundations, subjected to combined vertical (V), horizontal (H) and moment (M) loading, has been investigated (Bransby and Randolph, 1998; Bransby and Yun, 2009; Vulpe et al., 2013), where the benefits of skirts on bearing capacity, especially the horizontal capacity, have been acknowledged. Several studies have been conducted to reveal the soil layering effect on the bearing capacity of skirted foundations (e.g. Kellezi et al. 2008). However, no direct recommendation for assessment of capacity has been provided due to lack of rigorous evidence. Therefore it requires further studies on how soil layering (e.g. in sand over clay soils that is of interest in this thesis) affects caisson ultimate capacity.
2.4 Cyclic loading response of monopod suction caisson

To date, physical model tests have been the main approach to investigate the response of monopod suction caissons under cyclic loading. These were carried out either at single gravity (e.g. Byrne and Houlsby, 2004; Zhu et al., 2013; Foglia et al., 2014; Nielsen et al., 2017a) or at realistic stress levels in a centrifuge (Watson and Randolph, 2006; Cox et al., 2014). A limited number of studies have been carried out through reduced scale field tests (Houlsby et al., 2005, 2006), field measurements (Nielsen et al., 2017b) and numerical analysis (Achmus et al., 2013b; Zhang and Chen, 2017). This section summarises previous findings regarding the effect of cyclic loading on 1) accumulated rotation and 2) foundation-soil stiffness.

2.4.1 Accumulated rotation

Most previous studies considered cyclic loads with large amplitudes and small cycle numbers \((N < 1000)\) to study extreme loading events. These studies indicated the favourable effect of an increase in vertical load (Villabobos, 2006) and preloading (Byrne...
and Houlsby, 2004; Watson and Randolph, 2006) on the response of a suction caisson. In
comparison to moderate load cycles, cyclic loads with large amplitudes increased the
accumulation of plastic rotation (Villabobos 2006; Houlsby et al., 2005, 2006). Caisson
behaviour under extreme loading conditions were characterised through a backbone curve
from monotonic tests (Byrne and Houlsby, 2004) or using a simple strain accumulation
 technique (Watson and Randolph, 2006).

Several physical model tests (Zhu et al., 2013; Foglia et al., 2014; Nielsen et al., 2017a;
Cox et al., 2014) were recently carried out to investigate caisson behaviour in sand with
respect to a larger number of loading cycles. These studies provided an understanding of
caisson rotation in relation to number of cycles through parametric studies, although the
involved cycle numbers (typically $N = 1,000 \sim 10,000$) were three to four orders of
magnitude fewer than those required for fatigue design. In the framework of these studies,
cyclic loads were characterised by parameters $\zeta_b$ and $\zeta_c$ (LeBlanc et al., 2010)

$$
\zeta_b = \frac{M_{\text{max}}}{M_{\text{ult}}}, \quad \zeta_c = \frac{M_{\text{min}}}{M_{\text{max}}} \tag{2.10}
$$

where $M_{\text{min}}$ and $M_{\text{max}}$ are the minimum and maximum moments in a load cycle and $M_{\text{ult}}$
is the ultimate moment capacity. The parameter $\zeta_b$ characterises the cyclic load magnitude
and can vary between 0 and 1. The parameter $\zeta_c$ characterises the cyclic load symmetry
and can vary between -1 and 1, with $\zeta_c \geq 0$ and $\zeta_c < 0$ representing one-way and two-way
loading respectively. The accumulated rotation in these studies was normalised by the
rotation corresponding to $M_{\text{max}}$ in a static loading test which enables application of the
results at scales other than those associate with the model tests. Figure 2.2 presents the
effect of cyclic loading characteristics ($\zeta_b$ and $\zeta_c$) on accumulated rotation (after
normalisation) reported in Zhu et al. (2013) for a caisson in sand. These together with the
finding of Cox et al. (2014) and Foglia et al. (2014) illustrate the larger accumulation of
rotation for 1) larger cyclic magnitude (or larger $\zeta_b$) and 2) two-way cyclic loading at $\zeta_c$
\approx -0.7 \text{ for fully drained conditions in sand. Nielsen et al. (2017a) reported a slightly different observation in partially drained sand where the caisson had experienced precycling: symmetric two-way loading (}\zeta_c \approx -1\text{) led to the largest accumulated rotation while no rotation was accumulated for one-way loading with } \zeta_c > 0.17. \text{ The difference was thought to result from higher excess pore pressures under two-way loading compared with one-way loading. The accumulation of rotation in these studies was characterised by a power law relationship (see dashed lines in Figure 2.2), which provides guidance for estimating the caisson rotation in sand with respect to a given number of cycles and for particular load characteristics.}
Nielsen et al. (2017b) presented the rotation measurement of a monopod suction caisson supporting a met mast, which was installed at Dogger Bank. The caisson (diameter $D = 15$ m and skirt length $L = 7.5$ m) was installed through dense sand to underlying stiff clay. It was observed that 1) the accumulated rotation was $0.06^\circ$ after one to two months of operation and stabilised afterwards, and 2) the rotation was reversed after storms and no permanent deformation occurred when large waves passed the structure. The absence of permanent deformation during storms was explained by the generation of negative pore pressure, which enhances the foundation performance within a short duration and thus counteracts the loads from the wave impact. These observations help to understand the realistic rotation of a caisson supporting a wind turbine.

The above findings (apart from Nielsen et al. (2017b)) are all relevant to caisson performance under unidirectional cyclic loading. As discussed in Chapter 1, the directions of wind and wave loads vary with time such that the foundations are subjected to multidirectional cyclic loading. Its effect on caisson behaviour is unclear. Several studies
are found on lateral loaded monopiles under multidirectional displacement paths (Mayoral et al., 2005; Su, 2012; Su and Li, 2013), multidirectional monotonic loading (Levy et al., 2007) and multidirectional cyclic loading (Sheil, 2014; Rudolph et al., 2014; Nanda et al., 2017). These studies revealed the effect of loading in one direction on the soil performance in other directions. Sheil (2014) reported that a static load in orthogonal direction to cyclic loading caused significant change to load-displacement response of a pile in sand under cyclic loading and the effect was dependent on the cyclic load symmetry ($\zeta_c$). Rudolph et al. (2014) and Nanda et al. (2017) observed a larger pile head displacement under multidirectional loading in comparison to unidirectional loading for a pile in sand. Although the above findings are relevant to monopiles (with much larger aspect ratio $L/D$ than suction caisson), it is implicated that multidirectional loading may lead to a different response of a suction caisson in comparison to unidirectional loading, which requires further research.

### 2.4.2 Foundation-soil stiffness

Unloading stiffness is commonly used to characterise the stiffness of the caisson-soil system during cyclic loading as this reflects the elastic stiffness. Under short-term and extreme cyclic loading events, reduced scale field tests (Houlsby et al., 2005, 2006) and physical model tests (Byrne and Houlsby, 2004; Villalobos, 2006) indicated a stiff caisson response at low cyclic amplitudes, with a gradual reduction of stiffness and increase of hysteresis at larger amplitudes.

Under long-term cyclic loading with constant load amplitudes, Zhu et al. (2013) and Cox et al. (2014) observed an increase or maintenance of the stiffness from tests on suction caisson in sand; see Figure 2.3 for typical results of Zhu et al. (2013). The changes were smaller than those observed from tests on monopiles in sand (e.g. LeBlanc et al., 2010; Li et al., 2010; Cuéllar et al., 2012; Nicolai and Ibsen, 2015; Chen et al., 2015), which indicated the general trend of increasing stiffness with number of cycles but at a
decreasing rate. For example, the increase is 13% to 49% after approximate 50,000 cycles in Nicolai and Ibsen (2015) and 20% to 30% after 5,000 cycles in Chen et al. (2015). The increase of stiffness results from the densification of sand particles as observed in Cuéllar et al. (2012) through topographic measurement of the soil surface before and after testing for a monopile in sand. This study, involving 5 million load cycles, also indicated that the soil around the foundation would reach its maximum density at some point between \(10^4\) to \(10^5\) cycles and therefore no further increase of stiffness would take place afterwards.

Cyclic loading characteristics appear to have an effect on the caisson stiffness (Zhu et al., 2013), i.e. stiffness decreases with increasing \(\zeta_b\) (cyclic load magnitude) and \(\zeta_c\) (cyclic load symmetry). This is consistent with the observations of LeBlanc et al. (2010) and Nicolai and Ibsen (2015) for monopiles in sand. This effect, however, was observed to be minimal in Cox et al. (2014) with loading conducted within or close to the elastic range of soil. The caisson stiffness appears to less dependent on the vertical load than the cyclic loading characteristics for the two load levels \((V/\gamma D^3 = 0.19\) and 0.57\) tested in Zhu et al. (2013).

![Figure 2.3. Unloading stiffness with number of loading cycles (Zhu et al., 2013).](image)
Chapter 2. Literature review

The studies discussed in this section illustrate the effect of cyclic loading characteristics (e.g. amplitude and symmetry) on the evolution of accumulated rotation and foundation-soil stiffness. The applicability of these findings require evaluation at larger number of cycles, in layered soil profiles (single-layer sand are mostly considered in these studies) and more realistic testing conditions (in these studies jacked installation and fully drained response in sand are typically involved).

2.5 Gaps in knowledge

Previous studies help to understand the behaviour of suction caissons when used as monopod foundations for offshore wind turbines. The observations, mostly relevant to single-layer soils (e.g. sand and clay), may not be applicable for layered soils that are common in offshore wind farm sites. For studies investigating the ultimate capacity and the cyclic loading performance, caisson installation was simplified as jacked (in most physical model tests) or embedded in soil (in numerical analyses), unlike the suction installation that is performed in the field. Despite moderate load cycles having the highest probability of occurrence and that may result in fatigue failure during the lifetime of a wind turbine, most previous studies focused on monotonic loading or short-term cyclic loading with large amplitudes, which are relevant to the design under extreme loading events. Several recent studies (Zhu et al., 2013; Foglia et al., 2014; Cox et al., 2014; Nielsen et al., 2017a) utilised the framework of LeBlanc et al. (2010) for monopiles in sand and provided guidance for the prediction of caisson response in relation to relatively larger number of cycles. Findings from these studies, however, are restricted to typically less than 10,000 cycles (which are much lower than the millions of load cycles required for fatigue design), unidirectional loading, and a single-layer sand profile. There is need for further research, therefore, to understand the behaviour of a monopod suction caisson in layered soils, especially following suction installation and under long-term and
multidirectional cyclic loading that the offshore environment apply to the wind turbine over its lifetime.

2.6 References


CHAPTER 3. EFFECT OF SOIL LAYERING ON MONOTONIC RESPONSE OF SUCTION CAISSON

3.1 Introduction
The aim of this section is to investigate the monotonic loading behaviour of a monopod suction caisson in single-layer and sand over clay layered soils through numerical analyses. This will generate an understanding of the effect of soil layering on the deformation mechanism and the ultimate capacity of the caisson-soil system, and provide guidance for the selection of soil profiles in the physical model tests.

3.2 Numerical model and analysis details
3.2.1 Geometry, meshes and boundary conditions
A three-dimensional finite element model of a monopod suction caisson embedded in soil was developed for the analyses. Due to the symmetry in both geometry and loading conditions, only half of the caisson-soil system (see Figure 3.1) was studied to save computational time. The suction caisson in the analyses has a diameter $D = 16$ m, skirt length $L = 8$ m and skirt thickness $t = 0.08$m, which are typical values for monopod suction caissons in prototype. The ratio $t/D = 0.005$ is typical in practice, but the skirt thickness has been increased (doubled) here to improve the calculation efficiency. The thickness of the caisson lid is $0.01D$. To minimise boundary effects, the height and width of the soil domain were selected as $3D$ and $7D$.

The commercial software package Abaqus 6.14 was used for the analyses. Both the suction caisson and soil were meshed by eight-noted brick elements with reduced integration ‘C3D8R’. The meshes (Figure 3.1) were refined through preliminarily trials to achieve the desirable accuracy as well as acceptable computational efficiency. The
whole model comprises of 22388 elements and 77184 degrees of freedom in total. The analyses were run via the Abaqus/Explicit approach. In the analyses, the suction caisson was treated as a rigid body and was assumed to be in place when generating the numerical model (or the installation process of caisson was neglected).

Figure 3.1 Distribution of mesh in the numerical model.

Displacement boundary conditions were imposed on the soil domain, with movement in all directions prevented at the base, out-of-plane movements prevented on the plane of symmetry and lateral movements prevented at the sides. The suction caisson was restricted from out-of-plane movements, but was free to move in other directions.

3.2.2 Soil and interface properties

Dense sand and stiff clay were considered in the analyses to represent soils characteristic of the seabeds in the North Sea (De Ruiter and Fox, 1975; Bond et al., 1997; BGS, 2002).
The clay was modelled as a linearly elastic – perfectly plastic material with a Tresca failure criterion. The undrained shear strength of clay is \( s_u = 80 \) kPa and Young’s modulus was selected to be \( E = 500 \) \( s_u \). The sand was modelled as a linearly elastic – plastic material with a Mohr-Coulomb failure criterion. The effective friction angle and dilation angle of sand was set to \( \varphi' = 40^\circ \) and \( \psi' = 10^\circ \) respectively. Table 3.1 summarises the sand and clay properties.

Table 3.1 Soil parameters for clay and sand.

<table>
<thead>
<tr>
<th></th>
<th>Clay</th>
<th>Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective unit weight ( \gamma' )</td>
<td>8 kN/m(^3)</td>
<td>11 kN/m(^3)</td>
</tr>
<tr>
<td>Young’s modulus ( E )</td>
<td>40000 kPa</td>
<td>60000 kPa</td>
</tr>
<tr>
<td>Poisson’s ratio ( \nu )</td>
<td>0.495</td>
<td>0.25</td>
</tr>
<tr>
<td>Internal friction angle ( \varphi' )</td>
<td>0°</td>
<td>40°</td>
</tr>
<tr>
<td>Dilation angle ( \psi' )</td>
<td>0°</td>
<td>10°</td>
</tr>
<tr>
<td>Undrained Shear Strength ( s_u ) (for clay)</td>
<td>80 kPa</td>
<td>-</td>
</tr>
<tr>
<td>Cohesion ( c' ) (for sand)</td>
<td>-</td>
<td>0.1 kPa</td>
</tr>
</tbody>
</table>

The soil profiles considered in the analyses are single-layer sand, clay and sand over clay layered soils with thickness of upper sand layer (\( H_{\text{sand}} \)) varying at 2 m, 4 m, 6 m, 8 m, 12 m and 16 m (or \( H_{\text{sand}}/L = 0.25, 0.5, 0.75, 1, 1.5 \) and 2), see Figure 3.2. The bottom of the sand layer is located above the skirt tip when \( H_{\text{sand}} \) is 2 m, 4 m 6 m and 8 m, and beneath the skirt tip when \( H_{\text{sand}} \) is 12 m and 16 m. For \( H_{\text{sand}}/L = 1 \) where skirt tip is at the clay surface, interaction properties for sand are applied at the skirt tip as sand particles may be carried down with the penetrating caisson.

The caisson-clay interfaces were assumed to be fully rough (bonded) with normal detachment prevented, while normal detachment was allowed at the interface at the sand layer. This was selected on the assumption that the negative excess pore pressure developed between the caisson and clay (low permeability) is sufficient to resist the separation, which however is not the case at the caisson-sand interface. The interaction at the caisson-sand interface was defined through the Coulomb friction law. If the surfaces
of caisson and sand were in contact, the normal pressure as well as the tangential resistance would be transferred via the interfaces while no force would be transferred if the interfaces were detached. The contact friction angle, which characterises the tangential resistance via caisson-sand interfaces, was chosen as two-thirds of the effective friction angle of the sand (i.e. the frictional coefficient \( \delta = \tan(2\phi'/3) \)).

Figure 3.2 Sketch of suction caisson in different soils: (a) clay, (b) \( H_{\text{sand}} = 2 \) m, (c) \( H_{\text{sand}} = 4 \) m, (d) \( H_{\text{sand}} = 6 \) m, (e) \( H_{\text{sand}} = 8 \) m, (f) \( H_{\text{sand}} = 12 \) m, (g) \( H_{\text{sand}} = 16 \) m and (h) sand.

### 3.2.3 Load application

Figure 3.3 illustrates the implementation of loading in numerical analyses. The reference position \( (RP) \) for loads is set to the centre of the caisson at the lid invert. A constant load
is applied in the vertical direction to simulate the constant self-weight of a wind turbine system (apart from the validation analyses). The lateral load \( H \), simulating wind and wave loads, is applied through a constant rate of lateral displacement, \( h_{\text{disp}} \), (sufficiently slow to ensure quasi-static response) at an eccentricity \( e \) above the \( RP \), which results in an overturning moment \( M = H e \) at the caisson in addition to the lateral load. The ratio of the moment to the horizontal load (i.e. \( M/H \)) remains constant at \( e \) as \( h_{\text{disp}} \) increases.

Simulations were carried out under constant vertical loads of 10 MN (corresponding to the self-weight of the whole structure) and lateral loading eccentricity of \( e = 3.5 D \) (i.e. 56 m) which are the typical values for 5~7.5 MW offshore wind turbines.

3.2.4 Validation

A series of analyses were conducted in single-layer clay to validate the numerical model, with one for vertical compressive loading and others for lateral (or eccentric lateral) loading. The obtained ultimate loads (i.e. \( V_{\text{ult}} \) and \( H_{\text{ult}} \)) are divided by \( A_{su} \) (where \( A \) is the

\[
\text{Figure 3.3 Sketch of loading in numerical analyses.}
\]
area of caisson lid and $s_u$ is the undrained shear strength of clay) allowing for the bearing capacity factors (i.e. $N_{cv} = V_{ult}/A_{su}$ and $N_{ch} = H_{ult}/A_{su}$) to be compared with values from the literature, which are in good agreement, as shown in Table 3.2 and Table 3.3. These ultimate loads were determined according to the ‘intersection method’ as illustrated in Section 2.3.

Table 3.2 Values of $N_{cv}$ obtained from numerical analyses and analytical solutions (caisson aspect ratio: $L/D = 0.5$).

<table>
<thead>
<tr>
<th>Source</th>
<th>$N_{cv}$</th>
<th>Soil-skirt interface</th>
<th>Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>This study</td>
<td>10.50</td>
<td>Rough</td>
<td>Numerical</td>
</tr>
<tr>
<td>Mana et al. (2010)</td>
<td>10.50</td>
<td>Rough</td>
<td>Numerical</td>
</tr>
<tr>
<td>Gourvenec and Mana (2011)</td>
<td>10.42</td>
<td>Rough</td>
<td>Numerical</td>
</tr>
<tr>
<td>Hung and Kim (2012)</td>
<td>10.78</td>
<td>Rough</td>
<td>Numerical</td>
</tr>
<tr>
<td>Zhan and Liu (2010)</td>
<td>9.79</td>
<td>Coulomb Friction</td>
<td>Numerical</td>
</tr>
<tr>
<td>Martin (2001)</td>
<td>9~12*</td>
<td>Rough</td>
<td>Analytical solution</td>
</tr>
</tbody>
</table>

*9 and 12 in range 9~12: values determined from lower and upper bound solutions

Table 3.3 Values of $N_{ch}$ obtained from numerical analyses (caisson aspect ratio: $L/D = 0.5$).

<table>
<thead>
<tr>
<th>Source</th>
<th>$N_{ch}$</th>
<th>Soil-skirt interface</th>
<th>Method</th>
<th>Eccentricity</th>
</tr>
</thead>
<tbody>
<tr>
<td>This study</td>
<td>4.88</td>
<td>Rough</td>
<td>Numerical</td>
<td>0 $D$</td>
</tr>
<tr>
<td>Hung and Kim (2012)</td>
<td>4.82</td>
<td>Rough</td>
<td>Numerical</td>
<td></td>
</tr>
<tr>
<td>Zhan and Liu (2010)</td>
<td>4.29</td>
<td>Coulomb Friction</td>
<td>Numerical</td>
<td></td>
</tr>
<tr>
<td>This study</td>
<td>0.70</td>
<td>Rough</td>
<td>Numerical</td>
<td>2 $D$</td>
</tr>
<tr>
<td>Zhan and Liu (2010)</td>
<td>0.68</td>
<td>Coulomb Friction</td>
<td>Numerical</td>
<td></td>
</tr>
</tbody>
</table>

### 3.3 Results and discussion

Figure 3.4 provides the monotonic responses (in the form of overturning moment $M$ in relation to rotation $\theta$) for the soil conditions considered. The moment resistance decreases as the thickness of the upper sand layer $H_{sand}$ increases when $H_{sand}/L < 1$ and is very close for cases with $H_{sand}/L \geq 1$. The ultimate moment capacity $M_{ult}$ (determined from the ‘intersection method’) decreases approximately linearly with $H_{sand}$ when $H_{sand}/L < 1$ and remains constant when $H_{sand}/L \geq 1$, see Figure 3.5. The deformation mechanisms underlying the above responses are presented in Figure 3.6.
Figure 3.4 Moment and rotation responses of suction caisson in different soil conditions

\[(e = 3.5\, D, V = 10\, MN)\].

Figure 3.5 Moment and rotation responses of suction caisson in different soil conditions

\[(e = 3.5\, D, V = 10\, MN)\].
In single-layer clay, the soil displacement contour exhibits a scoop mechanism with rotational displacement dominating the failure mechanism. In this case, the rotation centre is located approximately at the centre of caisson, which is consistent with the ‘scoop-slide mechanism’ as proposed Bransby and Yun (2009). For single-layer sand cases with $H_{sand}/L \geq 1$, a wedge mechanism is apparent with lateral displacement dominating the movement of soil and the area of mobilised soil is mainly above the skirt tip level (see Figure 3.6e ~ h), where the rotation centre is lower than that in single-layer clay case. Since the soil failure mechanism is confined to the sand layer for $H_{sand}/L \geq 1$, it is reasonable that load-displacement responses and the ultimate capacity (in Figure 3.4 and Figure 3.5) are similar. For sand with a layer thickness of $H_{sand}/L = 0.25$, 0.5 and 0.75, a combination of scoop and wedge mechanism exhibits and the relevant caisson responses lie in between those for the clay and the sand cases.
Figure 3.6 Displacement contour (at rotation $\theta = 0.5^\circ$) in different soil profiles: (a) clay, (b) $H_{sand} = 2\text{m}$, (c) $H_{sand} = 4\text{m}$, (d) $H_{sand} = 6\text{m}$, (e) $H_{sand} = 8\text{m}$, (f) $H_{sand} = 12\text{m}$, (g) $H_{sand} = 12\text{m}$ and (h) sand.

In the foregoing section, the interaction via the skirt-clay interface is treated as fully rough with normal detachment prevented (i.e. fully bonded), which gives the upper limit estimation of the capacity when caisson is (partially) embedded in clay (i.e. single-layer clay and layered soil cases with $H_{sand}/L < 1$). If separation occurred between clay and caisson, this would reduce the moment resistance. However, the moment resistance for cases with $H_{sand}/L < 1$ should also remain between the results for single-layer sand and single-layer clay. For cases with $H_{sand}/L \geq 1$, the caisson is entirely embedded in the sand and thus the corresponding caisson behaviour is independent of the skirt-clay interface.
3.4 Concluding remarks

This section has presented a series of numerical analyses investigating the effect of soil layering on the monotonic loading response of a suction caisson. The considered soil profiles are single-layer sand, single-layer clay and sand over clay with thickness of sand 0.25, 0.5, 0.75, 1, 1.5 and 2 times the skirt length (i.e. $H_{\text{sand}}/L = 0.25, 0.5, 0.75, 1, 1.5$ and 2). It is observed that caisson responses in layered soils with $H_{\text{sand}}/L < 1$ (i.e. the bottom of the sand layer is above skirt tip) is bounded by the results for single-layer sand and single-layer clay, as expected. The ultimate moment capacity for these soil cases exhibits an approximately linear trend with the change of sand layer thickness. For layered soil cases with $H_{\text{sand}}/L \geq 1$ (i.e. caisson is fully embedded in the sand layer), the caisson response as well as the deformation mechanisms are similar to those in the single-layer sand case.

Hence, the following soil profiles are investigated in the remainder of this thesis:

- Sand over clay with $H_{\text{sand}}/L = 0.5, 1$ and 1.5, with the latter two selected to provide experimental evidence to substantiate the findings from this chapter.

- Single-layer sand and single-layer clay to improve understanding of caisson response (under cyclic loading) in conditions that provide bounds to that in sand over clay layered soils.

3.5 References


CHAPTER 4. THE RESPONSE OF SUCTION CAISSONS TO LONG-TERM LATERAL CYCLIC LOADING IN SINGLE-LAYER AND LAYERED SEABEDS

ABSTRACT: Suction caissons are being increasingly considered as an alternative foundation type to monopiles for offshore wind turbines. Single caisson foundations (or monopods) for offshore wind turbines are subjected to lateral cyclic loading from wind and waves acting on the structure. Recent studies have considered the response of suction caissons to such loading in sand, but have generally been limited to a few thousand cycles, whereas offshore wind turbines will generally experience millions of loading cycles over their lifetime. This paper presents the results from a programme of caisson tests in sand, clay and sand over clay seabed profiles, where each test involved about one million cycles of lateral load. The capacity and rotation response is shown to approach that measured in the sand seabed when the sand-clay interface is located at or beneath the caisson skirt tip. In contrast to previously published studies in sand, one-way cyclic loading is the most onerous loading symmetry for a layered seabed with a sand thickness equal to half the skirt length. However, the rotation for this seabed profile is essentially identical if the load is sustained or cyclic, provided that the cyclic loading remains one-way. Lateral cyclic loading was seen to increase caisson capacity by up to 30% – with a bias towards clay-dominated seabed profiles – and stiffness by up to 50%. Such stiffness increases needs to be considered when assessing the system dynamics for the offshore wind turbine, as demonstrated in the paper.

Keywords: Suction caisson; cyclic loading; offshore wind; physical modelling; layered soil.
4.1 Introduction

Offshore wind turbines are typically supported by monopile foundations, which are thin walled steel piles that can have a mass of up to 1,000 tonnes as in the Gode offshore wind farm (Schroeder et al., 2015). Such massive foundations require substantial installation vessels to handle and overboard the monopile. An alternative and potentially less expensive foundation is a suction caisson (see Figure 4.1), which is a large upturned bucket that is installed in the seabed by pumping water from the caisson interior. Suction caissons were first used as a foundation for a 3MW turbine at Frederikshavn, Denmark in 2002 (Ibsen and Brincker, 2004), and were further employed in offshore wind farm sites at Horns Rev 2, Denmark in 2009 (LeBlanc et al., 2009) and at Dogger Bank, UK in 2013 for Meteorological Masts.

![Figure 4.1 An offshore wind turbine supported by a suction caisson.](image-url)
Chapter 4. The response of suction caissons to long-term lateral cyclic loading in single-layer and layered seabeds

As shown by Figure 4.1, the loading on a suction caisson supporting an offshore wind turbine includes the vertical load due to the turbine weight and lateral cyclic loads due to waves and wind that act at and above sea level respectively, resulting in an overturning moment, $M$ at the caisson. An offshore wind turbine is expected to experience between $10^7$ and $10^8$ relatively low level loading cycles over its 25 year lifetime (Bhattacharya, 2014), which may result in a non-recoverable rotation of the turbine. This can be an issue for the serviceability design of a turbine (DNV, 2013), to the extent that the permanent accumulated rotation of the turbine during operation may need to be kept within very tight limits (e.g. 0.25° for the Thornton Bank offshore wind farm (Peire et al., 2009).

A second issue relates to the potential for cyclic loading to change the foundation stiffness, which influences the stiffness and thus natural frequency of the wind turbine as a whole. Resonance may result if the natural frequency of the system approaches a forcing frequency. These forcing frequencies are the rotor frequency, 1P, and the blade passing frequency, 3P (for a three-bladed turbine) (see Figure 4.2), with most designs opting for a ‘soft-stiff’ system, such that the system natural frequency lies between the 1P and 3P frequencies. Mitigating the risk of resonance requires a reliable prediction of foundation and system stiffness over the course of the design life of the wind turbine.

Attempts made to address these issues in sand include model tests and a limited number of reduced scale field tests. These tests typically involved less than 1,000 cycles of lateral load and focused on extreme loading conditions (Byrne and Houlsby, 2004; Houlsby et al., 2005; Houlsby et al., 2006; Kelly et al., 2006; Watson and Randolph, 2006). Model tests reported by Zhu et al. (2013), Cox et al. (2014) and Foglia et al. (2014) investigated the response of suction caissons in sand under more moderate cyclic load levels, but typically limited by 10,000 load cycles, three to four orders of magnitude fewer than the number of moderate load cycles that an offshore wind turbine is expected to experience over its lifetime. This limitation is the first motivation for this study.
Chapter 4. The response of suction caissons to long-term lateral cyclic loading in single-layer and layered seabeds

Figure 4.2 Typical loading frequencies of a three bladed 5MW NREL wind turbine (after Bhattacharya et al., 2013).

The seabeds in many sites considered for offshore wind development are layered. For example, in Dogger Bank, North Sea, the thickness of the superficial sand overlying stiff clay varies from several to tens of meters (Forewind 2011, 2013), such that the caisson is likely to be embedded in a ‘sand over clay’ seabed profile. The experimental evidence of suction caissons under lateral cyclic loading has been limited to ‘single-soil’ seabeds, such that the conclusions drawn from these studies are unlikely to be relevant to seabeds such as those at Dogger Bank. This limitation is the second motivation for this study. This paper extends the experience of suction caissons under lateral cyclic loading through a series of model tests involving long-term lateral cyclic loading of a suction caisson in sand, clay and sand over clay soil profiles.

4.2 Methodology, experimental set-up and programme

4.2.1 Modelling approach and scaling

The experimental approach adopted here is single-gravity model tests, as reduced scale field-testing was not considered feasible for the parameter base of interest in this study,
and centrifuge testing is generally not feasible for the $10^6$ loading cycles targeted in these experiments. Results from these long-term single-gravity tests coupled with results from centrifuge tests (involving much fewer loading cycles) reported in Zhu et al. (2018)\textsuperscript{1} provide a means of assessing the long-term response at field scale, accounting for effects due to installation, stress level and partial drainage in the sand. Conducting reduced scale single-gravity experiments requires careful consideration of scaling to ensure that the results are meaningful at field scale. Since this study is mainly focussed on the accumulated caisson rotation, consideration needs to be given to the soil stiffness. The soil stiffness at very small strain level (conventionally expressed by $G_0$ or $G_{\text{max}}$) is considered here, as the rotation of caisson during each cycle is minimal. The shear modulus of clay, $G_{\text{clay}}$, is linked to the undrained shear strength, $s_u$, through the rigidity index, $I_r$, according to:

$$G_{\text{clay}} = I_r s_u$$ \hspace{1cm} (4.1)

where $I_r$ can be evaluated using (Mayne, 2001)

$$I_r = \frac{\exp(5.96 - 0.04I_p)}{1 + \ln(1 + 0.04(OCR - 1)^{3.2})^{0.8}}$$ \hspace{1cm} (4.2)

in which $OCR$ is the over-consolidation ratio and $I_p$ is the plasticity index.

The shear modulus of sand, $G_{\text{sand}}$, is governed by the stress level (Iwasaki et al., 1978; Seed and Idriss, 1970):

$$G_{\text{sand}} = A p_a \left( \frac{\sigma'_v}{p_a} \right)^n$$ \hspace{1cm} (4.3)

where $\sigma'_v$ is the representative effective vertical stress, $p_a$ is atmospheric pressure and $A$ is a dimensionless constant increasing with relative density, $D_r$. The exponent, $n$, in

\textsuperscript{1} Chapter 5
Equation (4.3) varies between 0.435 at very small strains and 0.765 at very large strains (Wroth et al., 1979), but in consideration of suction caisson behaviour at laboratory and field scales, may be taken as 0.5 (LeBlanc et al., 2010; Kelly et al., 2006). The behaviour of layered seabeds is of interest here, and the approach taken has been to ensure that the ratio of clay to sand stiffness is equivalent at model and field scales, which from Equations (4.1) and (4.3) leads to

\[
\frac{s_{u,m}}{s_{u,f}} = I_{r,f} A_m \left( \frac{\sigma'_{v,m}}{\sigma'_{v,f}} \right)^n
\]  

(4.4)

where the subscripts ‘f’ and ‘m’ represent field- and model-scales respectively.

Equation (4.4) requires that \( A_m \) and \( A_f \) are selected a priori, which requires consideration of the appropriate relative density at model scale (given a targeted relative density at field scale). The accepted approach for modelling problems in sand is to ensure that friction angles in the laboratory and in the field are compatible, which will require the laboratory sample to be at a lower relative density than in the field (Kelly et al., 2006; LeBlanc et al., 2010; Zhu et al., 2013). This may be quantified using Bolton’s equations (Bolton, 1986) that capture the variation in the difference between the peak and critical state friction angles with mean effective stress level and relative density:

\[
\phi'_{\text{peak}} - \phi'_{\text{crit}} = m[D_r(10 - \ln p') - 1]
\]  

(4.5)

where \( \phi'_{\text{peak}} \) and \( \phi'_{\text{crit}} \) are the peak and critical state friction angles respectively, the parameter \( m \) is taken as 3 under triaxial conditions and 5 under plane strain conditions and \( p' \) is the mean effective stress at failure. A simplified scaling law between field- and model- scales may therefore be expressed as

\[
\frac{\phi'_{\text{peak,m}} - \phi'_{\text{crit,m}}}{\phi'_{\text{peak,f}} - \phi'_{\text{crit,f}}} = \frac{D_{r,m}(10 - \ln p'_m) - 1}{D_{r,f}(10 - \ln p'_f) - 1}
\]  

(4.6)
which provides a basis for selecting $A_m$ and $A_f$.

### 4.2.2 Soil properties and sample preparation

A layered seabed profile, with dense sand ($D_{r,f} = 80\%$) overlying stiff clay ($s_{u,f} = 80$ kPa), was targeted in the experiments, in an attempt to replicate typical North Sea seabeds (De Ruiter and Fox, 1975; Bond et al., 1997; BGS, 2002). As described later, the model caisson was based on a field scale caisson with a diameter, $D = 16$ m and a skirt length, $L = 8$ m, modelled at a reduced scale of 1:100 such that the effective stress ratio, $\sigma'_{v,m}/\sigma'_{v,f} = 1/100$. The mean effective stress at failure (assume $K_0 = 0.5$), $p'$, is considered at a representative depth equal to half the caisson skirt length, such that for a peak friction angle of 40˚, $p' = 120$ kPa and 1.2 kPa at field and model scale respectively. Equation (4.6) then requires that the relative density at model scale is $D_{r,m} \approx 42\%$, which together with $D_{r,f} = 80\%$ leads to $A_m/A_f = 0.6$ (Seed and Idriss, 1970). Rigidity indices at model and field scale are based on the OCR and plasticity index (Equation (4.2)). An OCR of 10 has been adopted at field scale based on North Sea conditions (Le et al., 2014), whereas the stress history in the model tests result in an OCR of 100. Adopting these values in Equation (4.2) together with $I_p = 34$ at model and field scale (see Table 4.1), result in $I_{r,m}/I_{r,f} = 1.85$. An undrained shear strength of the clay at model scale can then be calculated using Equation (4.4) (with $s_{u,f} = 80$ kPa, $A_m/A_f = 0.6$, $I_{r,m}/I_{r,f} = 1.85$, $\sigma'_{v,m}/\sigma'_{v,f} = 0.01$ and $n = 0.5$), giving $s_{u,m} = 9$ kPa.

The soil samples were prepared using a fine to medium sub-angular silica sand and kaolin clay with material properties as listed in Table 4.1. Layered soil samples were prepared as sand overlying clay, with sand layer thicknesses equal to 0.5, 1.0 and 1.5 times the skirt length ($H_{sand}/L = 0.5$, 1.0 and 1.5). Single-layer sand and clay samples were also prepared to bound the responses. The samples were created in steel circular sample containers with an internal diameter of 600 mm and an internal height of 400 mm.
Table 4.1 Properties of silica sand (Chow et al., 2015; Lee et al., 2013) and kaolin clay (Stewart, 1992; Richardson et al., 2009).

<table>
<thead>
<tr>
<th>Silica sand</th>
<th>Kaolin clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.65</td>
</tr>
<tr>
<td>Particle size, $d_{50}$ (mm)</td>
<td>0.19</td>
</tr>
<tr>
<td>Minimum dry density, $\rho_{\text{min}}$ (kg/m$^3$)</td>
<td>1461*</td>
</tr>
<tr>
<td>Maximum dry density, $\rho_{\text{max}}$ (kg/m$^3$)</td>
<td>1774+</td>
</tr>
<tr>
<td>Critical state friction angle, $\phi_{\text{crit}}$ (º)</td>
<td>30</td>
</tr>
<tr>
<td>Coefficient of consolidation, $c_v$ (m$^2$/year)</td>
<td>$&gt; 60,000$</td>
</tr>
</tbody>
</table>

* derived according to test standard ASTM D4253-00
+ derived according to test standard ASTM D4254-00

Clay samples were created by consolidating kaolin slurry at an initial moisture content of 120% (approximately twice the liquid limit, see Table 4.1) in a consolidation press to a final vertical effective stress of 110 kPa. The clay was then allowed to swell for one week. This duration was chosen on the basis of results from periodic T-bar tests that were conducted on a sample that was allowed to swell for two weeks; the strength reduction after one week was greater than 90% of the reduction after two weeks. Layered soil samples were prepared by scraping the required depth of clay to leave a level clay surface before adding the overlying sand layer. The sand layers (including the single-layer sand samples) were prepared by air pluviation before saturation with water from the base of the sand. Saturation was achieved using four plastic pipes located along the inner wall of the soil container with the pipe inverts positioned just above the clay layer (see Figure 4.3a) or at the base of the container for the single-layer sand samples. The targeted sample height was 325 mm, which was achieved to within 5 mm across the sixteen samples prepared for this study. A 50 mm layer of free water was maintained at the surface of the sample (using a float valve on the water supply line) over the course of the tests.
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Figure 4.3 Experimental arrangement at different stages: (a) before installation, (b) after installation, (c) release of installation actuator in preparation for cyclic loading and (d) cyclic loading.
Cone penetrometer tests (CPTs) were conducted to characterise each sample using a model cone penetrometer with a diameter, \( d = 10 \text{ mm} \). Figure 4.4 shows profiles of total cone resistance, \( q_c \), with normalised penetration depth, \( z/L \) (i.e. penetration depth, \( z \) normalised by the caisson skirt length, \( L \)) measured across each sample. The penetration velocity in the CPTs was \( v = 1 \text{ mm/s} \) such that the dimensionless velocity, \( v' = vd/c_v = 121 \) and \(< 0.01\) for the clay and sand respectively (where \( c_v \) is the vertical coefficient of consolidation, taken as \( c_v = 2.6 \text{ m}^2/\text{year} \) for the clay and \( c_v \) is at least \( 60,000 \text{ m}^2/\text{year} \) for the sand (Lee et al., 2013), see details in Table 4.1), such that the cone resistance in Figure 4.4 is primarily undrained in clay and drained in sand (Chung et al., 2006).

As shown by Figure 4.4 the development of \( q_c \) with depth is broadly consistent in the sand layers for each sample, and is lower than in the clay layers for \( z/L < 1 \). The \( q_c \) profiles in the clay layers resembles the undrained shear strength profiles determined from T-bar tests in the clay samples, as shown by the inset plot in Figure 4.4, where \( s_u \) was determined...
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from the measured T-bar resistance using the commonly adopted T-bar factor, \( N_{T\text{-bar}} = 10.5 \) (Low et al., 2010). Also shown on the \( s_u \) inset plot in Figure 4.4 is the classical expression (Ladd et al., 1977)

\[
s_u = \sigma'_v \left( \frac{s_u}{\sigma'_v} \right)_{nc} \text{OCR} \Lambda \tag{4.7}
\]

where \( \sigma'_v \) is the vertical effective stress determined using an effective unit weight, \( \gamma' = 7.1 \text{ kN/m}^3 \) as established from moisture content measurements made on post-testing sample cores, \( (s_u/\sigma'_v)_{nc} \) is the normally consolidated undrained shear strength ratio, and \( \Lambda \) is the plastic volumetric strain ratio (Schofield and Wroth, 1968). The best fit between Equation (4.7) and the T-bar measurements was obtained using \( (s_u/\sigma'_v)_{nc} = 0.18 \), which is typical for penetrometer derived measurements in kaolin clay (Chow et al., 2014; Fu et al., 2015; O’Beirne et al., 2017) and \( \Lambda = 0.83 \), which is within the typical range, \( \Lambda = 0.7 – 0.9 \) (Mitchell and Soga, 2005). Equation (4.7) leads to \( s_u \) in the range 5 to 10 kPa, which is comparable with the targeted \( s_u = 9 \text{ kPa} \) for these tests.

The relative density of the sand (in both the single-layer and layered samples) was determined from global measurements of sample (or layer) mass and volume after saturation. These measurements gave \( D_r = 39 \pm 1\% \ (\gamma' \approx 9.8 \text{ kN/m}^3) \) across fourteen samples, close to the targeted \( D_r = 42\% \).

4.2.3 Model caisson, instrumentation and experimental arrangement

The model caisson had a diameter, \( D = 160 \text{ mm} \), a skirt length, \( L = 80 \text{ mm} \) and a skirt thickness, \( t = 1 \text{ mm} \). This corresponds to a 100 mm skirt thickness at field scale, which although between two and five times higher than used in practice, provides a broadly consistent effective Young’s modulus, \( E_e = 64EI/\pi D^4 \) (where \( E \) is the Young’s modulus of the caisson material and \( I \) is the moment of inertia, which for a caisson, \( I = \pi[D^4 - (D - 2t)^4]/64; \) Randolph (1981)), noting that the model caisson skirt was
fabricated from aluminium rather than steel. The caisson skirt was anodised such that the surface roughness was $R_a = 1.62 \, \mu m$ as measured with a roughness profilometer. The caisson lid was made from polymethyl methacrylate (PMMA) for visual access and featured a vent as shown in Figure 4.5b. A 30 mm diameter aluminium ‘vertical loading shaft’, connected rigidly to the caisson lid, allowed the lateral loads to be applied eccentrically and also served as a means of monitoring the caisson rotation (as described later). This shaft was solid to ensure that shaft deflections due to the eccentrically applied load were negligible relative to the shaft displacements due to the caisson rotation. The combined weight of the caisson and shaft provides a vertical load, $V = 25 \, N$. The magnitude of the corresponding dimensionless group is $V/\gamma' D^3 = 0.62$ (in sand; $\gamma' = 9.8 \, kN/m^3$), which is within the range employed in existing studies; $V/\gamma' D^3 = 0.57$ (Zhu et al., 2013), $V/\gamma' D^3 = 0.69$ (Cox et al., 2014) and $V/\gamma' D^3 = 0.86$ (Foglia et al., 2014), and is within the $V/\gamma' D^3 = 0.09 – 0.91$ range reported in Foglia and Ibsen (2016) for field scale suction caissons supporting offshore wind turbines.

Figure 4.3 shows the general experimental arrangement at various stages in the tests. Before the caisson was installed it was connected via the vertical loading shaft to an actuator located above the sample container (Figure 4.3a). A vent in the caisson lid allowed for expulsion of air and water during penetration. The installation actuator (see Figure 4.3a) was mobilised to install the caisson, whilst maintaining verticality, whereas the loading actuator was used to apply the lateral loads to the vertical loading shaft (see Figure 4.3d). Caisson penetration resistance during installation was measured by a load cell (‘vertical load cell’ in Figure 4.3) located in series with the vertical loading shaft. The loading arm on the horizontal axis of the loading actuator connected to the vertical loading shaft on the caisson using a hinge and slot arrangement as shown in Figure 4.5. The hinge ensured no bending moment developed at the load application point, whilst the slot permitted free vertical movement of the hinge to ensure that the loading did not
inadvertently change the vertical displacement of the caisson, and also allowed the hinge to be connected to the vertical loading arm during installation of the caisson. Lateral loads were measured by a second load cell (‘horizontal load cell’ in Figure 4.3) located in series with the loading arm.

Figure 4.5 Photograph showing experimental arrangement during caisson installation: (a) overview and (b) close-up showing the detail of the load attachment point and the model caisson penetrating the soil sample.

Caisson rotation and displacement was determined from measurements made by four laser sensors (Lasers 1 and 2 oriented towards the vertical loading shaft and Lasers 3 and 4 oriented towards a target that was parallel with the caisson lid but remained above the water surface, see Figure 4.3).
4.2.4 Test programme and procedure

The lateral load $H$ was applied at an eccentricity, $e$ from the reference point, defined here as the centre of caisson at the lid invert ($RP$ in Figure 4.3d), which coincides with the mudline when the skirt is fully penetrated. The resulting overturning moment at the load reference point is $M = H \cdot e$, where the eccentricity $e$ was selected for these experiments as 3.5 times the caisson diameter to reflect the lateral loading from wind and waves on a 5 – 7.5 MW wind turbine located in 20 – 50 m of water. Vertical movement of the caisson over the course of the cyclic tests was less than 0.2% of the eccentricity, such that the associated moment, $M$, also varied by less than 0.2%.

The experimental programme encompasses both monotonic and cyclic tests in a range of layered and non-layered soil profiles, with variations in the magnitude and symmetry of the cyclic loading. Table 4.2 summarises the test programme in this study with different loading conditions and soil profiles. Monotonic tests were included in the test programme to provide the ‘first-loading’ stiffness and to quantify the ultimate moment capacity of the caisson in each soil profile, from which the magnitude of the load cycles was scaled. The cyclic loads were simplified as sine waves with the load magnitude and symmetry described using the parameters $\zeta_b$ and $\zeta_c$ respectively (LeBlanc et al., 2010)

$$\zeta_b = \frac{M_{\text{max}}}{M_{\text{ult}}}, \quad \zeta_c = \frac{M_{\text{min}}}{M_{\text{max}}}$$

(4.8)

where $M_{\text{min}}$ and $M_{\text{max}}$ are the minimum and maximum moments in a load cycle and $M_{\text{ult}}$ is the ultimate moment capacity. Hence, $\zeta_b$ can vary between 0 and 1, whereas $\zeta_c$ can vary between -1 and 1, with $\zeta_c \geq 0$ and $\zeta_c < 0$ representing one-way and two-way loading respectively (see Figure 4.6).
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Table 4.2 Monotonic and cyclic loading test program.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Test type</th>
<th>Soil profile</th>
<th>Soil profile</th>
<th>Number of cycles $N$</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>Monotonic</td>
<td>Sand</td>
<td>Sand</td>
<td>-</td>
</tr>
<tr>
<td>C1</td>
<td>Cyclic</td>
<td>0.4</td>
<td>0.1</td>
<td>1,204,998</td>
</tr>
<tr>
<td>P2</td>
<td>Monotonic</td>
<td>Clay</td>
<td>Clay</td>
<td>-</td>
</tr>
<tr>
<td>C2</td>
<td>Cyclic</td>
<td>0.4</td>
<td>0.1</td>
<td>1,239,149</td>
</tr>
<tr>
<td>P3</td>
<td>Monotonic</td>
<td>Sand over clay ($H_{sand}/L = 0.5$)</td>
<td>-</td>
<td>380,049</td>
</tr>
<tr>
<td>C3</td>
<td>Cyclic</td>
<td>0.4</td>
<td>-0.3</td>
<td>905,398</td>
</tr>
<tr>
<td>C4</td>
<td>Cyclic</td>
<td>0.4</td>
<td>-0.7</td>
<td>1,310,071</td>
</tr>
<tr>
<td>C5</td>
<td>Cyclic</td>
<td>0.4</td>
<td>0.5</td>
<td>150,549</td>
</tr>
<tr>
<td>C6</td>
<td>Cyclic</td>
<td>0.4</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>C7</td>
<td>Cyclic</td>
<td>0.2</td>
<td>0.1</td>
<td>1,017,664</td>
</tr>
<tr>
<td>C8</td>
<td>Cyclic</td>
<td>0.7</td>
<td>0.1</td>
<td>1,112,751</td>
</tr>
<tr>
<td>P4</td>
<td>Monotonic</td>
<td>Sand over clay ($H_{sand}/L = 1.0$)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>C10</td>
<td>Cyclic</td>
<td>0.4</td>
<td>0.1</td>
<td>1,200,049</td>
</tr>
<tr>
<td>P5</td>
<td>Monotonic</td>
<td>Sand over clay ($H_{sand}/L = 1.5$)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>C11</td>
<td>Cyclic</td>
<td>0.4</td>
<td>0.1</td>
<td>1,199,263</td>
</tr>
</tbody>
</table>

Figure 4.6 Definition of cyclic parameters $\zeta_b$ and $\zeta_c$ (after LeBlanc et al., 2010).

Each test involved the same installation procedure, with the full testing procedure involving:
• **Installation.** The penetration velocity for caisson installation was \( v = 0.05 \text{ mm/s} \), which gives a dimensionless velocity, \( v' (= v/t_{cv}) = 0.6 \) in clay and \( v' < 0.01 \) in sand, such that the installation response may be considered partially drained in clay and drained in sand (Finnie and Randolph, 1994; Chung et al., 2006; Yi et al., 2012). Installation was deemed complete when the caisson penetration resistance increased markedly, signifying contact of the caisson lid with the surface of the soil sample. The caisson vent was then sealed and the installation actuator was removed (with minimal interference) to prepare for the lateral load test (Figure 4.3c).

• **Monotonic loading.** The monotonic tests were conducted in displacement control, with a displacement rate at the hinge of 0.05 mm/s. Each monotonic test was maintained until the caisson had rotated through about 2°, which was more than sufficient for failure. This involved a loading duration of \( t = 400 \text{ s} \), with a corresponding dimensionless time factor, \( T = c_v t/D^2 < 0.0013 \) for clay and > 10 for sand, such that the response is expected to be undrained in the clay and drained in the sand.

• **Cyclic loading.** The cyclic tests were conducted in (lateral) load control, such that the moment, \( M \), at the load reference point \( (M = H \cdot e) \) was also controlled. The caisson was firstly preloaded to the maximum cyclic load, \( M_{\text{max}} \), at a rate of 0.05 Nm/s, which resulted in a \( M - \theta \) response (up to \( M_{\text{max}} \)) that was consistent between the preloading stage in the cyclic tests and the monotonic tests. The cyclic loads were then applied at a frequency of 1 Hz. This frequency was selected to balance the duration of a cyclic test (approximately two weeks per test for the targeted one million load cycles) with high-quality load control (noting that the actuation system uses the load cell signal in a feedback loop). This frequency results in a fully drained response in the sand (due to the high permeability, see Table 4.1).
However, in the clay the 1 Hz frequency results in an undrained response in the clay over one cycle, but a partially drained response over many cycles, which is considered relevant for field conditions.

- **Post-cyclic monotonic loading.** After cyclic loading the lateral load on the caisson was reduced to zero before loading the caisson monotonically (in the same way as for the initial monotonic tests) to quantify post-cyclic stiffness and capacity.

The test programme summarised in Table 4.2 is comprised of 16 caisson tests (in sixteen separate soil samples), of which 5 were monotonic and 11 were cyclic with a post-cyclic monotonic stage. A baseline load case with \( \zeta_b = 0.4 \) and \( \zeta_c = 0.1 \) was repeated for each soil profile, representing one-way loading for a fatigue limit state (LeBlanc et al., 2010). Variations in \( \zeta_b \) and \( \zeta_c \) were considered for the layered soil profile with \( H_{sand}/L = 0.5 \), prompted by observations from the monotonic tests for each soil profile. One million load cycles was achieved in most tests, with the exception of tests C3 and C6 that ended prematurely (\( N = 380,049 \) and \( N = 150,549 \) respectively) due to an issue with the load control. Test C7 is a sustained loading case with \( \zeta_b = 0.4 \) and \( \zeta_c = 1 \) (i.e. \( M = M_{max} = 0.4M_{ult} \)), with a load duration of six days (equal to that required for \( N \approx 500,000 \)). This test was included to examine the role of consolidation in the clay layer under sustained loading relative to that under cyclic loading.

### 4.3 Results and discussion

Although the use of dimensionless groups forms a sound basis for considering results from small scale model tests at field scale (Kelly et al., 2006; Byrne, 2014), it is unclear what the appropriate dimensionless groups should be for layered soil profiles. For this reason the results are presented in model scale, with corresponding centrifuge experiments (Zhu et al., 2018) forming a basis for considering the likely response at field scale.
4.3.1 Ultimate moment capacity

Figure 4.7 presents the caisson moment-rotation response measured in the monotonic tests. The response is initially stiff, before softening to an approximately constant moment capacity (for the single-layer sand samples and the samples with $H_{sand}/L \geq 1$), or to a moment capacity that gradually increases with increasing rotation (for the single-layer clay sample and the sample with $H_{sand}/L = 0.5$). Consequently, the ultimate moment capacity has been quantified using the simple graphical construction method shown on Figure 4.7. The caisson moment-rotation responses are essentially coincident for the single-layer sand sample and the samples with $H_{sand}/L \geq 1$, with ultimate moment capacity, $M_{ult} \approx 2$ Nm. The sample with $H_{sand}/L = 0.5$ and the single-layer clay sample provide higher ultimate moment capacities; $M_{ult} \approx 4$ Nm and 6 Nm, respectively. Figure 4.7 indicates that moment capacity is mainly derived from shearing of the soil above skirt tip level ($z/L \leq 1$), and is broadly consistent with the magnitude of $q_c$ at $z/L \leq 1$ (see Figure 4.4).

Figure 4.7 Monotonic test results for each soil profile.
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### 4.3.2 Accumulated rotation

Example results from the cyclic tests are provided in Figure 4.8 for the single-layer sand (Figure 4.8a) and clay (Figure 4.8b) profiles, and for a cyclic load magnitude, \( \zeta_b = 0.4 \) and load symmetry, \( \zeta_c = 0.1 \). Evidently rotation increases during cycling, with the rotation (after \( N = 1 \times 10^6 \)) in the test in the single-layer clay being approximately half that in the test in the single-layer sand. The accumulation of rotation with cycle number is more evident in Figure 4.10, which quantifies the accumulated rotation as \( \Delta \theta(N)/\theta_0 \), where \( \Delta \theta(N) = \theta_N - \theta_0 \), and \( \theta_0 \) and \( \theta_N \) are the maximum rotation during preloading to \( M_{\text{max}} \) and in cycle number \( N \), respectively (see Figure 4.9).
Figure 4.8 Typical cyclic moment-rotation responses during cyclic loading (only data for cycles labelled $N$ shown for clarity): (a) sand and (b) clay.

Figure 4.9 Definition of accumulated rotation and stiffness (after LeBlanc et al., 2010).
Figure 4.10a presents the variation of normalised accumulated rotation $\Delta \theta(N)/\theta_0$ with cycle number $N$ for $\zeta_b = 0.4$, $\zeta_c = 0.1$ in each soil profile $\Delta \theta(N)/\theta_0$ is highest for the single-layer sand profile and lowest for the single-layer clay profile. Similar to observations from the monotonic tests, the long-term cyclic response (for this loading scenario) appears to be governed by the sand thickness relative to the skirt length. For the layered soil profiles with $H_{\text{sand}}/L \geq 1$, $\Delta \theta(N)/\theta_0$ is similar to that measured in the single-layer sand profile, whereas $\Delta \theta(N)/\theta_0$ in the soil profile with $H_{\text{sand}}/L = 0.5$ is close to that measured in the single-layer clay profile, albeit that for the latter rotation appears to stabilise at about $N = 300,000$. The corresponding dimensionless time, $T = c_v t / D^2$ (where $t$ is the elapsed cyclic load duration) is also shown on Figure 4.10. $T = 1.0$ at cycle number, $N = 300,000$. There are no existing theoretical or numerical solutions to assess the expected degree of consolidation for this boundary value problem. However, consideration of other consolidation solutions suggests that $T = 1.0$ would be associated with at least 90% consolidation (Gourvenec and Randolph, 2010; Feng and Gourvenec, 2015), with the associated increase in soil strength. Effects of cyclic loading conditions on consolidation are evident in Figure 4.10b and c and also in the context of the post-cyclic response (as discussed later).

The effect of loading symmetry is examined on Figure 4.10b, which shows the development of normalised rotation, $\Delta \theta(N)/\theta_0$, with cycle number, $N$, for the layered soil profile with $H_{\text{sand}}/L = 0.5$, a load magnitude, $\zeta_b = 0.4$ and load symmetry in the range $\zeta_c = -0.7$ to 1.0. $\zeta_c = 1.0$ represents the case where the load is not cyclic but sustained at $M = M_{\text{max}}$. $\Delta \theta(N)/\theta_0$ is higher under one-way loading ($\zeta_c > 0$) than under two-way loading ($\zeta_c < 0$) for a layered soil profile with $H_{\text{sand}}/L = 0.5$. Although the effect of load symmetry is not systematically investigated for other soil profiles in this study, Zhu et al. (2013) made the opposing observation for sand, with two-way loading leading to higher rotation than one-way loading. The close agreement between the rotation response for the
$H_{\text{sand}}/L = 0.5$ sample and the single-layer clay sample on Figure 4.10a under one-way loading ($\zeta_c = 0.1$) suggests that when the skirts penetrate the clay (by at least $0.5L$) the rotation response is governed by the clay. This echoes the observation on Figure 4.10b that (for $H_{\text{sand}}/L = 0.5$) the rotation response for $\zeta_c = 1.0$ is highly consistent with that for $\zeta_c = 0.5$ and 0.1, with no discernible difference in the response under pure sustained loading ($\zeta_c = 1.0$) than under one-way cyclic loading ($\zeta_c = 0.5$ and 0.1).

Figure 4.10c also provides results from tests conducted in the layered soil profile with $H_{\text{sand}}/L = 0.5$, but where the loading was one-way with $\zeta_c = 0.1$ and the cyclic load magnitude varied between $\zeta_b = 0.2$ and $\zeta_b = 0.7$. Although there is no obvious effect of $\zeta_b$ on the normalised rotation, $\Delta \theta(N)/\theta_0$, the inset figure in Figure 4.10c confirms the expected increase in absolute rotation $\theta_N$ with increasing $\zeta_b$. 

(a)
Figure 4.10 Accumulated rotation with number of loading cycles: (a) different soil profiles ($\zeta_b = 0.4$, $\zeta_c = 0.1$); (b) $\zeta_b = 0.4$ (soil profile $H_{\text{sand}}/L = 0.5$); (c) $\zeta_c = 0.1$ (soil profile $H_{\text{sand}}/L = 0.5$).
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Figure 4.10 also includes empirical fits to each dataset. These fits describe the evolution of normalised rotation, $\Delta \theta(N)/\theta_0$ with cycle number, $N$, using a power law (Cox et al., 2014; Zhu et al., 2013)

$$\frac{\Delta \theta(N)}{\theta_0} = \beta \times N^\alpha$$  (4.9)

where $\alpha$ and $\beta$ are dimensionless variables. $\beta$ is used here rather than $T$ adopted in previous studies (e.g. LeBlanc et al. 2010 and Zhu et al. 2013) to avoid confusion with dimensionless time, $T$. Regression analyses resulted in $\alpha = 0.28$ for all test data, noting that in tests C2 and C5 the rotation stabilised after approximately 300,000 cycles, and this stabilised behaviour was not considered in the regression analyses. $\alpha = 0.28$ is broadly consistent with $\alpha = 0.30$ as established by Cox et al. (2014) from centrifuge tests on suction caissons in dry dense sand, and close to $\alpha = 0.31$ reported from single-gravity experiments on monopiles in dry loose sand that were also subjected to lateral cyclic loading (LeBlanc et al., 2010; Abadie et al., 2015). The value is however, lower than $\alpha = 0.39$ reported by Zhu et al. (2013) from single-gravity tests on lateral cyclic loading of suction caissons in loose dry silty sand but higher than $\alpha = 0.18$ from single-gravity tests in dense sand (Foglia et al., 2014).

The magnitude of $\beta$ in Equation (4.9) for cyclic loading given by $\zeta_b = 0.4$ and $\zeta_c = 0.1$ is summarised in Figure 4.11a for the different soil profiles, where the variation in sand thickness is quantified by $H_{sand}/L$, with a single-layer sand profile represented by $H_{sand}/L = 3$. Consistent with Figure 4.10a, $\beta$ (and hence the accumulated rotation) is lowest at 0.05 for $H_{sand}/L = 0$ (i.e. a single-layer clay profile) and gradually increases towards 0.15 in single-layer sand, with layered profiles lying between these limits.

Figure 4.11b shows the variation in $\beta$ with cyclic load symmetry, $\zeta_c$, for a layered soil profile given by $H_{sand}/L = 0.5$, in sand with $\zeta_c = 0.1$ (C1), and data reported by Zhu et al.
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(2013) for sand with $\zeta_b = 0.5$. Figure 4.11b indicates that the current result for sand is in good agreement with the Zhu et al. (2013) data, which indicate that highest rotation in sand is expected for $\zeta_c \approx -0.7$. In contrast to the Zhu et al. (2013) observation for sand, the current study indicates that in a layered soil profile given by $H_{\text{sand}}/L = 0.5$ the highest rotation is observed when $\zeta_c > 0$.

Normalised rotation – but not absolute rotation – is independent of the cyclic load magnitude, $\zeta_b$, for a layered soil profile with $H_{\text{sand}}/L = 0.5$ (Figure 4.9c). This is further supported by Figure 4.11c, which compares $\beta$ for $H_{\text{sand}}/L = 0.5$ at different $\zeta_b$ values together with the previously reported and current data for sand. The sand data indicate that $\beta$ increases linearly with increasing $\zeta_b$ in Zhu et al. (2013). The data for the layered soil profile indicate that $\beta$ remains constant over the range $\zeta_b = 0.2 - 0.7$. On this basis, the following simple expression (also shown on Figure 4.11b) may be used to quantify $\beta$ for a layered soil profile with $H_{\text{sand}}/L = 0.5$:

$$\beta = \begin{cases} 0.07(\zeta_c + 1) & \text{for } \zeta_c < 0 \\ 0.07 & \text{for } \zeta_c \geq 0 \end{cases}$$

(4.10)

The normalised caisson rotations in Figure 4.10 are in the range 0.3 to 9.0. The corresponding absolute rotations are between 0.03 and 0.45°, which although on the limit of (or may possibly exceed) a serviceability limit state, are highly dependent upon the initial stiffness, which is expected to be different at laboratory and field scales. Equation (4.9) can be used to forecast the expected caisson rotation at field scale, given an assumed initial rotational stiffness. Adopting lower and upper bounds on $\beta$ of 0.05 and 0.15 respectively (for the case of $\zeta_b = 0.4$ and $\zeta_c = 0.1$, see Figure 4.11a), results in a rotation that is between 2.4 and 7.2 times the initial rotation (to $M_{\text{max}}$) after one million cycles. Appropriate values of initial stiffness, including effects from suction installation and possible partial drainage is explored through centrifuge experiments (Zhu et al., 2018).
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(a)

(b)
Figure 4.11 Variation of undrained parameter $\beta$ with (a) soil profiles, (b) $\zeta_c$ and (c) $\zeta_b$.

### 4.3.3 Unloading stiffness

Figure 4.12a presents the variation in normalised unloading stiffness, $k_N/k_1$, with cycle number, $N$, where $k_1$ and $k_N$ are the unloading stiffnesses in the first cycle and cycle number $N$, respectively (see Figure 4.9). Unloading stiffness is seen to increase, although the apparent scatter in the data – due to very small changes in rotation in each cycle – has a slight pollutant effect. Figure 4.12a shows that the increases are more immediate in the soil profiles dominated by sand ($H_{\text{sand}}/L > 1$), which reflects densification, with later increases in the clay dominated profiles attributed to consolidation. Figure 4.12b and c show that for $H_{\text{sand}}/L = 0.5$ the increase in stiffness is independent of load symmetry, but is higher for increasing cyclic load magnitude. The long-term stiffness appears to stabilise, following densification of the sand and consolidation of the clay, with increases by typically 20 - 50%. Such changes in the foundation stiffness need to be considered...
carefully as this will alter the natural frequency of the system. To illustrate this point, when the foundation stiffness calculated by Arany et al. (2016) for ten offshore wind farms is allowed to increase by 50%, the system natural frequency (calculated using the Arany et al. (2016) approach) increases by between 1.5 and 3.5%. However, the increase in the system natural frequency would be up to 22% if the initial foundation stiffness was an order of magnitude lower (noting that the seabeds in these ten cases were typically dense sands and stiff clays).
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Figure 4.12 Unloading stiffness with number of loading cycles: (a) different soil profiles ($\zeta_b = 0.4, \zeta_c = 0.1$); (b) $\zeta_b = 0.4$ ($H_{\text{sand}}/L = 0.5$); (c) $\zeta_c = 0.1$ ($H_{\text{sand}}/L = 0.5$).
4.3.4 Post-cyclic behaviour

A monotonic test was performed after each cyclic test to assess the effect of cycling on capacity and stiffness. Figure 4.13 compares the monotonic moment-rotation responses measured in tests with and without pre-cycling. In Figure 4.13 the rotation in the post-cyclic response is that relative to that at the end of cycling to facilitate a more direct comparison with the initial monotonic response. It is clear from Figure 4.13 that lateral cyclic loading generally increases capacity and stiffness, with the stiffness magnitude in the post-cyclic test being consistent with that measured in the last few cycles of the cyclic tests. The increase in ultimate moment capacity – quantified using the simple graphical construction method described earlier – is up to approximately 30% in the clay dominated samples \( (H_{\text{sand}}/L = 0 \text{ and } 0.5) \) and approximately 10% in the sand dominated samples \( (H_{\text{sand}}/L \geq 1) \). However, in Figure 4.13c the post-cyclic response in the single-layer clay exhibits a peak moment capacity not observed in the monotonic tests conducted before cyclic loading. If \( M_{\text{ult}} \) is taken as this peak moment capacity, the increase in moment capacity is 50%. The higher \( M_{\text{ult}} \) in the clay dominated samples is attributed to cyclic-loading induced consolidation. The effect of cyclic loading on moment capacity has not been observed in tests on sand – neither the single-layer sand tests reported here or in previous studies (e.g. Abadie, 2015). As discussed earlier, consolidation plays a beneficial role in stabilising caisson rotation and the unloading stiffness, but may also allow for design efficiencies with respect to moment capacity.
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(a)

(b)
Figure 4.13 Monotonic responses after cyclic loading: (a) single-layer sand, $H_{\text{sand}}/L = 1.0$ and $1.5$, (b) $H_{\text{sand}}/L = 0.5$ and (c) single-layer clay.

### 4.4 Concluding remarks

This Chapter has considered the response of a suction caisson to the type of lateral cyclic loading experienced by an offshore wind turbine in the layered seabeds prevalent in many of the existing and proposed wind farm developments. The focus in the experiments reported here is the effect of lateral cyclic loading on three key aspects: caisson rotation (as the serviceability limit state design is strict, e.g. maximum rotations of up to 0.5°; (DNV, 2013)); foundation stiffness (as stiffness changes need to be considered in the fatigue limit state, with the potential that changes in foundation stiffness push the natural frequency of the system towards the dominating forcing frequencies); and lateral capacity (as this needs to be considered in the ultimate limit state design). Single-gravity model tests were conducted in sand, clay and sand over clay seabeds, with each test typically
involving one million cycles of lateral load. Caisson rotation in layered seabeds is similar to that in sand when the sand thickness is at least equal to the caisson skirt, and reduces to that in clay as the sand thickness reduces. The lower rotation measured in the clay seabeds and where the caisson skirt penetrates the lower clay layer is attributed to consolidation induced strength increases, which eventually stabilised the rotation in the clay seabed. The effect of load symmetry was investigated for a layered seabed, where the upper sand layer extended to half the caisson skirt length. These tests revealed that the most onerous loading was one-way loading, unlike previous studies in sand that show partial two-way loading to be the most onerous. The rotation response in the layered seabed was consistent provided there was no reversal in the load direction, and the cycling the load did not appear to cause more or less rotation that when the load was sustained. The evolution of caisson rotation with cycle number can be described for each soil profile and loading scenario using a simple power relationship, which forecasts a caisson rotation (after one million cycles of moderate loads) that is 2.4 and 7.2 times the rotation in the first cycle for a clay and sand seabed respectively – sand over clay layered seabeds fall within these limits. These ranges can assist in assessing the likely rotation of a turbine over its lifetime given an appropriate initial stiffness.

Cyclic loading has the effect of increasing the capacity by up to 30% and foundation stiffness by up to 50%. The latter is expected to alter the natural frequency of the wind turbine, particularly when the initial system natural frequency is much smaller than the fixed base natural frequency. These changes are attributed to densification of the sand and consolidation induced strength increases in the clay.

The conclusions reached in this Chapter relate to unidirectional cyclic loading and the validity of these conclusions need to be checked when the load direction varies according to the changing direction of the wind and waves.
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4.5 References


Chapter 4. The response of suction caissons to long-term lateral cyclic loading in single-layer and layered seabeds


CHAPTER 5. SUCTION CAISSON FOUNDATIONS FOR OFFSHORE WIND ENERGY: INSTALLATION AND CYCLIC RESPONSE IN SAND AND SAND OVER CLAY

ABSTRACT: The monopod suction caisson is an alternative foundation concept to the commonly used monopile for offshore wind turbines. This paper considers experimental data on the installation and long-term response of a suction caisson in sand and sand over clay seabeds. The tests were conducted in a centrifuge – such that the effects of soil stress level and installation method in model testing are quantified, which lends confidence to predictions of the response at prototype scale. Suction installation through dense sand was shown to be achievable, even in the presence of an underlying clay layer, which limits the flow paths for the required seepage induced reduction of effective stresses around the skirt tip. Existing methods for the prediction of required suction and resulting caisson penetration rate, which were modified for the layered soil, are shown to capture the measured response quite well. The program included tests featuring aspects that have commonly been adopted in laboratory testing but do not reflect field conditions, such as jacked installation and full drainage in the sand, to link published data to this contribution. The centrifuge tests focused on sand over clay seabeds, and considered variations in the cyclic load magnitude and symmetry. One-way cyclic loading in sand over clay is seen to result in higher rotation than two-way loading, which contrasts with findings from previous studies in sand. Excess pore pressure dissipation in the clay layer leads to strength increases that stabilise caisson rotation and increase stiffness. The centrifuge tests are considered collectively with equivalent single-gravity tests to form a basis for predicting the long-term response of a monopod suction caisson. Example design scenarios are explored, which illustrate implications of the findings for field conditions.
5.1 Introduction

Foundations account for around 30% of the cost of an offshore wind farm development (Pelc and Fujita, 2002; Junginger et al., 2004; Snyder and Kaiser, 2009). One of the foundation concepts identified as having the highest potential to lead to cost savings is the monopod suction caisson (Carbon Trust, 2012) (Figure 5.1a) which is essentially a large upturned steel bucket, with a skirt length to diameter ratio (aspect ratio) that is typically in the range 0.5 – 1.0 for sandy seabeds.
The suction caisson is initially lowered to the seabed and penetrates the soil through self-weight while water is allowed to escape through a vent in the lid. This vent is then closed and water is pumped out of the caisson chamber, creating a differential pressure (or suction) across the top lid of the caisson relative to the ambient pressure. This drives the caisson into the seabed. In sand, the applied suction will induce a seepage flow from outside to inside, which significantly reduces the penetration resistance at the skirt tip through reduction of effective stresses (Tjelta, 1994). An illustration of suction installation in sand is presented in Figure 5.1b. No significant seepage flow will occur in clay sediments due to the relatively low permeability.

Once the offshore wind turbine (OWT) is installed, suction caissons are exposed to cyclic loading from the ocean environment (wind and waves) acting on the superstructure. These loads act laterally at a distance above the mudline and induce an overturning moment at the foundation level (Figure 5.1a), which may result in unrecoverable rotation of the turbine. Strict limitations are placed on accumulated rotation of OWTS, owing to the performance sensitivity of the turbine nacelle to non-verticality (Bhattacharya, 2014), e.g. DNV (2016) note a rotation limit of 0.5°. Furthermore, OWTS are designed as a ‘soft-
stiff system, with the targeted system natural frequency falling in the narrow range between the forcing frequencies of the rotor (1P) and the blade passing frequency (3P for a three-blade turbine), respectively. However, the system natural frequency is a function of the foundation stiffness, which may change as a result of long-term cyclic loading. The evolution of stiffness hence needs to be understood to ensure that the system eigenfrequencies are avoided.

Research on cyclic loading of suction caissons has focused on either sand (Byrne and Houlsby, 2004; Houlsby et al., 2006; Kelly et al., 2006; Senders, 2008; Zhu et al., 2013; Cox et al., 2014; Foglia et al., 2014) or clay (Houlsby et al., 2005; Villalobos et al., 2010), with limited data available for layered soils (Zhu et al., 2018). However, more than 80% of European wind power has been installed in the North Sea (EWEA, 2016), where soil profiles often consist of a surface sand layer over stiff clay (Bond et al., 1997). Zhu et al. (2018) provide the only publicly available database to date of caisson response under lateral cyclic loading in sand over clay. These experiments were performed as scaled model tests at single gravity; the caisson was installed by jacking as in most experiments, showing trends of rotation accumulation that were consistent across the database and with published research on this topic. Furthermore, much of the experimental work in sand has ensured drained conditions, either by conducting the tests in dry sand (e.g. Zhu et al., 2013 and Cox et al., 2014), or by selecting a loading frequency and pore fluid permeability that lead to drained conditions. Although effects of the pore fluid response have been investigated in centrifuge tests involving vertical cyclic loading (Senders, 2008; Bienen et al., 2018b), there is no corresponding experimental evidence.
for lateral cyclic loading. There is therefore, a pressing need to close the knowledge gap on the long-term response of suction caissons in layered soils to lateral cyclic loading.

This paper addresses this with results from a series of centrifuge tests providing the experimental evidence to explore the following:

- effect of the underlying clay layer on suction installation through dense sand;
- effect caused by suction installation on the subsequent response under lateral cyclic loading in sand (closing gaps in existing knowledge) and sand over clay;
- effect of the soil stress level (model testing in the centrifuge compared with testing at single gravity);
- effect of the resulting drainage regime in sand on stiffness and rotation during lateral cyclic loading;
- effect of a clay layer underlying sand on foundation performance under lateral cyclic loading (following suction installation as in the prototype);
- effect of cyclic load magnitude and symmetry on stiffness and rotation in sand over clay.

The discussion of the centrifuge test results is organised in the sequence of installation response, capacity, accumulated rotation, cyclic stiffness and post-cyclic behaviour.

5.2 Centrifuge testing details

The centrifuge tests were carried out at 100g using the beam centrifuge at the University of Western Australia (UWA) (Randolph et al., 1991).

5.2.1 Apparatus and instrumentation

Figure 5.2 presents the apparatus used in the centrifuge tests. The model caisson has a diameter, $D = 80$ mm, skirt length, $L = 40$ mm and skirt thickness, $t = 0.5$ mm. This corresponds to a prototype caisson with $D = 8$ m, $L = 4$ m and $t = 50$ mm. The skirt
thickness, although slightly larger than used in practice, provides a broadly consistent effective Young’s modulus, \( E_e = \frac{64EI}{\pi D^4} \) (where \( E \) is the Young’s modulus of the caisson material and \( I \) is the moment of inertia which for a caisson, \( I = \pi[D^4 - (D - 2t)^4]/64; \) Randolph (1981)), noting that the model caisson was fabricated from aluminium rather than steel. The surface roughness of the skirt was \( R_a = 0.96 \mu m \) as measured with a roughness profilometer. The model caisson was instrumented with three pressure transducers: a pair of total and pore pressure transducers (TPT and PPT) located on the caisson lid invert and a further TPT at the top of the caisson lid (see Figure 5.2a).
The model caisson was manipulated using a dual axis (vertical and horizontal) electrical actuator. The vertical axis was used to install the caisson and to maintain a constant vertical load on the caisson, whereas the horizontal axis was used to apply the lateral loading. The caisson was connected to the vertical axis of the actuator via an aluminium bending arm that was instrumented with full Wheatstone bridge strain gauges at 75 and 165 mm above the invert of the caisson lid. The lateral load was calculated in real time and controlled from the bending measurements, assuming linear moment distribution between the strain gauges. As shown in Figure 5.2, a hinge and axial load cell were located between the top of the bending arm and the connection to the vertical axis of the actuator. The load cell was used to measure (and control) the vertical load during the tests. A servomotor-controlled keyway ensured verticality of the caisson during installation (with the hinge locked) and free rotation (i.e. zero bending moment) at the lateral load application point (with the hinge unlocked).
Suction installation of the caisson was achieved using a syringe pump (House, 2002) that extracted fluid from the caisson interior, creating the pressure differential across the caisson lid. The installation and loading process required that the caisson was vented during self-weight installation, hydraulically connected to the syringe pump during suction installation and sealed during loading. This was achieved using a three-way valve located on the caisson lid, which was controlled using a motor that switched the ‘open port’ on the valve using the motor and wire arrangement depicted on Figure 5.2.

Caisson displacement was measured using three LDTs (Linear Displacement Transducers) that were positioned between an independent fixed reference beam and plates attached to the caisson lid. Vertical and lateral displacement as well as rotation of the caisson were calculated from these measurements.

### 5.2.2 Sample preparation and characterisation

The tests were performed in sand samples and in samples with clay overlain by sand, in which the height of the sand layer, $H_{\text{sand}}$, was half the caisson skirt length, $L$ (i.e. $H_{\text{sand}}/L = 0.5$). This layered profile was selected on the basis of results from the single-gravity tests (Zhu et al., 2018) showing that caisson behaviour in sand over clay at $H_{\text{sand}}/L = 0.5$ differs from that in sand and in clay.

The soil strengths were targeted to represent North Sea conditions, with a relative density, $D_r = 80\%$ for the sand and an undrained shear strength, $s_u = 80$ kPa for the clay (De Ruiter and Fox, 1975; Bond et al., 1997; BGS, 2002). The soil samples were prepared using commercially available silica sand and kaolin clay. Material properties for the sand and clay are provided in Table 5.1. Sand samples were prepared by dry pluviation, followed by saturation from the sample base. Two sand samples were prepared; one saturated with water (viscosity, $\nu_w = 1$ cSt) and one saturated with water containing 2.3% Methocel F450 (DOW, 2002), a cellulose ether to obtain a fluid viscosity, $\nu_c = 700$ cSt. The high viscosity
pore fluid was adopted to generate partially drained conditions in the sand as considered relevant for field conditions (Bienen et al., 2018a, b). The sand over clay sample comprised 20 mm of sand overlying overconsolidated clay. The clay layer was consolidated from kaolin slurry with a water content of 120% (i.e. approximately twice the liquid limit, see Table 5.1) in a consolidation press to a final vertical effective stress of 700 kPa. Excess water was removed from the clay surface before preparing the sand layer, which was dry pluviated onto the clay and then saturated with the cellulose ether pore fluid using saturation pipes (at each corner of the sample container) that extended to the base of the sand. The sand over clay sample was spun in the centrifuge at 100g for four days before testing to allow for reconsolidation of the clay. All three samples had a final height of 150 ± 3 mm and were covered by about 60 mm of free fluid (using 2 mm of mineral oil on the surface to prevent evaporation) during testing. The relative density of the sand was $D_r = 83 \pm 1\%$ across the three samples, as determined from global measurements of soil mass and volume, which gave an effective unit weight $\gamma' = 10.7$ kN/m$^3$. The average effective unit weight of the clay was determined from moisture content measurements made on post-testing cores, which gave an average $\gamma' = 8.5$ kN/m$^3$.

Table 5.1 Engineering properties of silica sand (after Chow et al., 2015) and kaolin clay (after Stewart, 1992; Richardson et al., 2009).

<table>
<thead>
<tr>
<th>Silica sand</th>
<th>Kaolin clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.65</td>
</tr>
<tr>
<td>Mean particle size, $d_{50}$ (mm)</td>
<td>0.19</td>
</tr>
<tr>
<td>Minimum dry density, $\rho_{\text{min}}$ (kg/m$^3$)</td>
<td>1461‡</td>
</tr>
<tr>
<td>Maximum dry density, $\rho_{\text{max}}$ (kg/m$^3$)</td>
<td>1774*</td>
</tr>
<tr>
<td>Critical state friction angle, $\varphi'_{cv}$ (º)</td>
<td>30</td>
</tr>
<tr>
<td>Coefficient of consolidation, $c_v$ (m$^2$/year) at $D_r = 83%$</td>
<td>16,000*</td>
</tr>
</tbody>
</table>

* when saturated with 700 cSt cellulose ether pore fluid
† when saturated with 1 cSt water pore fluid
‡ derived according to test standard ASTM D4253-00
§ derived according to test standard ASTM D4254-00
Cone penetrometer tests (CPTs) were conducted to characterise each sample and to provide a basis for comparing samples. These CPT tests and ball penetrometer tests (as described later) were performed at 100g. The model cone penetrometer had a diameter, \(d = 7\,\text{mm}\), and was penetrated at a velocity, \(v = 1\,\text{mm/s}\). This gives a dimensionless velocity, \(v' = \frac{vd}{c_v} = 30\) in the clay, \(v' = 0.014\) in the sand saturated with 700 cSt pore fluid and \(v' = 2 \times 10^{-5}\) in the sand saturated with water. The coefficient of consolidation used in the calculation of \(v'\) is taken as \(c_v = 7.5\,\text{m}^2/\text{year}\) for the clay, \(c_v = 16,000\,\text{m}^2/\text{year}\) for the sand saturated with 700 cSt pore fluid pore fluid and \(c_v = 1.1 \times 10^{7}\,\text{m}^2/\text{year}\) for the sand saturated with water. Hence, the cone resistance is expected to be undrained in the clay, close to drained in the sand with high viscosity pore fluid, and drained in the (low viscosity) water saturated sand (Chung et al., 2006; Finnie and Randolph, 1994; Yi et al., 2012). Depth profiles of cone tip resistance, \(q_c\), are provided in Figure 5.3a. The cone tip resistance in the high viscosity pore fluid sand sample is slightly higher than that in the sand saturated with water, which may be due to the expected increase in \(q_c\) with \(v'\) for a dilatant soil, and is consistent with that in the sand over clay sample until approaching the clay layer (at \(z/L \approx 0.4\)).

The clay layer was characterised using a ball penetrometer, with a diameter \(d = 9\,\text{mm}\), which was penetrated through the sand and the underlying clay at \(v = 0.8\,\text{mm/s}\). The undrained shear strength of the clay was determined as \(s_u = \frac{q_{ball}}{N_c}\), where \(q_{ball}\) is the net ball penetration resistance and \(N_c\) is a capacity factor, taken here as \(N_c = 10.5\) (Martin and Randolph, 2006). Profiles of \(s_u\) with depth are provided in Figure 5.3b and are in the range \(s_u = 70 – 90\,\text{kPa}\). Figure 5.3b also includes \(s_u\) determined using the classical expression for overconsolidated soil strength (Ladd et al., 1977)

\[
s_u = \sigma'_v \left( \frac{s_u}{\sigma'_v} \right)_{nc} \text{OCR} \wedge 
\]

(5.1)
where $\sigma'_v$ is the vertical effective stress, $OCR$ is the over-consolidation ratio, $(s_u/\sigma'_v)_{nc}$ is the normally consolidated undrained shear strength ratio and $\Lambda$ is the plastic volumetric strain ratio (Schofield and Wroth, 1968). The best fit between Equation (5.1) and the measurements was obtained using $(s_u/\sigma'_v)_{nc} = 0.19$, which is typical for centrifuge penetrometer measurements in kaolin clay (O'Beirne et al., 2016; Fu et al., 2015; Chow et al., 2014) and $\Lambda = 0.83$, which is within the typical range, $\Lambda = 0.7 – 0.9$ (Mitchell and Soga, 2005).
5.2.3 Test programme and procedure

The representative height of the resultant lateral load, $H$, from wind and waves was selected as $3.5D$ above the mudline, inducing an overturning moment, $M = H \cdot 3.5D$. These cyclic loads were simplified as being sinusoidal and regular, with the load magnitude and symmetry described using the parameters $\zeta_b$ and $\zeta_c$ respectively (LeBlanc et al., 2010)

$$
\zeta_b = \frac{M_{\text{max}}}{M_{\text{ult}}}, \quad \zeta_c = \frac{M_{\text{min}}}{M_{\text{max}}}
$$

(5.2)

where $M_{\text{min}}$ and $M_{\text{max}}$ are the minimum and maximum moments in a load cycle, and $M_{\text{ult}}$ is the ultimate moment capacity. Hence, $\zeta_b$ can vary between 0 and 1, whereas $\zeta_c$ can vary between -1 and 1, with $\zeta_c \geq 0$ and $\zeta_c < 0$ representing one-way and two-way loading, respectively.
The test programme is summarised in Table 5.2 using the test name convention S-n, where S refers to the sample number and n refers to the test number in that sample, e.g. 3-6 refers to the sixth test performed in Sample 3. Samples 1 and 2 are sand samples saturated with water and a high viscosity pore fluid, respectively, while Sample 3 is the sand over clay sample. A total of 11 tests were performed:

- Tests 1-1 and 1-2 examine the effect of soil stress level through comparison with results from identical tests performed at single gravity and reported by Zhu et al. (2018);
- Tests 3-1 to 3-6 explore the effect of the underlying clay layer on suction installation through comparison with tests 2-2 and 2-3 in sand;
- Tests 1-1, 2-2 and 3-1 quantify the ultimate moment capacity in the different soil samples, covering the combined effects of installation method, drainage response, and soil layering;
- Tests 1-2 and 2-1 investigate the influence of drainage conditions on the response to cyclic loading;
- Tests 2-1 and 2-3 investigate the effect of installation method on the response to cyclic loading;
- Tests 2-3 and 3-3 examine the role of soil layering on the response to cyclic loading;
- Tests 3-2 to 3-6 consider the effect of cyclic load magnitude and symmetry on the response to cyclic loading in sand over clay.
Table 5.2 Centrifuge test programme.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Soil sample</th>
<th>Installation type</th>
<th>Loading type</th>
<th>$\zeta_b$</th>
<th>$\zeta_c$</th>
<th>Cycles $N$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>Sand</td>
<td>Jacked</td>
<td>Monotonic</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1-2</td>
<td>Sand</td>
<td>Jacked</td>
<td>Cyclic</td>
<td>0.4</td>
<td>0.1</td>
<td>5,222</td>
</tr>
<tr>
<td>2-1</td>
<td>Sand</td>
<td>Jacked</td>
<td>Cyclic</td>
<td>0.4</td>
<td>0.1</td>
<td>547</td>
</tr>
<tr>
<td>2-2</td>
<td>Sand</td>
<td>Suction</td>
<td>Monotonic</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2-3</td>
<td>Sand</td>
<td>Suction</td>
<td>Cyclic</td>
<td>0.4</td>
<td>0.1</td>
<td>16,377</td>
</tr>
<tr>
<td>3-1</td>
<td>Sand over clay ($H_{sand}/L_{clay} = 0.5$)</td>
<td>Suction</td>
<td>Monotonic</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3-2</td>
<td>Sand</td>
<td>Suction</td>
<td>Cyclic</td>
<td>0.7</td>
<td>0.1</td>
<td>2,369</td>
</tr>
<tr>
<td>3-3</td>
<td>Sand</td>
<td>Suction</td>
<td>Cyclic</td>
<td>0.4</td>
<td>0.1</td>
<td>91,793</td>
</tr>
<tr>
<td>3-4</td>
<td>Sand</td>
<td>Suction</td>
<td>Cyclic</td>
<td>0.4</td>
<td>0.5</td>
<td>4,543</td>
</tr>
<tr>
<td>3-5</td>
<td>Suction in sand (700 cSt)</td>
<td>Suction</td>
<td>Cyclic</td>
<td>0.4</td>
<td>-</td>
<td>16,999</td>
</tr>
<tr>
<td>3-6</td>
<td>Suction in clay (1 cSt)</td>
<td>Suction</td>
<td>Cyclic</td>
<td>0.55</td>
<td>0.1</td>
<td>2,677</td>
</tr>
</tbody>
</table>

The maximum number of load cycles was $N = 91,793$ in Test 3-3. Although this is considerably fewer than the $\sim 10^6$ loading cycles experienced by an OWT over its lifetime, adopting $N = 10^6$ in each of the eight cyclic loading tests would have required approximately six months of centrifuge spinning. A more efficient approach is adopted in this work, which involves investigating the long-term response in tests conducted at single gravity (Zhu et al., 2018), and examining the validity of these tests (in terms of installation method, stress level and drainage response) through comparison with the centrifuge tests reported here.

Each test involved the following testing procedure, as summarised in Figure 5.4:

- **Self-weight penetration.** The model caisson was penetrated under vertical load control at 0.2 kPa/s to mimic a controlled set-down until the targeted applied stress, $V/A = 40$ kPa was reached ($V = 2$ MN, 200 N in model scale and the area, $A = \pi D^2/4$). The valve on the caisson lid was then switched from venting the free fluid to forming a hydraulic connection with the syringe pump.
• **Suction-assisted installation.** The remaining caisson installation was achieved by pumping the fluid from the interior of the caisson at a constant flow rate of 196.4 mm$^3$/s (model scale), whilst maintaining the applied vertical stress constant at 40 kPa. After full installation the caisson was sealed by closing the valve on the caisson lid.

• **Application of additional self-weight.** The vertical load was increased from 200 N to 350 N (i.e. the applied stress increased from $V/A = 40$ to 70 kPa) to model the increase in self-weight due to the installation of the wind turbine above the foundation. The keyway was then disengaged to free the hinge and allow free rotation at the load application point during lateral loading.

• **Monotonic or cyclic loading.** Loading commenced 25 minutes after completion of the installation process, which is equivalent to approximately 6 months at prototype scale (for diffusion problems) to achieve similar drainage conditions to the field (typically 3 - 6 months between the caisson installation and installation of the tower, nacelle and rotor). The monotonic tests were conducted in displacement control, with a (lateral) displacement rate at the hinge of 0.025 mm/s (such that the response is expected to be drained in the sand and undrained in the clay). Each test was maintained until the caisson rotation exceeded 1.5° or until there was no further change in the measured moment resistance. The cyclic loading tests were conducted in load control using the lateral load that was derived from the bending moment measured by the loading arm. The caisson was initially loaded to the maximum cyclic load (i.e. equivalent to $M_{\text{max}}$) at 1 N/s, which resulted in a $M - \theta$ response (up to $M_{\text{max}}$) that was consistent between the initial loading stage in the cyclic tests and the monotonic tests, before applying the sinusoidal cyclic loading histories at a model frequency of 0.5 Hz to target drained, partially drained and undrained behaviour in Samples 1, 2 and 3,
respectively. The vertical applied stress was held constant at 70 kPa during both monotonic and cyclic loading.

- **Post-cyclic monotonic loading.** After each cyclic loading test the lateral load on the caisson was reduced to zero before loading the caisson monotonically (in the same way as the initial monotonic tests, i.e. in displacement control at 0.025 mm/s) to quantify post-cyclic stiffness and capacity. The lateral load was then reduced to 0 N as a step change and held constant for 5 to 10 mins, to allow drainage characteristics to be evaluated.

The assumed suction caisson self-weight of 40 kPa is realistic but was also selected to achieve self-weight penetration to approximately the peak bearing resistance in the sand layer, placing onerous conditions on the suction installation. In the high viscosity pore fluid sand sample (Tests 2-2 and 2-3), a higher self-weight of 70 kPa was chosen to achieve installation conditions comparable to those adopted in Bienen et al. (2018a), which provides a link between the two datasets. Where the caisson was installed by jacking in Samples 1 and 2 (Tests 1-1, 1-2 and 2-1), the caisson was penetrated at 0.2 kPa/s until full penetration was achieved (i.e. $z/L = 1$). The applied vertical stress was then changed to 70 kPa and kept constant during lateral loading.

All operations were controlled remotely without stopping the centrifuge such that the soil stress level was maintained over the entire testing process.
5.3 Results and discussion

5.3.1 Installation

Profiles of penetration resistance measured during installation are provided in Figure 5.5a for the jacked installation tests in sand. Suction installation is expected to result in lower penetration resistance due to the seepage induced reduction in effective stresses around the skirt tip. Figure 5.5b presents the required suction, \( p \), with penetration depth in sand. Suction installation commenced after the end of self-weight installation, which achieved \( z/L = 0.45 - 0.48 \). The required suction pressure varies from \( p = 0 \) at \( z/L \sim 0.45 \) to \( p \sim 50 \) kPa at full penetration, which was achieved at \( z/L = 0.93 - 0.98 \) (see Table 5.3). The corresponding total vertical stress is approximately 120 kPa at full penetration (70 kPa self-weight stress plus 50 kPa suction), which is only 48% of the 250 kPa required for full penetration in the jacked installations.
Figure 5.5 Penetration resistance measured during (a) jacked installation and (b) suction installation in sand.
Table 5.3 Penetration depths during and after installation.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Soil type</th>
<th>Installation method</th>
<th>Penetration depth z/L</th>
<th>Before loading</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Under self-weight</td>
<td>After suction installation</td>
</tr>
<tr>
<td>1-1</td>
<td>Sand</td>
<td>Jacked</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1-2</td>
<td>Jacked</td>
<td>-</td>
<td>-</td>
<td>0.99</td>
</tr>
<tr>
<td>2-1</td>
<td>Sand</td>
<td>Jacked</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2-2</td>
<td>Suction</td>
<td>0.45</td>
<td>0.98</td>
<td>-</td>
</tr>
<tr>
<td>2-3</td>
<td>Suction</td>
<td>0.48</td>
<td>0.93</td>
<td>-</td>
</tr>
<tr>
<td>3-1</td>
<td>Sand over clay</td>
<td>Suction</td>
<td>0.22</td>
<td>0.98</td>
</tr>
<tr>
<td>3-2</td>
<td>Suction</td>
<td>0.26</td>
<td>0.91</td>
<td>-</td>
</tr>
<tr>
<td>3-3</td>
<td>Suction</td>
<td>0.19</td>
<td>0.98</td>
<td>-</td>
</tr>
<tr>
<td>3-4</td>
<td>Suction</td>
<td>0.30</td>
<td>0.98</td>
<td>-</td>
</tr>
<tr>
<td>3-5</td>
<td>Suction</td>
<td>0.24</td>
<td>0.90</td>
<td>-</td>
</tr>
<tr>
<td>3-6</td>
<td>Suction</td>
<td>0.21</td>
<td>0.93</td>
<td>-</td>
</tr>
</tbody>
</table>

Also provided in Figure 5.5 are penetration resistances predicted using the methods of Houlsby and Byrne (2005a) and Senders and Randolph (2009). The input parameters in Table 5.4 were selected to provide the best agreement with the measurements. Slightly different parameters for both methods were reported by Bienen et al. (2018a) for the same suction caisson in the more angular Baskarp silica sand and by Senders and Randolph (2009) for a longer caisson in the same silica sand. Both prediction methods broadly capture the measured responses. For jacked installation, the quality of agreement extends over the full penetration depth for the Houlsby and Byrne method, whereas the prediction using the Senders and Randolph method matches the response well to z/L ≈ 0.5 (see Figure 5.5a). At larger penetration (z/L > 0.5) the direct correlation with constant factors to relate the cone tip resistance to the suction caisson penetration resistance does not capture all aspects of the foundation response.
### Table 5.4 Input parameters for the prediction of required suction.

<table>
<thead>
<tr>
<th>Soil sample</th>
<th>Reference</th>
<th>Approach</th>
<th>Parameter</th>
<th>Value</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand (Sample 2)</td>
<td>Houlsby and Byrne (2005a)</td>
<td>Bearing capacity theory</td>
<td>$K_{\tan \delta}$</td>
<td>0.5</td>
<td>Friction coefficient of interface between skirt and sand (value estimated for a smooth skirt surface in sand)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\phi'$</td>
<td>43°</td>
<td>Friction angle at mean vertical stress of 20 kPa (value determined from database of Lehane and Liu (2013))</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$m$</td>
<td>1.3</td>
<td>Multiple of diameter over which vertical stress is enhanced (value determined from back analysis of measured results)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$k_{\text{inside}}/k_{\text{outside}}$</td>
<td>1.0</td>
<td>Ratio of sand permeability inside and outside of caisson (value determined from back analysis of measured results)</td>
</tr>
<tr>
<td></td>
<td>Senders and Randolph (2009)</td>
<td>CPT correlation</td>
<td>$k_f$</td>
<td>0.0025</td>
<td>Friction coefficient (value calculated from equation 2 in the reference)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$k_p$</td>
<td>0.23</td>
<td>End-bearing coefficient (value determined from back analysis of measured results)</td>
</tr>
<tr>
<td>Sand layer (Sample 3)</td>
<td>Houlsby and Byrne (2005a)</td>
<td>Bearing capacity theory</td>
<td>$K_{\tan \delta}$</td>
<td>0.5</td>
<td>Friction coefficient of interface between skirt and sand (value estimated for a smooth skirt surface in sand)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\phi'$</td>
<td>44.5°</td>
<td>Friction angle at mean vertical stress of 10 kPa (value determined from database of Lehane and Liu (2013))</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$m$</td>
<td>1.3</td>
<td>Multiple of diameter over which vertical stress is enhanced (value determined from back analysis of measured results)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$k_{\text{inside}}/k_{\text{outside}}$</td>
<td>2.5</td>
<td>Ratio of sand permeability inside and outside of caisson (value determined from back analysis of measured results)</td>
</tr>
<tr>
<td>Clay layer (Sample 3)</td>
<td>Houlsby and Byrne (2005b)</td>
<td>Bearing capacity theory</td>
<td>$\alpha_o, \alpha_i$</td>
<td>0.4</td>
<td>Interface friction ratio between skirt and clay (value adopted according to Chen and Randolph 2004)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$N_c$</td>
<td>10</td>
<td>Bearing capacity factor (value falls in the range of lower and upper bounds, i.e., 7 – 12 (Martin and Randolph 2001))</td>
</tr>
</tbody>
</table>
Figure 5.6a shows that the profile of suction pressure with depth in the sand over clay sample (Sample 3) is consistent with that in the sand (Sample 2) until the skirt tip approaches the sand-clay interface. The comparison between Sample 2 and Sample 3 on Figure 5.6a is made by considering the additional penetration, $\Delta z$, after the penetration due to self-weight, which was $z/L \sim 0.4$ in Sample 2 (under $V/A = 70$ kPa) and $z/L \sim 0.2$ in Sample 3 (under $V/A = 40$ kPa). Suction pressure in the sand over clay sample reduces just before the skirts reach the sand-clay interface at $z/L = 0.45$ as shown in Figure 5.6b. This reduction is maintained as the skirts pass into the underlying clay to $z/L \sim 0.6$, at which point the pressure required to continue advancing the caisson starts to increase. At the end of the installation process, $z/L = 0.90 – 0.98$ in sand over clay, with slight increases following suction installation due to the increase in $V/A$ from 40 to 70 kPa (see Table 5.3). Predictions of required suction in the sand and clay layers are also shown in Figure 5.6b, which were obtained by modifying the Houlsby and Byrne (2005a, b) models for single-layer soil cases to consider soil layering, as detailed in the Appendix.
Seepage flow has a significant effect on the caisson penetration rate during suction installation (Bienen et al., 2018a). Figure 5.7a presents the normalised caisson penetration rate, $\dot{z}A/q_{pump}$ where $\dot{z}$ is the caisson penetration rate and $q_{pump}$ is the pumping flow rate. $\dot{z}A/q_{pump} = 1$ signifies that the pumping action is fully transferred to caisson penetration (i.e. no additional seepage flow). This is indeed the case at the commencement of suction installation but gradually decreases, both in the sand sample (Figure 5.7a) and in the sand over clay sample (Figure 5.7b, $z/L < 0.5$), indicating an increase in seepage flow with increased required suction to overcome the increasing penetration resistance with depth. In sand over clay, the lower permeability of the clay prevents seepage flow and increases the efficiency of the suction installation on approach of the bottom layer, with reduced
suction required (Figure 5.6b) for direct transfer of the pumping action to caisson advancement (Figure 5.7b, $\dot{z}A/q_{pump} \approx 1$). The pumping flow rate translates to caisson penetration without significant seepage losses until the caisson is fully penetrated. The suction-induced seepage flow, $q_{seep}$, can be estimated from the method of Houlsby and Byrne (2005a) for the sand and assumed as approximately zero for the clay, which together with the pumping rate, $q_{pump}$, is used to predict the caisson penetration rate $\dot{z} = (q_{pump} - q_{seep})/A$. The predictions, with input parameters as listed in Table 5.5, are seen to capture the measured response well (see Figure 5.7).
Figure 5.7 Normalised caisson penetration rate in: (a) high viscosity pore fluid saturated sand (Sample 2) and (b) sand over clay (Sample 3).

Table 5.5 Input parameters for the prediction of penetration rate.

<table>
<thead>
<tr>
<th>Soil sample</th>
<th>Reference</th>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand (Sample 2)</td>
<td>Houlsby and Byrne (2005a)</td>
<td>$k_c$</td>
<td>$1.7 \times 10^{-7}$ m/s*</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$k_{\text{inside}}/k_{\text{outside}}$</td>
<td>1</td>
</tr>
<tr>
<td>Sand layer (Sample 3)</td>
<td></td>
<td>$k_c$</td>
<td>$1.7 \times 10^{-7}$ m/s*</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$k_{\text{inside}}/k_{\text{outside}}$</td>
<td>2.5</td>
</tr>
</tbody>
</table>

* estimated from $k_c = k_w v_c/v_w$ where viscosity of cellulose ether fluid, $v_c = 700$ cSt, viscosity of water, $v_w = 1$ cSt and permeability of sand saturated with water, $k_w = 1.2 \times 10^{-4}$ m/s (relative density of 83%; Tran (2005)).

5.3.2 Ultimate moment capacity

Figure 5.8 compares the response measured in the monotonic lateral loading tests in all three samples. In the inset of Figure 5.8, the initial stages of the monotonic responses are plotted together with the results from the initial stage of the cyclic loading tests (from
zero moment to $M_{\text{max}}$). The initial stiffness is seen to be broadly similar in the sand samples (Samples 1 and 2, up to $\theta \approx 0.01^\circ$) and higher than in the sand over clay sample (Sample 3) due to the relatively weaker bottom clay layer. At larger rotation (Figure 11), the resistance reaches a plateau when the caisson was installed by jacking (Test 1-1), while the response following suction installation in sand is slightly softer overall before reaching higher capacity at rotations larger than approximately $0.3^\circ$ (Test 2-2). The ultimate moment capacity $M_{\text{ult}}$ is obtained according to the ‘intersection method’ as illustrated in Figure 5.8 as 11.0 MN-m, 11.4 MN-m and 9.8 MN-m (prototype scale) for Tests 1-1, 2-2 and 3-1, respectively.

5.3.3 Accumulated rotation under cyclic loading

Effects of installation method, stress level and drainage in sand

The accumulation of rotation with cycle number is examined in Figure 5.9 for tests in sand with $\zeta_b = 0.4$ and $\zeta_c = 0.1$. Rotation data are expressed in normalised form,
\[ \Delta \theta(N) / \theta_0 = (\theta_N - \theta_0) / \theta_0 \] (LeBlanc et al., 2010), where \( \theta_0 \) and \( \theta_N \) are the maximum rotation during first loading to \( M_{\text{max}} \) and in cycle number \( N \), respectively (Figure 5.10).

The rotation response can be captured by a power law:

\[ \frac{\Delta \theta(N)}{\theta_0} = \beta \times N^\alpha \] (5.3)

where \( \beta \) quantifies the initial rotation from \( \theta_0 \) to \( \theta_1 \) and \( \alpha \) quantifies the rate of rotation accumulation with cycle number, \( N \). The best fit (based on least-squares regression) with the tests on Figure 5.9 and other tests in this research was obtained using \( \alpha = 0.29 \), which is comparable with \( \alpha = 0.31 \) for monopiles (Abadie et al., 2015; LeBlanc et al., 2010), \( \alpha = 0.30 \) (Cox et al., 2014) and \( \alpha = 0.28 \) (Zhu et al., 2018) for suction caissons. Zhu et al. (2013) report a higher \( \alpha = 0.39 \) for suction caissons in loose dry silty sand, and suction caisson data in dense sand reported by Foglia et al. (2014) gave \( \alpha = 0.18 \). Values of \( \theta_0 \), \( \beta \) and \( \alpha \) from this study and previous work are summarised in Table 5.6. Although the rate of caisson rotation (captured by \( \alpha \)) is identical in all tests in this research, the initial rotation when loaded to \( M_{\text{max}}(\theta_0) \) (which reflects the cyclic load magnitude and soil type), and the accumulated rotation after one cycle \( \Delta \theta(1) = \theta_1 - \theta_0 \) (captured by \( \beta \)) differs for each test. This directly affects the absolute magnitude of accumulated rotation, which is held to strict limits over the design life (e.g. 0.5º, DNV 2016). The different initial rotation that arises in sand and sand over clay is quantified in Table 5.6 for different soil self-weight stress levels (i.e. at \( 1g \) and \( Ng \)), different drainage responses and as a result of the more realistic suction assisted installation over jacked installation.

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1 \( \beta \) is used here rather than \( T \) as adopted in previous studies (e.g. LeBlanc et al., 2010; Zhu et al., 2013) to avoid confusion with the dimensionless time, \( T \).
The effect of sand permeability and loading rate is emphasised in the centrifuge test data investigating drainage\(^1\), which indicate the accumulated rotation at \(N = 1\) (i.e. \(\Delta \theta (1) = \theta_0 \times \beta\)) to be higher (by a factor of approximately four for these tests) when the installation response is drained (in water) than partially drained (in high viscosity pore fluid), following jacked installation. Jacked installation appears to lead to lower rotation at \(N = 1\), which highlights the importance of understanding the effects of the installation process on the soil state and optimising the pumping flow rate to minimise loosening. The additional information of initial rotation following jacked installation (Tests 1-1 and 2-1) allows accumulated rotation of suction caissons in sand to be predicted using previously published rates of accumulation (e.g. Cox et al., 2014) as the difference to suction assisted installation is now known.

\[\text{Figure 5.9 Effect of installation method on caisson rotation in sand (}\zeta_b = 0.4, \zeta_c = 0.1).\]

\(^1\) Full dissipation of excess pore pressures was achieved in less than 30 s in the sand with high viscosity pore fluid (Sample 2) following the step change of lateral load at the end of the test. In the sand saturated with water (Sample 1), the dissipation period was too short to be measured, but should be approximately 700 times less than that in Sample 2 due to the difference in pore fluid viscosity.
Assessment of the long-term response to cyclic loading requires data over large numbers of cycles. The centrifuge tests considered here are considered collectively with the equivalent single-gravity test data reported in Zhu et al. (2018) that involved up to one million loading cycles. A comparison is provided in Figure 5.11a for jacked installation in sand ($\zeta_b = 0.4, \zeta_c = 0.1$), which shows that the long-term rate of accumulation is almost identical: $\alpha = 0.28$ in the single-gravity tests and $\alpha = 0.29$ in the centrifuge tests, although the magnitude of $\Delta \theta(N)/\theta_0$ at $N = 1$ is lower in the centrifuge tests ($\beta = 0.06$) than in the single-gravity tests ($\beta = 0.15$). The same rate of accumulated rotation between single gravity and centrifuge tests is also shown to hold for sand over clay ($\zeta_b = 0.4, \zeta_c = 0.1$, Figure 5.11b). This comparison provides support to the approach of using single-gravity tests to assess long-term behaviour, whilst employing centrifuge tests, involving fewer number of loading cycles, to quantify the response at relevant stress levels, including suction installation and pore pressure response.
Figure 5.11 Accumulated caisson rotation for (a) jacked installation in fully drained sand and (b) sand over clay following jacked installation in single gravity test (Zhu et al., 2018) and suction installation in Test 3-3 of this centrifuge study ($\zeta_b = 0.4$, $\zeta_c = 0.1$).
Chapter 5. Suction caisson foundations for offshore wind energy: installation and cyclic response in sand and sand over clay

Effect of underlying clay layer

Figure 5.12 compares $\Delta \theta(N)/\theta_0$ during cyclic loading with $\zeta_b = 0.4$ and $\zeta_c = 0.1$ in sand (Test 2-3) to that in sand over clay (Test 3-3) following suction installation. Also included on Figure 5.12 are fits to the data using Equation (5.3), with $\alpha = 0.29$. The response in the two tests appears broadly similar, although rotation accumulation is initially more rapid in the sand and the rotation eventually stabilises in the sand over clay at $N \approx 10^4$. This stabilisation is not observed in the sand, and although it may be argued that this is due to the lower number of cycles ($N = 16,377$), equivalent tests in Zhu et al. (2018) each with $N \sim 10^6$ show stabilisation in the sand over clay and continuing rotation in the sand. The rotation stabilisation is due to consolidation-induced strength increases in the clay layer, as considered in more detail later in the paper. The step change of lateral load applied at the end of the tests provides an indication of the timeframe, with less than 20% of excess pore pressure dissipation observed over a period of 100 s (model scale). Although the magnitude of $\Delta \theta(N)/\theta_0$ is similar in the sand and the sand over clay, the absolute rotation is slightly higher in the sand over clay, as shown by the inset figure.

Effect of cyclic load magnitude and symmetry

Figure 5.13 allows for an examination of the effect of cyclic load symmetry (Figure 5.13a) and magnitude (Figure 5.13b) on rotation accumulation in sand over clay. Variations in cyclic load symmetry are represented by $\zeta_c = 0.5, 0.1$ and -0.7 at constant cyclic load magnitude, $\zeta_b = 0.4$, whilst the cyclic load magnitude is varied as $\zeta_b = 0.4, 0.55$ and 0.7 at a fixed one-way load symmetry, $\zeta_c = 0.1$. Figure 5.13a shows that the normalised accumulated rotation, $\Delta \theta(N)/\theta_0$, is similar for both one-way cyclic loading cases considered ($\zeta_c = 0.5$ and 0.1) and larger than that under two-way cyclic loading ($\zeta_c = -0.7$). The normalised rotation, $\Delta \theta(N)/\theta_0$, is similar and accumulates with cycle number at the same rate ($\alpha = 0.29$; the stabilised rotation observed in some of the tests was not
included in the regression analysis to obtain $\alpha$ and $\beta$, but as shown by the inset figure, the absolute rotation, $\theta_N$, increases with $\zeta_b$. The increase is apparent by the first cycle indicating that the rotation is simply increasing with load magnitude during the initial loading to $M_{\text{max}}$. The above trends are consistent with the observations of Zhu et al. (2018) from single gravity tests in the same soil profile ($H_{\text{sand}}/L = 0.5$) where the caisson was installed by jacking and water was used as pore fluid.

The rotation in Test 3-3 (in sand over clay) stabilised at about $N = 10^4$. Similar behaviour is also apparent in Test 3-5, although the effect is not as prominent due to the lower number of cycles involved in this test ($N = 16,999$). As similar behaviour was not observed in the sand samples, this stabilising response must be due to strength changes in the clay layer. Supporting evidence is provided in Figure 5.14 which plots the pore-pressure response for Test 3-3 and Test 3-5. Approximately 90% of the excess pore pressure (measured at the invert of the caisson lid) is dissipated by $N = 10^4$, which is approximately the same point at which the rotation stabilised. This consolidation will cause a strength increase in the clay, which will limit the rotation. Also shown on Figure 5.14 is the corresponding pore pressure response for Test 2-3 in sand (saturated with the high viscosity pore fluid), where the pore pressure, $u$, is normalised by the average maximum pore pressure, $u_i$, measured in the sand over clay tests. Accumulation of pore pressures during cyclic loading in sand is negligible compared with that in sand over clay.

Figure 5.13 and Figure 5.14 also include the dimensionless time $T = c_v t / D^2$ (applicable to the sand over clay results), which is shown as a secondary horizontal axes in Figure 5.13 and Figure 5.14. Although the pore pressure is measured at the invert of the caisson lid, which is at the sand interface in the sand over clay tests, from a drainage perspective the soils act in series through sand and clay and because of its much lower value, the $c_v$ for the clay is taken as the effective $c_v$. The use of $T$ (rather than $N$) permits assessment of consolidation for other caisson dimensions and soil properties. For example, caisson
rotation stabilises in the sand over clay tests at $T \approx 0.7$, which for the prototype equivalent of the caisson and soils used in these centrifuge experiments, corresponds to a duration of approximately 6 years.

Figure 5.12 Effect of an underlying clay layer on accumulation of caisson rotation during cyclic loading with $\zeta_b = 0.4$ and $\zeta_c = 0.1$. 

![Graph showing the effect of an underlying clay layer on caisson rotation](image-url)
Table 5.6 Fitted α and β from tests in sand and sand over clay ($H_\text{sand}/L = 0.5$).

<table>
<thead>
<tr>
<th>$\alpha$</th>
<th>$\beta$</th>
<th>$N$</th>
<th>$\theta_0$</th>
<th>Soil type</th>
<th>Install.</th>
<th>Pore fluid in sand</th>
<th>$\zeta_b$</th>
<th>$\zeta_c$</th>
<th>Approach</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.18</td>
<td>0.53</td>
<td>$\sim 10^4$</td>
<td>*</td>
<td>Sand</td>
<td>Jacked</td>
<td>Water</td>
<td>0.4</td>
<td>-0.05</td>
<td>1g</td>
<td>Foglia et al. (2014)†</td>
</tr>
<tr>
<td>0.39</td>
<td>0.10</td>
<td>10,325</td>
<td>*</td>
<td>Dry sand</td>
<td></td>
<td>Dry sand</td>
<td>0.37</td>
<td>0</td>
<td>1g</td>
<td>Zhu et al. (2013)†</td>
</tr>
<tr>
<td>0.30</td>
<td>0.10</td>
<td>71</td>
<td>*</td>
<td>Dry sand</td>
<td></td>
<td>Dry sand</td>
<td>0.4</td>
<td>0.02</td>
<td>Ng</td>
<td>Cox et al. (2014)†</td>
</tr>
<tr>
<td>0.28</td>
<td>0.15 ± 0.03</td>
<td>1,204,998</td>
<td>0.017°</td>
<td>Sand</td>
<td></td>
<td>Water</td>
<td>0.4</td>
<td>0.1</td>
<td>1g</td>
<td>Zhu et al. (2018)†</td>
</tr>
<tr>
<td></td>
<td>0.06</td>
<td>5,222</td>
<td>0.013°</td>
<td></td>
<td>Suction</td>
<td>Water</td>
<td>0.4</td>
<td>0.1</td>
<td></td>
<td>This study, test 1-2</td>
</tr>
<tr>
<td>0.03</td>
<td>0.45</td>
<td>547</td>
<td>0.006°</td>
<td></td>
<td>Suction</td>
<td>High viscosity pore fluid</td>
<td>0.4</td>
<td>0.1</td>
<td></td>
<td>This study, test 2-1</td>
</tr>
<tr>
<td>0.45</td>
<td>0.67</td>
<td>16,377</td>
<td>0.013°</td>
<td></td>
<td>Suction</td>
<td>High viscosity pore fluid</td>
<td>0.4</td>
<td>0.1</td>
<td></td>
<td>This study, test 2-3</td>
</tr>
<tr>
<td>0.62</td>
<td>0.62</td>
<td>91,793</td>
<td>0.029°</td>
<td></td>
<td>Suction</td>
<td>High viscosity pore fluid</td>
<td>0.4</td>
<td>0.1</td>
<td></td>
<td>This study, test 3-3</td>
</tr>
<tr>
<td>0.17</td>
<td>0.52</td>
<td>4,543</td>
<td>0.035°</td>
<td></td>
<td>Suction</td>
<td>High viscosity pore fluid</td>
<td>0.4</td>
<td>0.5</td>
<td></td>
<td>This study, test 3-4</td>
</tr>
<tr>
<td>0.52</td>
<td>0.29</td>
<td>16,999</td>
<td>0.032°</td>
<td></td>
<td>Suction</td>
<td>High viscosity pore fluid</td>
<td>0.4</td>
<td>-0.7</td>
<td></td>
<td>This study, test 3-5</td>
</tr>
<tr>
<td>0.29</td>
<td>0.29</td>
<td>2,369</td>
<td>0.042°</td>
<td></td>
<td>Suction</td>
<td>High viscosity pore fluid</td>
<td>0.55</td>
<td>0.1</td>
<td></td>
<td>This study, test 3-6</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.7</td>
<td>0.1</td>
<td></td>
<td>This study, test 3-1</td>
</tr>
</tbody>
</table>

* Not known

† Only data for $\zeta_b \approx 0.4$ and $\zeta_c \approx 0.1$ are provided here. Parameters relevant to other values of $\zeta_b$ and $\zeta_c$ can be found in the original studies.
Figure 5.13 Accumulated rotation with number of loading cycles for tests in sand over clay: (a) $\zeta_b = 0.4$ with $\zeta_c = 0.5, 0.1$ and -0.7, and (b) $\zeta_c = 0.1$ with $\zeta_b = 0.4, 0.55$ and 0.7.
Figure 5.14 Pore-pressure response during cyclic loading in sand over clay.

Application to field conditions

Combining the response measured in these centrifuge tests (i.e. reflecting appropriate installation methods and stress levels) with the long-term response measured in the single-gravity tests (Zhu et al., 2018) provides an opportunity to predict the rotation of a field scale caisson. This may be achieved using Equation (5.3) with values of $\alpha$ and $\beta$ listed in Table 5.6. For example for a sand over clay seabed, a fatigue limit state would consider $\zeta_b = 0.4$ and $N = 10^6$ (LeBlanc et al., 2010). Caisson rotation then becomes $\theta = \theta_0 (1 + \beta \times N^\alpha) = 0.032(1 + 0.17 \times (10^6)^{0.29}) = 0.3^\circ$ for two-way loading ($\zeta_c = -0.7$) and $\theta = 0.029(1 + 0.67 \times (10^6)^{0.29}) = 1.1^\circ$ for one-way loading ($\zeta_c = 0.1$). On the basis of this simple calculation, the one-way loading would exceed the DNV (2016) rotation limit of $0.5^\circ$, although this conservative estimate neglects the stabilisation of the response due to consolidation in the underlying clay layer at $T \approx 0.7$ as indicated by the
test data. In contrast the same calculations for one-way loading scenario in sand would lead to a more moderate rotation of 0.3° (i.e. using $\beta = 0.45$ and $\theta_0 = 0.013^\circ$).

### 5.3.4 Unloading stiffness

This section examines the effect of cyclic loading on unloading stiffness, $k$, determined from the maximum and minimum loads and rotations in each cycle, as illustrated in Figure 5.10. The rotation amplitude in each cycle ($\theta_{\text{max}} - \theta_{\text{min}}$) is very small ($< 0.01^\circ$), which approaches the resolution limit of the measurement system. Hence, only the unloading stiffness from tests with acceptable measurement of the rotation amplitude is discussed.

**Stiffness in sand**

The evolution of the unloading stiffness with cycle number for tests in sand is shown in Figure 5.15a, where the unloading stiffness is expressed as $k_N / k_1$, which is the unloading stiffness in cycle $N$ relative to that in the first cycle. The evolution of $k_N / k_1$ is similar over the initial ten cycles, and the test results most relevant to the prototype (with high viscosity pore fluid) exhibit a steady increase in stiffness throughout, reaching $k_N / k_1 = 1.2$ after $N = 547$ in Test 2-1 and $k_N / k_1 = 1.5$ after $N = 16,377$ in Test 2-3. The stiffness ratio in the test in the water saturated sand increases to $k_N / k_1 = 1.6$ before showing a drop.

Figure 5.15b compares the response in Test 1-2 with the single-gravity test equivalent to Test 1-2 reported by Zhu et al. (2018). The normalised stiffness, $k_N / k_1$, is almost identical for both tests, including the decrease around $N = 1000$, although this decrease was recovered in the single-gravity test (involving $N \approx 10^6$), reaching $k_N / k_1 = 1.4$. Given the agreement in the rotation response in equivalent single-gravity and centrifuge tests (see Figure 5.11) and the unloading stiffness agreement on Figure 5.15b, unloading stiffness at realistic stress levels is also expected to recover in the long-term.
Figure 5.15 Evolution of unloading stiffness with number of loading cycles in sand ($\zeta_b = 0.4$, $\zeta_c = 0.1$): (a) with different pore fluids and installation methods, and (b) compared with long-term data reported in Zhu et al. (2018).
Stiffness in sand over clay

Figure 5.16a presents the normalised unloading stiffness $k_N/k_1$ in sand over clay (Sample 3). Tests involving one-way loading ($\zeta_c \geq 0$) at relatively low load magnitudes ($\zeta_b = 0.4$) show a reduction in unloading stiffness that partially or fully recovers over the duration of the test. In contrast, the low load magnitude two-way cyclic loading test ($\zeta_c = -0.7$, $\zeta_b = 0.4$) and the one-way cyclic loading test at a higher load magnitude ($\zeta_c = 0.5$, $\zeta_b = 0.7$) show little change in stiffness over the first 200 cycles, but then start to increase, moderately at first, but more rapidly at $N \approx 5,000$ in Test 3-5, which reaches $k_N/k_1 \approx 2.75$ after $N = 10,000$. The point at which the stiffness starts to increase ($N \approx 200$) appears to be consistent with when the pore pressure measured at the lid invert starts to reduce (see Figure 5.14). The more rapid increase in stiffness observed at $N \approx 5,000$ in Test 3-5 is coincident with when pore pressure dissipation is complete, which as shown by Figure 5.14, leads to stabilisation of the rotation and hence a rapid increase in stiffness. Comparisons with equivalent single-gravity tests (see Figure 5.16b) show that the stiffness increase is more moderate and steady in the long-term. The disparity may be due to stress level effects and warrants further attention given the potential for stiffness changes to affect the system dynamics.
Figure 5.16 Evolution of unloading stiffness with number of loading cycles in sand over clay: (a) with different cyclic load magnitude and symmetry ($\zeta_b$ and $\zeta_c$), and (b) compared with long-term data reported in Zhu et al. (2018).
5.3.5 Post-cyclic behaviour

A typical moment-rotation response for a complete test in sand over clay is provided in Figure 5.17. As expected, the stiffness during cyclic loading is moderately higher than the initial stiffness before cyclic loading, as evident from the unload-reload loops which are shown at $N = 1, 10, 100, 1000$ and at the end of the cyclic loading. The stiffness in the post-cyclic monotonic stage of the test is consistent with that at the end of cyclic loading, but reduces when $M > M_{\text{max}}$.

Figure 5.18 presents the moment-rotation responses from both post-cyclic monotonic tests and monotonic tests without cyclic loading, with $\Delta \theta = 0^\circ$ at the start of the post-cyclic monotonic test. As suggested by Figure 5.18, cyclic loading typically leads to a stiffer response in both sand and sand over clay. The ultimate moment capacity (determined using the intersection method) is also higher, with increases of between 21% and 25% in sand, and between 14% and 26% in sand over clay, except for the case of relatively high maximum cyclic load (Test 3-2). At larger rotation ($\theta > 1.5^\circ$), the responses in sand converge, but not in sand over clay due to the consolidation induced increase in strength in the underlying clay layer.
Figure 5.17 Monotonic loading responses after cyclic loading together with typical packets of load cycles for Test 3-2 in sand over clay.
5.4 Concluding remarks

This Chapter presents results from a centrifuge study on the installation and lateral cyclic loading response of a monopod suction caisson for offshore wind turbines in sand and sand over clay soil profiles, where the depth of the upper sand layer is half the caisson skirt length. The main findings are:

1) Suction installation was successful, even when an underlying clay layer provided a boundary to the flow paths for the required reduction of effective stresses around the skirt tip through seepage in the dense sand layer. The required suction and resulting caisson penetration rate can be predicted well using existing methods, which have been modified here to suit sand over clay soil profiles.
2) Differences in caisson rotation were evident by the first cycle and were affected by soil profile, installation method, drainage response, cyclic load magnitude and symmetry.

3) Once normalised by the first cycle, the long-term trend of accumulated of caisson rotation was similar across all tests and consistent with equivalent single-gravity tests.

4) The centrifuge and single-gravity databases can be considered collectively to inform the expected whole-life response at prototype scale.

5) In sand over clay, one-way cyclic loading resulted in a larger rotation than two-way cyclic loading, which contrasts with previous findings in sand.

6) Pore pressure accumulation during cyclic loading was negligible in sand, but more significant when there was an underlying clay layer. Consolidation of the clay layer has a beneficial effect, eventually stabilising caisson rotation and increasing the moment capacity. This also leads to an increase in stiffness – the effect of this increase on the natural frequency of the offshore wind turbine needs to be considered in the design. In comparison, stiffening of the response through densification in sand is more immediate and modest.

5.5 References


Chapter 5. Suction caisson foundations for offshore wind energy: installation and cyclic response in sand and sand over clay


Chapter 5. Suction caisson foundations for offshore wind energy: installation and cyclic response in sand and sand over clay

(Gaidin, C. & White, D. J. (eds)). Leiden, the Netherlands: CRC Press/Balkema (Taylor & Francis Group), pp. 667-673.


Chapter 5. Suction caisson foundations for offshore wind energy: installation and cyclic response in sand and sand over clay


Appendix

The prediction for the upper sand layer of the sand over clay sample \((z/L < 0.5)\) was assumed to be identical to that in sand only (Houlsby and Byrne, 2005a). In this method, the calculated pore pressure factor, \(a\), is controlled by the flow net for the sand. When the caisson penetrates towards the significantly less permeable underlying clay layer, this factor needs to be modified to consider changes in the flow net, as expressed in Cotter (2010):

\[
a = a_0 + 0.15 \frac{z}{D} \left( \frac{z}{H_{\text{sand}}} \right)^2 + \frac{D}{H_{\text{sand}}} - 1 \tag{A1}
\]

where \(a_0\) is the pore pressure factor for sand, \(z\) is the penetration depth, \(H_{\text{sand}}\) is the thickness of the upper sand layer and \(D\) is the caisson diameter. In this study, the modified pore pressure factor was used to predict the penetration response in the upper sand layer.

Prediction of the required suction for caisson penetration in the underlying clay layer was modified from the prediction method for clay (Houlsby and Byrne, 2005b) as follows.

When the caisson skirt tips penetrate the underlying clay, caisson installation resistance is comprised of friction along the caisson-sand and caisson-clay interfaces \(F_{f_{\text{sand}}} \) and \(F_{f_{\text{clay}}} \) and skirt tip resistance \(V_{\text{tip}}\):

\[
R = F_{f_{\text{sand}}} + F_{f_{\text{clay}}} + V_{\text{tip}} \tag{A2}
\]

The frictional force in the upper sand layer \(F_{f_{\text{sand}}}\) can be expressed as the integration of the frictional stress over the thickness of the sand layer

\[
F_{f_{\text{sand}}} = \int_0^{H_{\text{sand}}} \sigma'_{\text{vo}} dz (K\tan\delta)_o (\pi D_o) + \int_0^{H_{\text{sand}}} \sigma'_{\text{vi}} dz (K\tan\delta)_i (\pi D_i) \tag{A3}
\]

where \(\sigma'_{\text{vo}}\) and \(\sigma'_{\text{vi}}\) are the effective vertical stresses inside and outside the caisson, \(D_o\) and \(D_i\) are the outer and inner diameters of the caisson and \(K\tan\delta\) is the friction coefficient of
Chapter 5. Suction caisson foundations for offshore wind energy: installation and cyclic response in sand and sand over clay

interface between skirt and sand. Generally, $\sigma'_{vo}$ and $\sigma'_{vi}$ are affected by seepage flow developed during suction installation in sand (Figure 5.1b). However, when the caisson skirt tip reaches the underlying clay layer, the seepage flow from outside to inside the caisson is prevented (Figure 5.1c), which translates to a normalised penetration rate $\dot{z}A/q_{pump} = 1$ in the clay layer (see Figure 5.7b). Assuming $\sigma'_{vo}$ and $\sigma'_{vi}$ are not influenced by the flow gradient, Equation (A3) can be simplified as (Houlsby and Byrne, 2005a)

$$ F_{f,sand} = \gamma_s' Z_o^2 \left[ \exp \left( \frac{H_{sand}}{Z_o} \right) - 1 - \left( \frac{H_{sand}}{Z_o} \right) \right] (K \tan \delta)(\pi D_o) $$

$$ + \gamma_s' Z_i^2 \left[ \exp \left( \frac{H_{sand}}{Z_i} \right) - 1 - \left( \frac{H_{sand}}{Z_i} \right) \right] (K \tan \delta)(\pi D_i) $$

(A4)

where $\gamma_s'$ is the effective unit weight of sand, $Z_o$ and $Z_i$ are parameters that consider the enhancement of vertical stress in the sand under the downward movement of the caisson.

The frictional force in the clay $F_{f,clay}$ can be written as

$$ F_{f,clay} = (z - H_{sand}) [\alpha_o \bar{s}_u (\pi D_o) + \alpha_i \bar{s}_u (\pi D_i)] $$

(A5)

where $\alpha_o$ and $\alpha_i$ are interface friction ratios on the outside and inside of the caisson respectively, and $\bar{s}_u$ is the average undrained shear strength from the sand-clay interface ($z = H_{sand}$) to the skirt tip penetration depth. Although the skirt tip resistance, $V_{tip}$, is only resisted by the underlying clay layer, the self-weight stress from the upper sand layer needs to be taken account as

$$ V_{tip} = [\gamma_s' H_{sand} + \gamma_c' (z - H_{sand}) + s_u N_c] (\pi Dt) $$

(A6)

where $\gamma_c'$ is the effective unit weight of clay, $N_c$ is a bearing capacity factor, $s_u$ is the undrained strength at the skirt tip penetration depth and $t$ is the skirt thickness.
CHAPTER 6. THE RESPONSE OF SUCTION CAISSONS TO MULTIDIRECTIONAL LATERAL CYCLIC LOADING IN SAND OVER CLAY

ABSTRACT: Offshore wind turbines and their foundations experience loading from varying directions, yet understanding of the effect of changes in the direction of applied cyclic loading on foundation response remains limited. This study investigates the behaviour of a suction caisson foundation in sand over clay under various types of multidirectional cyclic loading. The findings are significant: although the caisson rotation (or tilt) is affected by changes in load direction, the caisson rotation is less than that in a unidirectional test with the same cyclic load magnitude and symmetry, meaning that a unidirectional test may serve as a conservative estimate of caisson rotation for multidirectional loading. This contrasts with previous findings for monopiles in sand, where prediction of the accumulated displacement based on unidirectional cyclic loading was substantially un-conservative for multidirectional loading. Changes in unloading stiffness due to changes in load direction are slight, varying by an amount that is no greater than the change measured over the course of a unidirectional test. Foundation stiffness and ultimate capacity following multidirectional cyclic loading are largely unaffected, unlike unidirectional cyclic loading, where consolidation of the clay layer improves both the stiffness and capacity.

Keywords: Suction caisson; multidirectional cyclic loading; offshore wind; physical modelling; layered soil.
Chapter 6. The response of suction caissons to multidirectional lateral cyclic loading in sand over clay

6.1 Introduction

While monopiles remain the favoured foundation type for bottom-founded offshore wind turbines, suction caissons represent an increasingly considered alternative foundation concept, which offers cost-effective installation with minimal acoustic emissions. The installation does not require specialist equipment as the suction caisson skirt is penetrated into the seabed initially under self-weight, and in a second stage by pumping water from the caisson interior (a process that can be reversed for removal of the caisson). Suction caissons were first used as the foundation of a 3 MW wind turbine at Frederikshave, Denmark in 2002 (Ibsen and Brincker, 2004). Since then they have received considerable attention (e.g. Byrne and Houlsby, 2004; Cox et al., 2014; Foglia et al., 2014), with a number of proprietary offshore trial installations in the North Sea (Tjelta, 2015).

Figure 6.1a shows a typical offshore wind turbine supported by a suction caisson. The resultant of the environmental wind and wave loads act laterally at some distance, $e$, above the seabed, inducing a horizontal load, $H$, and an overturning moment, $M$, at the foundation. The wind turbine system is continuously exposed to these loads, which are cyclic in nature, over its (typically) 25 year service life. This cyclic loading has the potential to lead to an accumulated rotation of the caisson, and hence the wind turbine, and also to alter the foundation stiffness. Both aspects need to be considered in the design process, as strict limits on accumulated rotation are typically enforced (e.g. 0.5°; DNV (2016)), and changes in foundation stiffness affects the system dynamics, with requirements that the natural frequency of the system avoids the forcing frequencies of the rotor frequency and the blade passing frequency.
Figure 6.1 (a) Monopod suction caisson foundation for an offshore wind turbine and (b) wind rose at meteorological stations: Fino 1 in North Sea and Fino 2 in Baltic Sea (http://fino.bsh.de/).
Suction caisson response to lateral cyclic loading has been investigated experimentally in single-gravity tests (e.g. Byrne and Houlsby, 2004; Zhu et al., 2013; Foglia et al., 2014; Nielsen et al. 2017; Zhu et al., 2018a¹), centrifuge tests (e.g. Watson and Randolph, 2006; Cox et al., 2014; Zhu et al., 2018b²) and reduced scale field tests (Houlsby et al., 2005, 2006). These studies considered the effects of cyclic load magnitude and symmetry, but not load direction – i.e. the cyclic loading was unidirectional. However, wind and wave direction varies. Example wind roses (graphical representations of wind intensity and direction) measured from Fino 1 and 2 meteorological stations in the North Sea and Baltic Sea (Figure 6.1b) show that the load direction varies between 0° and 360°, although typically limited to Southwest (or between 180° and 270°). The response of suction caissons under multidirectional cyclic loading is not understood, although there is a limited number of studies on the effect of multidirectional loading on monopiles (Levy et al., 2007; Su 2012; Su and Liu, 2013; Sheil 2014; Rudolph et al., 2014; Nanda et al., 2017). This limitation is the motivation for this study.

This paper considers single-gravity model caisson tests in a sand over clay layered seabed. This layering profile is prevalent in the North Sea (and consists of a surface sand layer of up to 10 m over stiff clay, Bond et al., 1997), where more than 80% of European wind power has been installed (EWEA, 2016). This new data investigating the effect of multidirectional loading provides a link with previous research on suction caissons under unidirectional cyclic loading in the same soil profile (Zhu et al., 2018a, b), which considerably adds to the limited database of suction caisson response in layered soils.

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¹ Chapter 4
² Chapter 5
6.2 Details of the experiments

The experimental approach adopted in this work is to conduct reduced scale model tests at single gravity. As shown by Zhu et al. (2018b), conducting long-term cyclic loading tests on suction caissons at single gravity is an efficient modelling approach, particularly when the results are coupled with short-term centrifuge tests that capture the effect of suction installation, and quantify the initial loading stiffness. The experimental arrangement adopted here is similar to that described in Zhu et al. (2018a), but with modifications to permit load direction changes.

6.2.1 Soil properties and sample preparation

The soil samples were prepared as sand over clay, with a sand layer thickness equal to 0.5 times the skirt length (or \( H_{\text{sand}}/L = 0.5 \)). The targeted soil properties were \( D_r = 42\% \) for the sand and \( s_u = 9 \text{ kPa} \) for the clay, selected to capture the behaviour of dense sand (\( D_r = 80\% \)) overlying stiff clay (\( s_u = 80 \text{ kPa} \)) in a 1:100 reduced scale single-gravity model test, as this soil profile is typical for areas of wind farm development in the North Sea (De Ruiter and Fox, 1975; Bond et al., 1997; BGS, 2002). The scaling approach ensures similitude in both the friction angle and the ratio of sand stiffness to clay stiffness in the field and the model, as discussed in detail in Zhu et al. (2018a).

The soil samples were created in steel circular sample containers with an internal diameter of 600 mm and an internal height of 400 mm using a fine to medium sub-angular silica sand and kaolin clay; the material properties of the sand and clay are summarised in Table 6.1. The underlying clay layer was prepared by consolidating kaolin clay slurry at an initial moisture content of 120% (approximately twice the liquid limit, see Table 6.1) in an oedometric style consolidation press in stress increments to a final vertical effective stress of 110 kPa. After consolidation the clay was left to swell for one week to reach a steady undrained shear strength as assessed from periodic T-bar penetrometer tests. The
The clay surface was then scraped carefully to achieve the targeted clay depth of 285 mm and a level surface. The overlying 40 mm sand layer was prepared by air pluviation, followed by saturation with water from the base of the sand layer. This was achieved using four plastic pipes, perforated at the base, which were located along the inner wall of the sample container (see Figure 6.2a) with their inverts positioned at the clay sand interface. After saturation a 50 mm layer of free water was added to the sample surface and maintained over the course of the tests (using a float valve on the water supply line). The height of the four samples prepared for this study was 325 ± 2 mm.

Table 6.1 Properties of silica sand (Chow et al., 2015; Lee et al., 2013) and kaolin clay (Stewart, 1992; Richardson et al., 2009).

<table>
<thead>
<tr>
<th></th>
<th>Silica sand</th>
<th>Kaolin clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity, (G_s)</td>
<td>2.65</td>
<td>2.6</td>
</tr>
<tr>
<td>Particle size, (d_{50}) (mm)</td>
<td>0.19</td>
<td>Liquid limit, LL (%)</td>
</tr>
<tr>
<td>Minimum dry density, (\rho_{\text{min}}) (kg/m(^3))</td>
<td>1461(^*)</td>
<td>Plastic limit, PL (%)</td>
</tr>
<tr>
<td>Maximum dry density, (\rho_{\text{max}}) (kg/m(^3))</td>
<td>1774(^+)</td>
<td>Plastic index, (I_p) (%)</td>
</tr>
<tr>
<td>Critical state friction angle, (\phi'_{\text{crit}}) (º)</td>
<td>30</td>
<td>Angle of internal friction (\phi') (º)</td>
</tr>
<tr>
<td>Coefficient of consolidation, (c_v) (m(^2)/year)</td>
<td>&gt; 60,000</td>
<td>Coefficient of consolidation, (c_v) (m(^2)/year) at (\sigma'_{\text{v}} = 110\text{kPa})</td>
</tr>
</tbody>
</table>

\(^*\) derived according to test standard ASTM D4253-00
\(^+\) derived according to test standard ASTM D4254-00

A model cone penetrometer with a diameter, \(d = 10\) mm was used to characterise the soil samples. The cone penetrometer was penetrated at velocity, \(v = 1\) mm/s such that the dimensionless velocity, \(v' = vd/c_v = 121\) and < 0.01 for the clay and sand respectively (where \(c_v\) is the vertical coefficient of consolidation, taken as \(c_v = 2.6\) m\(^2\)/year for the clay and \(c_v\) is at least 60,000 m\(^2\)/year for the sand (Lee et al., 2013), see details in Table 6.1). The magnitudes of \(v'\) are such that the cone resistance is expected to be undrained in clay and drained in sand (Chung et al., 2006). Figure 6.3 shows profiles of total cone resistance, \(q_c\), with normalised penetration depth, \(z/L\) (i.e. penetration depth, \(z\), normalised
by the caisson skirt length, $L$) measured across each sample. The development of cone tip resistance with depth is consistent between the four samples, with $q_c$ increasing through the sand layer and more rapidly when reaching the clay surface at $z/L = 0.5$, before remaining approximately constant in the clay layer. The undrained shear strength of clay was determined from T-bar tests penetrated through the sand into the clay layer, as shown by the inset plot in Figure 6.3, where $s_u$ was determined from the measured T-bar resistance using the T-bar factor, $N_{T-bar} = 10.5$ (Martin and Randolph, 2006). Also shown on the $s_u$ inset plot in Figure 6.3 is the classical expression (Ladd et al., 1977)

$$s_u = \sigma'_v \left( \frac{s_u}{\sigma'_v} \right)_{nc} OCR \Lambda$$

(6.1)

where $\sigma'_v$ is the vertical effective stress determined using an effective unit weight, $\gamma' = 7.1$ kN/m$^3$ (as established from moisture content measurements made on post-testing sample cores), $(s_u/\sigma'_v)_{nc}$ is the normally consolidated undrained shear strength ratio, $OCR$ is the over-consolidation ratio and $\Lambda$ is the plastic volumetric strain ratio (Schofield and Wroth, 1968). The best fit between Equation (6.1) and the T-bar measurements was obtained using $(s_u/\sigma'_v)_{nc} = 0.18$, which is typical for penetrometer derived measurements in kaolin clay (Chow et al., 2014; Fu et al., 2015; O'Beirne et al., 2017) and $\Lambda = 0.83$, which is within the typical range, $\Lambda = 0.7 - 0.9$ (Mitchell and Soga, 2005). As shown in Figure 6.3, $s_u$ is in the range of 6 to 10 kPa over the penetration depth. The relative density of the sand layers was $D_r = 39 \pm 1\%$ ($\gamma' \approx 9.8$ kN/m$^3$) across the four samples as determined from global measurements of sand mass and volume after saturation. These measurements of $s_u$ and $D_r$ are consistent with those in the unidirectional tests reported in Zhu et al. (2018a).
Figure 6.2 Experimental arrangement at different stages: (a) before installation, (b) after installation, (c) release of installation actuator in preparation for cyclic loading and (d) cyclic loading.
6.2.2 Model caisson, instrumentation, experimental arrangement and testing procedure

The model caisson had a diameter, \( D = 160 \text{ mm} \), a skirt length, \( L = 80 \text{ mm} \) and a skirt thickness, \( t = 1 \text{ mm} \). The caisson skirt was made of aluminium and was anodised, which gave a surface roughness of \( R_a = 1.62 \mu m \). The caisson lid was made from polymethyl methacrylate (PMMA) allowing for a visual access of the lid-soil interface. A vent (Figure 6.4c) on the caisson lid allowed for expulsion of air and water during penetration. The caisson lid was connected rigidly to a 30 mm diameter solid aluminium ‘vertical loading shaft’ that allowed the lateral loads to be applied at an eccentricity, \( e \) from the invert of the caisson lid, such that overturning moments are generated at the caisson. This shaft was sufficiently rigid such that the deflections due to the eccentrically applied load were negligible relative to the displacement at the loading point. The combined weight of the caisson and shaft results in a vertical load, \( V = 25 \text{ N} \), or in dimensionless terms, \( V'\gamma' D^3 = 0.62 \) (in sand; \( \gamma' = 9.8 \text{ kN/m}^3 \)), which is identical to that adopted in the baseline.
unidirectional tests reported in Zhu et al. (2018a), and within the $V/\gamma' D^3 = 0.09 - 0.91$ range reported in Foglia and Ibsen (2016) for field scale suction caissons supporting offshore wind turbines.

Figure 6.4 provides a photo of the apparatus, with details of the pulley system that allowed the application of multidirectional loading, and the displacement measurement system. Figure 6.2 shows the experimental arrangement at various stages in the tests. Each test follows the same procedure involving:

- **Installation.** Before the caisson was installed it was connected via the shaft and a load cell (‘vertical load cell’ in Figure 6.2a) to the installation actuator located atop the frame, to ensure that the caisson remained vertical during installation. The load cell measured the penetration resistance during installation. The caisson was installed at a velocity of $v = 0.05 \text{ mm/s}$, which corresponds to a dimensionless velocity, $v' (= vt/c_v) = 0.6$ in clay and $v' < 0.01$ in sand, such that partially drained in clay and drained in sand is expected during installation (Finnie and Randolph, 1994; Chung et al., 2006; Yi et al., 2012). Installation was deemed complete when the caisson lid made contact with the soil surface (Figure 6.2b), assessed on the basis of visual observations through the caisson lid, which were confirmed when the caisson installation resistance increased suddenly. The vent on the caisson lid was then sealed and the installation actuator was removed (Figure 6.2c). Throughout the installation process the lateral loading apparatus was connected, but with slack in the wire in the pulley system, such that the lateral load was zero and interference during installation was avoided (Figure 6.2a). The wire was supported on a Teflon collar to minimise friction, which was held in position by an aluminium clamp.

- **Cyclic loading.** The loading actuator (located on the side of the frame, see Figure 6.2 and Figure 6.4) was used to apply tensile lateral loads at a fixed height on the
loading shaft (Figure 6.2d). Figure 6.4b shows the pulley system that allowed lateral loads to be applied in load directions of 0°, 45°, 90°, and 135°, noting that the maximum load direction variation (135°) more than spans the typical load variation of 90° indicated by the wind roses. A load cell (‘horizontal load cell’ in Figure 6.2a) located in series on the loading wire, applied a one-way lateral load, $H$, which generated a moment, $M = H \cdot e$ at the reference point ($RP$, Figure 6.2d). Each cyclic load test involved a preload stage, in which the caisson was loaded to the maximum cyclic load, $M_{\text{max}}$, at a rate of 0.05 Nm/s. The cyclic loads were then applied at a frequency of 1 Hz (resulting in a drained response in sand and an undrained response in clay over one cycle but partially drained response in clay over many cycles; Zhu et al. (2018a)) for the target number of cycles. The tension in the wire was then reduced to zero to allow for a change in the load direction. A 10 min duration for the load direction change was maintained for each change in load direction in each test.

- **Post-cyclic monotonic loading.** Following cyclic loading, the tension in the wire was released such that $H = M = 0$, and the wire adjusted such that load direction returned to the initial loading direction (0°). The caisson was then loaded monotonically under displacement control, with a displacement rate at the load application point of 0.05 mm/s, to quantify post-cyclic stiffness and capacity. This stage had a duration of $t = 400$ s, with a corresponding dimensionless time factor, $T = c_v t/D^2 < 0.0013$ for clay and $> 10$ for sand, such that the response is expected to be undrained in the clay and drained in the sand.

Caisson rotation and displacements were determined from measurements made by four laser sensors oriented towards a target that was parallel with the caisson lid but remained above the water surface as shown by Figure 6.4c.
6.2.3 Testing programme

The lateral loads were applied at an eccentricity, $e = 3.5D$ above the mudline, such that the moment loading at the load reference point, $M = H \cdot 3.5D$, is typical of that experienced by a 5 – 7.5 MW turbine in 20-50 m water depth (Cox et al. 2014; Petrini et al. 2010) and adopted in the equivalent unidirectional loading model tests reported in Zhu et al. (2018a).

Packets of cyclic loading were applied along four load directions, $0^\circ$, $45^\circ$, $90^\circ$ and $135^\circ$ (see Figure 6.5), where $0^\circ$ is the initial loading direction and the direction adopted for the post-cyclic monotonic tests. Tests M1 to M3 were designed to investigate the effect of change between the initial and one secondary load direction, i.e. the load direction

Figure 6.4 Photographs showing: (a) experimental arrangement during caisson installation and details of (b) pulley system and (c) displacement measurement system.
changed from $0^\circ$ to $45^\circ$, $0^\circ$ to $90^\circ$ and $0^\circ$ to $135^\circ$, respectively, in Tests M1, M2 and M3. Test M4 investigates the effect of a successive change in load direction from $0^\circ$ to $45^\circ$, $90^\circ$ and $135^\circ$. One sequence of multidirectional loading in M1 to M4 consists of 90,000 cycles with half at $0^\circ$ and half in the other direction(s), as illustrated in Figure 6.6. This sequence was repeated four times in each test, i.e. 360,000 cycles in total, to provide a basis for understanding the effect of repeated changes in loading direction on the caisson response.

Figure 6.5 Illustration of main loading directions.
Figure 6.6 Loading sequences showing load direction and number of loading cycles: (a) 0° to 45° (M1), (b) 0° to 90° (M2), (c) 0° to 135° (M3) and (d) 0°, 45°, 90° to 135° (M4).

The cyclic loads applied in each direction were simplified as sinusoidal waves with the load magnitude and symmetry described using the parameters $\zeta_b$ and $\zeta_c$ respectively (LeBlanc et al., 2010)

$$ \zeta_b = \frac{M_{\text{max}}}{M_{\text{ult}}} , \quad \zeta_c = \frac{M_{\text{min}}}{M_{\text{max}}} \tag{6.2} $$

where $M_{\text{min}}$ and $M_{\text{max}}$ are the minimum and maximum moments in a load cycle and $M_{\text{ult}}$ is the ultimate moment capacity, which was quantified in monotonic loading tests conducted in separate samples with the same soil layering and strength properties as described in Zhu et al. (2018a). $\zeta_b$ can vary between 0 and 1, whereas $\zeta_c$ can vary between -1 and 1, with $\zeta_c \geq 0$ and $\zeta_c < 0$ representing one-way and two-way loading, respectively.

As indicated by LeBlanc et al. (2010), whilst the load direction varies, the load symmetry
is typically one-way. Hence the load symmetry in these tests was given by $\zeta_c = 0.1$ to reflect the dominant load symmetry, with the cyclic load magnitude given by $\zeta_b = 0.4$ to represent a fatigue limit state (LeBlanc et al., 2010). These load characteristics, $\zeta_b = 0.4$ and $\zeta_c = 0.1$, are the same as the baseline load case in Zhu et al. (2018a, b), which permits a link between those datasets and that presented here.

6.3 Results and discussion

6.3.1 Definitions

The motion of the caisson under multidirectional cyclic loading is complex, as illustrated in Figure 6.7a. The lateral displacement at the loading point, $P$, relative to the reference point at the centre of caisson lid ($RP$) is described by $\Delta u$. Defining the initial loading direction as along the $x$ axis in Figure 6.7a, the current direction of loading ($H$ as illustrated in Figure 6.7a) is described by the angle, $\psi_L$, with respect to the $x$ axis. Hence the direction of the initial loading in each test is given by $\psi_L = 0^\circ$. The direction of the lateral displacement is described by the angle, $\psi_D$, between the $x$ axis and the projection of the caisson shaft centre line in the $x$-$y$ plane ($OP$ as illustrated in Figure 6.7a). Non-verticality of the caisson (and hence the substructure and the wind turbine it supports) is of particular concern in design and is quantified here by the tilt angle, $\theta$. The geometric relationship between the lateral displacement at the loading point, $\Delta u$, and the tilt angle is $\theta = \sin^{-1}(\Delta u/e) = \sin^{-1}(\sqrt{\Delta u_x^2 + \Delta u_y^2}/e)$ where $\Delta u_x$ and $\Delta u_y$ are components of $\Delta u$ in the $x$ and $y$ directions and $e$ is the lateral load eccentricity as defined earlier.

The unloading stiffness is defined as $k = (M_{\text{max}} - M_{\text{min}})/\theta_{\text{cyc}}$, where $\theta_{\text{cyc}}$ is the relative caisson rotation from $M_{\text{max}}$ to $M_{\text{min}}$ in each cycle. $\theta_{\text{cyc}}$ is the angle between caisson shaft centre lines and may be expressed by $\theta_{\text{cyc}} = \sin^{-1}(\Delta u_{\text{cyc}}/e)$ according to small angle approximation, where $\Delta u_{\text{cyc}}$ is the relative lateral displacement from the loading point $P_{\text{max}}$ (at $M_{\text{max}}$) to $P_{\text{min}}$ (at $M_{\text{min}}$) in each cycle as illustrated in Figure 6.7b. It is worthwhile
to note that $\theta_{\text{cyc}}$ is simply equal to $\theta_{\text{max}} - \theta_{\text{min}}$ if $\psi_D$ does not change in each cycle. As a convention, $\theta_{\text{cyc}}$ is hereafter referred to as the relative caisson rotation while $\theta$ as the caisson rotation.

![Diagram of caisson rotation and lateral displacement](image)

Figure 6.7 Notation for rotations and displacements: (a) caisson rotation, $\theta$, loading direction, $\psi_L$, and direction of lateral displacement, $\psi_D$, and (b) relative caisson rotation, $\theta_{\text{cyc}}$, in each cycle.

### 6.3.2 Lateral displacements at loading point

Figure 6.8 presents the evolution of lateral displacements, both in magnitude and direction, at the loading point during the first sequence of cyclic loading for each test. The lateral displacements ($\Delta u = \sqrt{\Delta u_x^2 + \Delta u_y^2}$) follow the load direction during the initial $N = 45,000$ cycles applied at $\psi_L = 0^\circ$ in all four tests. This loading packet results in a
permanent displacement (e.g. $\Delta u_x/D = 0.005$ and $\Delta u_y/D = 0$ (M1) in Figure 6.8a).

Although the relative displacement in each cycle in the secondary load direction generally follows this direction (as illustrated by the arrow indicating $\psi_L = 45^\circ$ (M1) in Figure 6.8a), the direction of overall displacement at the loading point is not consistent with but only gradually moves towards the direction of load application (e.g. $\psi_D < \psi_L = 45^\circ$ (M1) in Figure 6.8a). This trend is observed in all four tests, indicating the influence of the permanent displacement (due to the initial $N = 45,000$ cycles applied at $\psi_L = 0^\circ$) on the response in the secondary direction.
Chapter 6. The response of suction caissons to multidirectional lateral cyclic loading in sand over clay
Figure 6.8 Evolution of normalised lateral displacements at loading point during first loading sequence: (a) 0° to 45° (M1), (b) 0° to 90° (M2), (c) 0° to 135° (M3) and (d) 0°, 45°, 90° to 135° (M4).

To illustrate the observations in the secondary loading packets shown in Figure 6.8a, b and c, Figure 6.7a provides a schematic illustration. The lateral load $H$ (applied at the loading point $P$, at its current location) is comprised of two components: $H_p$ and $H_n$, parallel with and normal to $OP$ respectively, where $OP$ is the projection of the caisson shaft centre line in the x-y plane as illustrated in Figure 6.7a. $H_p$ affects the relative lateral displacement $\Delta u$ and the caisson rotation, $\theta$, while $H_n$ influences the direction of the resulting displacement, $\psi_D$. When the direction of displacement coincides with the applied load direction (i.e. $\psi_D = \psi_L$, with $H_p = H$ and $H_n = 0$), $\Delta u$ changes whilst $\psi_D$ remains constant. This behaviour was observed in the first packet of cyclic loading at $\psi_L = 0^\circ$. Hence, it is reasonable that the overall displacement gradually tends towards, but does not exceed the applied load direction.
Test M3 considers the effect of a sudden load direction change from $\psi_L = 0^\circ$ to $135^\circ$, with $N = 45,000$ in each packet of cycles. Test M4 examines a more progressive load direction change, with the load direction sweeping through $\psi_L = 0^\circ$, $45^\circ$, $90^\circ$ and $135^\circ$ to coincide with the load directions investigated in the remaining tests. The $\psi_L = 0^\circ$ load direction was maintained for $N = 45,000$ (consistent with the other tests), whilst the $\psi_L = 45^\circ$, $90^\circ$ and $135^\circ$ load directions were each maintained for $N = 15,000$, such that the total number of cycles applied in the first sequence was $N = 90,000$. Although both tests M3 and M4 conclude with the same load direction, i.e. $\psi_L = 135^\circ$, the displacements in each test are significantly different as an effect of the loading history (Figure 6.8c and d). After $N = 15,000$ at $\psi = 135^\circ$ in test M3, the normalised lateral displacements at the loading point are $\Delta u_x/D = -0.002$ and $\Delta u_y/D = 0.003$, whereas in test M4 the displacement direction is quite dissimilar to the load direction, with $\Delta u_x/D = 0.001$ and $\Delta u_y/D = 0.003$. The different displacement trajectory is considered to be as a result of the plastic displacements that resulted from the intervening load directions.

Figure 6.9 presents the evolution of $\Delta u_x/D$ and $\Delta u_y/D$ over the entire cyclic loading history ($N = 360,000$), in which the first loading sequences shown in Figure 6.8 were repeated four times. Consistent with the observations during load application in the secondary load directions shown in Figure 6.8, the direction of overall displacement in the repeated packets do not follow the applied load direction, i.e. $\psi_D \neq \psi_L$ (neither returning to $\psi_D = 0^\circ$ nor coinciding with the secondary direction, e.g. $\psi_D = 45^\circ$ (M1) in Figure 6.9a). The sector that the displacements cover narrowing with each repetition of the cyclic loading sequence. The lateral displacement relative to the caisson reference point, $RP$, continues to increase as the cyclic loading sequences are repeated (although at a decreasing rate), meaning that the caisson rotation is also increasing (as shown later in the paper). This is more pronounced when the load direction change does not exceed $90^\circ$ (Tests M1 and M2, Figure 6.9a and b) as the displacements are partially recovered when the load direction...
change exceeds 90° (Tests M3 and M4, Figure 6.9c and d), due to the tendency for the displacement path to follow the load direction. This is discussed in more detail later in the paper in terms of caisson rotation, \( \theta \) (recalling that the geometric relationship between \( \theta \) and the resultant magnitude of displacement, \( \Delta u = \sqrt{\Delta u_x^2 + \Delta u_y^2} = e \sin \theta \)).
Figure 6.9 Evolution of normalised lateral displacements at loading point during all loading sequences: (a) 0° to 45° (M1), (b) 0° to 90° (M2), (c) 0° to 135° (M3) and (d) 0°, 45°, 90° to 135° (M4).
6.3.3 Direction of displacements

Figure 6.10 shows the evolution of direction $\psi_D$ with cycle number. In this first packet of cycles, the direction coincides with the load direction, i.e., $\psi_D = \psi_L = 0^\circ$ which is consistent with the trajectories of lateral displacements shown in Figure 6.8. When the load direction changes after $N = 45,000$, $\psi_D$ increases quickly and tends towards a constant value (Tests M1 to M3). The magnitude of $\psi_D$ at the end of the second packet of cycles is lower than the load direction due to the permanent displacements due to loading at $\psi_L = 0^\circ$ in the first packet of cycles ($\psi_D$ approaches 28° for loading at 45° (Test M1), 70° for loading at 90° (Test M2), and 122° for loading at 135° (Test M3)). In contrast, the resulting direction is $\psi_D = 68^\circ$ when the load direction changes in 45° increments to a final load direction of 135° (Test M4), which is similar to $\psi_D = 70^\circ$ after the same number of cycles applied at a secondary load direction of 90° in Test M2. A permanent direction offset remains when the load direction returns to 0°, which is similar in Tests M1 and M4, and higher but also similar in Tests M2 and M3. These trends continue as the cyclic sequences are repeated over the duration of the loading history, with the highest $\psi_D$ in a packet $\psi_{\text{max}}$ decreasing slightly and lowest $\psi_D$ in a packet $\psi_{\text{min}}$ increasing slightly with each repetition of the cyclic sequence. After 360,000 cycles, $\psi_{\text{max}} = 26^\circ$, 62°, 109° and 50° in Tests M1, M2, M3 and M4 respectively, reduced by around 10% (M1 to M3) and 26% (M4) after the first sequence.
Figure 6.10 Direction of lateral displacement at loading point during the four sequences of cyclic loading.

6.3.4 Caisson rotation

Figure 6.11a presents the evolution of the maximum caisson rotation, $\theta$, in each cycle with cycle number in each multidirectional test together with unidirectional loading counterpart tests reported in Zhu et al. (2018a), with $\zeta_b = 0.4$, and $\zeta_c = 0.1$ and $\zeta_c = -0.7$. The latter was included as it features asymmetric two-way loading and is analogous to Tests M3 and M4, which also include load reversals, albeit not in every load cycle but at changes in load direction (from $0^\circ$ to $135^\circ$). Figure 6.11a also provides the displacement at the loading point, $\Delta u$, normalised by the caisson diameter $D$. 
Figure 6.11 Evolution of caisson rotation with number of cycles: (a) caisson rotation and (b) normalised rotation.
Figure 6.11a shows that overall the caisson rotation continues to increase when the change in load direction was $\psi_L \leq 90^\circ$ (M1 and M2), but reduces when the load direction changes from $0^\circ$ to $135^\circ$ (M3 and M4). In all tests the caisson rotation reduces immediately after a change to the secondary load direction by an amount that is proportional to the change in load direction. In tests M1 to M3 the caisson rotation increases quickly after this reduction, whereas in test M4 (with the incremental changes in load direction) the reduction continued until the load direction returns to $0^\circ$. This behaviour is consistent with the trajectory of lateral displacements shown in Figure 6.8, which is to be expected given that the caisson rotation $\theta$ is proportional to the magnitude of lateral displacement $\Delta u$, or $\theta = \sin^{-1}(\Delta u/e)$. Specifically, magnitude of $\Delta u$ (denoted as the distance from the displacement trajectory to the origin in Figure 6.8) in tests M1 to M3 continues to increase following the initial reduction after the change of load direction, while in M4 continues to decrease when the load direction changes to $90^\circ$ and $135^\circ$.

Generally, the overall caisson rotation under multidirectional loading is close to that under unidirectional loading with the same cyclic load and symmetry characteristics (i.e. $\zeta_b = 0.4$, $\zeta_c = 0.1$) for load direction changes of $45^\circ$ (Test M1) and $90^\circ$ (Test M2). When the load direction changes are greater than $90^\circ$ the rotation is lower, due to the compensating effect of loading components that act opposite to the initial direction of $0^\circ$, hence reducing the rotation generated in the previous packet. However, caisson rotation in the multidirectional tests is higher for a load direction of $135^\circ$ (Tests M3 and M4) than for the $\zeta_c = -0.7$ unidirectional test, in which with very little rotation accumulated over the 360,000 cycles.

Figure 6.11b shows the same data expressed in normalised form, $\Delta \theta(N)/\theta_0$ (Leblanc et al., 2010) where $\Delta \theta(N) = \theta_N - \theta_0$ and $\theta_0$ and $\theta_N$ are the maximum rotation during preloading to $M_{\text{max}}$ and in cycle number $N$, respectively. The dashed lines are empirical fits of $\Delta \theta(N)/\theta_0$ using a power law (Zhu et al., 2013; Cox et al., 2014).
\[
\frac{\Delta \theta (N)}{\theta_0} = \beta \times N^\alpha
\]  

(6.3)

where \( \alpha \) and \( \beta \) are dimensionless variables. The intercept \( \beta \) accounts for the initial rotation from \( \theta_0 \) to \( \theta_1 \) and was determined as \( \beta = 0.07 \) and \( \beta = 0.01 \) for \( \zeta_c = 0.1 \) and \( \zeta_c = -0.7 \), respectively, in the unidirectional tests with \( N \sim 10^6 \) reported in Zhu et al. (2018a). The magnitude of \( \beta \) will vary with stress level and was quantified in Zhu et al. (2018b) for prototype application. The exponent \( \alpha \) accounts for the rate of rotation accumulation and was shown to be independent of stress level (Zhu et al., 2018a, b). The response predicted by Equation (6.3) is also provided on Figure 6.11b using \( \alpha = 0.29 \) (Zhu et al., 2018a) and \( \beta=0.01 \) and \( \beta=0.07 \) for \( \zeta_c = -0.7 \) and \( \zeta_c = 0.1 \), respectively. As is to be expected, Equation (6.3) for \( \zeta_c = 0.1 \) provides a reasonable fit to the data over the first \( N = 45,000 \) cycles before the load direction changes, albeit that slight adjustments to \( \beta \) would improve the fit for individual tests. Over the remaining packets, Equation (6.3) using \( \zeta_c = 0.1 \) and \( \zeta_c = -0.7 \) bound the data, with the prediction for \( \zeta_c = 0.1 \) providing the best agreement, although the predictions become conservative for the higher changes in load direction. This contrasts with previous findings for monopiles in sand (Rudolph et al., 2014; Nanda et al., 2017), where prediction of the accumulated displacement based on unidirectional cyclic loading was substantially un-conservative for multidirectional loading.

### 6.3.5 Unloading stiffness

Figure 6.12 shows the variation in normalised unloading stiffness, \( k_N/k_1 \), with cycle number, \( N \), where \( k_1 \) and \( k_N \) are the unloading stiffnesses in the first cycle and cycle number \( N \), respectively. Over the course of the cyclic loading history, \( k_N/k_1 \) varies between 0.8 and 1.6, although the range of variation is similar (with \( k_N/k_1 \) between 1.0 and 1.5) during the initial \( N = 45,000 \) cycles of load applied at 0°, which was the same in all four tests. For comparison, the unloading stiffness in the unidirectional test reported...
in Zhu et al. (2018a) (with the same $\zeta_b$ and $\zeta_c$) was found to increase over 380,049 cycles to $k_N/k_1 \approx 1.4$.

6.3.6 Post-cyclic behaviour

The effect of multidirectional cyclic loading on the loading stiffness and ultimate capacity is examined in Figure 6.13, which compares the monotonic moment-rotation responses measured without and after unidirectional cyclic loading (Zhu et al., 2018a), with that measured at a loading direction of 0° after multidirectional cyclic loading. The results for test M1 (0° to 45°) was discarded as the soil was disturbed due to an operation error before the start of the post-cyclic monotonic test. In Figure 6.13 the rotation in the post-cyclic response is relative to that at the end of cycling to facilitate a more direct comparison with the initial monotonic response. The post-cyclic stiffness and ultimate capacity in the multidirectional tests are similar to those measured in the monotonic test without cyclic loading, which contrasts with the post-cyclic response after cyclic loading in the unidirectional test, in which the stiffness and ultimate capacity increased by 40% and 6%
respectively. These increases were attributed by Zhu et al. (2018a) to consolidation of the clay layer, which took place over \( N \approx 300,000 \) cycles of unidirectional loading, noting that the caisson and soil strength profiles were identical in the unidirectional and multidirectional tests. However, in the multidirectional tests the soil is loaded at \( 0^\circ \) periodically rather than continuously, such that the level of consolidation – and hence the stiffness and capacity – is expected to be lower. Support for this conclusion comes from Test M3, in which half of the load cycles were applied in a near-opposing load direction to \( 0^\circ \), and in which the stiffness in the post cyclic monotonic test is almost identical to that in the initial monotonic test.

![Figure 6.13 Monotonic response before and after cyclic loading.](image)

**6.4 Concluding remarks**

The multidirectional test results presented here represent the first available evidence of the effect of changes in lateral cyclic load direction on the response of a suction caisson.
Chapter 6. The response of suction caissons to multidirectional lateral cyclic loading in sand over clay

The tests were performed with the suction caisson installed in sand over clay under cyclic lateral loading acting at an eccentricity.

Changes in loading direction were found to influence the caisson out-of-verticality (or rotation) and foundation-soil stiffness, as expected. However, the evolution of caisson rotation under multidirectional cyclic loading was found to be bounded by relevant unidirectional test results and established predictions, with changes in load direction of up to 90º resulting in similar response as corresponding unidirectional loading and larger variations in load direction leading to less accumulated rotation. This is unlike previous studies for monopiles in sand that show larger accumulated displacement under multidirectional cyclic loading than unidirectional loading.

The effect of changes in loading direction on unloading stiffness is slight and the variation in unloading stiffness under multidirectional loading is similar to that measured in a unidirectional tests for the same cyclic load magnitude and symmetry characteristics. The post-cyclic loading stiffness and ultimate capacity in the initial loading direction were found not to be significantly affected by multidirectional cyclic loading. This is unlike unidirectional loading where the consolidation of the clay layer increases both the stiffness and capacity.

This conclusions reached in this Chapter improves the understanding of suction caisson behaviour under multidirectional cyclic loading, which could be extended with further study considering other soil conditions and more realistic loading characteristics (e.g. combined changes of cyclic magnitude, symmetry and direction) to establish recommendations for design.

6.5 References

Chapter 6. The response of suction caissons to multidirectional lateral cyclic loading in sand over clay


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Chapter 6. The response of suction caissons to multidirectional lateral cyclic loading in sand over clay


CHAPTER 7. CONCLUSIONS AND IMPLICATIONS

7.1 Summary

This thesis has considered the response of a monopod suction caisson in sand over clay layered soils when subjected to the type of long-term and multidirectional cyclic loading experienced by an offshore wind turbine over its design life. The focus is the effect of cyclic loading on 1) accumulated rotation (this is limited for the serviceability limit state design as this wind turbine system is sensitive to out-of-verticality) and 2) foundation stiffness (stiffness changes need to be considered in the fatigue limit state design as this affects system natural frequency). These are important aspects in the foundation design of offshore wind turbines.

The study started with a series of numerical analyses investigating suction caisson responses under monotonic loading. The analyses considered various soil profiles (sand, clay and sand over clay with thickness of upper sand layer 0.25, 0.5, 0.75, 1.0 and 1.5 times the skirt length, i.e. \( H_{sand}/L = 0.25, 0.5, 0.75, 1.0 \) and 1.5) and provided a basis for understanding the effect of soil layering on the performance of the suction caisson. The caisson behaviour under cyclic loading was investigated through an experimental study, involving two series of single-gravity testing programmes and a centrifuge testing campaign. The first series of single-gravity tests (Chapter 4) were carried out with typically one million of load cycles in sand, clay and sand over clay with \( H_{sand}/L = 0.5, 1.0 \) and 1.5. The second series of single-gravity tests (Chapter 6) was carried out in the sand over clay profile \( H_{sand}/L = 0.5 \) with the cyclic load direction changed between 0° and 135° in various patterns. These tests provided a basis for understanding caisson performance under long-term and multidirectional cyclic loading. The centrifuge tests
(Chapter 5) considered suction installation, realistic stress levels and probable drainage conditions, and provided a means of assessing the long-term response at field scale by complementing with the relatively short-term cyclic response with the long-term response measured in the single-gravity tests.

7.2 Main findings

7.2.1 Accumulated rotation

This thesis has made a particular contribution to the understanding of the evolution of accumulated rotation under long-term cyclic loading. This has been qualified through the first suite of single-gravity tests carried out with up to one million load cycles (Chapter 4), and quantified through the centrifuge tests (Chapter 5) for prototype scale and realistic conditions. The effect of cyclic loading on accumulated rotation is described in detail with respect to various soil profiles (sand, clay and sand over clay with variation in sand layer thickness, i.e. $H_{\text{sand}}/L = 0.5, 1.0$ and $1.5$), cyclic loading magnitudes and symmetries (one-way or two-way). The main findings are summarised here:

- Differences in caisson rotation are evident in the first cycle and are affected by installation method, drainage response, soil profile and cyclic load magnitude and symmetry. The centrifuge tests reveal that caisson rotation in sand is higher when the response is drained than partially drained following jacked installation. Jacked installation of caissons in sand, as performed in most tests to date, appears to lead to lower rotation in comparison to suction installation. The effects of loading symmetry on caisson rotation in sand and $H_{\text{sand}}/L = 0.5$ are distinct; one-way cyclic loading results in a larger rotation than two-way cyclic loading (which contrast with previously published studies for sand). Caisson rotation for $H_{\text{sand}}/L = 0.5$ increases with the magnitude of the cyclic load, but is similar when normalised...
by the rotation in the first cycle, as observed from both single-gravity tests (jacked installation) and centrifuge tests (suction installation).

- Once normalised by the first cycle, the long-term trend of accumulated caisson rotation appears to be unique according to both centrifuge and the equivalent single-gravity tests. This comparison provides support to the approach of utilising the centrifuge and single-gravity databases collectively to inform the expected whole-life response at field scale.

- The evolution of caisson rotation with cycle number can be described using a simple power relationship, which forecasts caisson rotation over a design number of cycles. The required parameters relevant to the prediction (including effects of installation method, drainage response, soil profile, cyclic load magnitude and symmetry) are provided based on the database established from the model tests reported in this thesis.

- The mechanism that controls caisson rotation depends on the soil profile. Caisson rotation in sand over clay profiles with a sand layer thickness larger than the skirt length \((H_{\text{sand}}/L \geq 1)\) appears to be similar to single-layer sand. This is consistent with the observation from numerical analyses (reported in Chapter 3) that the lateral loading of a caisson mainly mobilises the soil above skirt tip level, such that the caisson performs similarly in sand and \(H_{\text{sand}}/L \geq 1\). In these profiles, caisson rotation continues to increase, though at a decreasing rate, over one million load cycles. In contrast, caisson rotation in clay appears to stabilise after the excess pore pressure (that develops as a result of the cyclic loads) dissipates, improving the strength of the clay. For sand over clay with \(H_{\text{sand}}/L = 0.5\), caisson rotation is close to clay (for the same cyclic load scenario), indicating a consolidation dominated performance. This is because pore pressure
accumulation during cyclic loading is negligible in sand, but significant when there is an underlying clay layer.

7.2.2 Foundation-soil stiffness

Similar to the accumulated rotation, this thesis describes in detail the change of foundation-soil stiffness under long-term cyclic loading as assessed from observations from the first series of single-gravity tests and the centrifuge tests. The main findings are summarised here:

- Both the single-gravity and centrifuge tests reveal that cyclic loading typically leads to a stiffer caisson response, resulting from the beneficial effect of consolidation in clay (in clay and sand over clay with $H_{sand}/L = 0.5$) and densification in sand (in sand and sand over clay with $H_{sand}/L \geq 1$).

- The increase in foundation-soil stiffness due to densification in sand is more immediate and modest compared with the increase due to consolidation in clay. The long-term stiffness appears to stabilise, following densification of the sand and consolidation of the clay.

- Once normalised by the first cycle, the evolution of foundation-soil stiffness for sand is almost identical in single-gravity and centrifuge model tests. However, the stiffness increase for sand over clay with $H_{sand}/L = 0.5$ is more moderate and steady in single-gravity tests than in centrifuge tests, which indicates the potential effect of stress level on clay consolidation and warrants further attention.

- Considering the evolution of foundation-soil stiffness as the effect of cyclic loading, a range of stiffnesses and their effect on the natural frequency of the offshore wind turbine need to be considered in the design.
7.2.3 Effects of variation in load direction

A further major contribution of this thesis is demonstration of caisson performance due to a variation in load direction (or multidirectional cyclic loading). This was assessed through the second series of single-gravity model tests (Chapter 6), which were conducted in sand over clay with $H_{\text{sand}}/L = 0.5$ involving four load directions, i.e., $0^\circ$, $45^\circ$, $90^\circ$ and $135^\circ$, with $0^\circ$ the initial load direction. Three tests investigate the effect of change between the initial and one secondary load direction and the other one investigates the effect of a successive change in load direction from $0^\circ$ to $45^\circ$, $90^\circ$ and $135^\circ$. The main findings are summarised here:

- Changes in load direction are found to influence the caisson rotation, as expected. However, the range of rotation is bounded by relevant unidirectional test results and established predictions (reported in Chapter 4), with changes in load direction of up to $90^\circ$ resulting in a similar response as corresponding unidirectional loading, and larger variations in load directionality leading to less accumulated rotation. This contrasts with previous studies on monopiles in sand, that show larger accumulated pile head deformation under multidirectional cyclic loading than unidirectional loading.

- The unloading stiffness is not constant over the multidirectional cyclic loading history. However, changes in load direction do not increase the range of unloading stiffness beyond the variation evident in results from unidirectional cyclic loading. The post-cyclic loading stiffness and capacity in the initial load direction are not to be significantly affected by multidirectional cyclic loading for the number of cycles ($N = 360,000$) applied.
7.3 Implications

Based on the databases established in this work, this thesis provides an opportunity to estimate the rotation and stiffness of a prototype caisson (aspect ratio $L/D = 0.5$) in sand over clay seabeds over the lifetime of an offshore wind turbine, given the specific soil profile, cyclic loading amplitude and symmetry. This is shown by the example here.

Take an example of offshore wind farm site in Dogger Bank, North Sea, where the soil conditions typically comprise a surficial sand layer overlying stiff clay with the sand layer thickness varying from several to tens of meters (Forewind 2011, 2013). For example for a sand over clay seabed with sand layer thickness half of the skirt length, a fatigue limit state would consider a moderate cyclic load ($\zeta_b = 0.4$) and $N = 10^6$ loading cycles (LeBlanc et al., 2010). According to the parameters provided in Chapter 5, caisson rotation then becomes $\theta = \theta_0 (1 + \beta \times N^\alpha) = 0.032(1 + 0.17 \times (10^6)^{0.29}) = 0.3^\circ$ for two-way loading ($\zeta_c = -0.7$) and $\theta = 0.029(1 + 0.67 \times (10^6)^{0.29}) = 1.1^\circ$ for one-way loading ($\zeta_c = 0.1$). On the basis of this simple calculation, the one-way loading would exceed the DNV (2016) rotation limit of $0.5^\circ$. Note that it is a conservative estimation as this neglects 1) the stabilisation of the response due to consolidation in the underlying clay layer, 2) not-in-service period of a turbine (e.g. no wind conditions or turbine is parked and feathered), and 3) the reduction of rotation with changes in load direction larger than $90^\circ$. When changes in load direction are larger than $90^\circ$, the caisson rotation can be assessed from the prediction of unidirectional two-way cyclic loading cases, e.g. rotation from unidirectional loading with $\zeta_c = -0.7$, which appears to bound the rotation when load direction changes to $135^\circ$. In contrast to the sand over clay seabed, the same calculations for one-way loading scenario in sand would lead to a more moderate rotation of $0.3^\circ$ (i.e. using $\beta = 0.45$ and $\theta_0 = 0.013^\circ$).
A general trend of increase in foundation-soil stiffness is evident during the process of cyclic loading; this effect on the natural frequency of the offshore wind turbine needs to be considered in the design. The increase in stiffness in sand (approximately 50% as observed from all tests here) is more immediate and modest compared with the increases (up to 175%) in sand over clay with sand layer thickness being half of the skirt length. To illustrate this effect, when the foundation stiffness calculated by Arany et al. (2016) for ten offshore wind farms (involve typically dense sands and stiff clays similar to this study) is allowed to increase by 50% and 175%, the system natural frequency (calculated using the Arany et al. (2016) approach) increases by up to 3.5% and 7% respectively. Assuming the turbine is designed as a soft-stiff system, such a change would have the effect of moving the system natural frequency away from the 1P forcing frequency zone but close to the 3P zone (see Figure 1.5, Chapter 1).

### 7.4 Future study

The discussions in Section 7.3 are simple examples to illustrate the findings of this research. It is necessary to verify these findings with respect to more general conditions. The following recommendations for future research are made:

- **Investigate the caisson responses under more general soil conditions.** The established database for assessing caisson responses is mainly specific to dense sand over stiff clay. However, soil conditions (e.g. soil layering and soil properties) in reality could vary significantly with the sites. Therefore, similar approach to this study could be applied in more general soil conditions to extend the current database.

- **Consider more complex loading conditions.** Loading transferred to a suction caisson supporting an offshore wind turbine in the field is complex, but the
findings in this thesis do not account for the combined variation in load magnitude, one-way, two-way or asymmetric two-way cyclic nature, eccentricity and load direction over the design life. It would be worthwhile to investigate these effects in the future, with a view to establish recommendations for geotechnical design.

- Develop uncertainty prediction methods. The established database provides direct (or deterministic) predictions on the caisson rotation and stiffness over the lifetime of a wind turbine. Considering the uncertainty of soil conditions over the very large seabed footprint of a wind farm and uncertainty on the loading characteristics, probabilistic analyses may prove to be a useful tool for design.

### 7.5 References


