Centrifuge study on the cyclic performance of caissons in sand

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Suction caissons are currently considered as an alternative to monopile foundations for met masts and offshore wind turbines. This paper presents the results of a series of centrifuge tests conducted on cyclically loaded suction caissons in very dense dry sand. Two representative caisson foundations were modelled at a 1200 scale in a geotechnical centrifuge and were subjected to a number of different cyclic loading regimes, for up to 12 000 cycles, both of which add to previous data sets available in the literature. During each test, changes in stiffness, the accumulation of rotation and settlement of the system were measured. It was found that the rotational caisson stiffness increased logarithmically with the number of loading cycles, but to a much lower extent than previously reported for monopiles. Similarly the accumulation of rotation was also observed to increase with number of cycles and was well described using a power relationship. An aggregation of rotation was also observed during two-way tests and is believed to be caused by the initial loading cycles that create a differential stiffness within the local soil. Predictions were then made as to the behaviour of a prototype structure based upon the observed test results and established influence parameters.

1. Introduction

Most of the current generation of offshore wind turbines are founded on so-called monopile foundations; these are essentially large-diameter piles that are hammered into the seabed. Monopile foundations, by their very nature, are expensive owing to the pile-driving equipment and support vessels needed. In addition, the noise generated by piling requires environmental mitigation. The cost of such foundation arrangements alone makes up around 20% of an offshore
wind project (Carter, 2007; DTI, 2001, 2007; Kühn et al., 1998; Larsen et al., 2005). An alternative foundation for offshore wind turbines is the suction caisson, which is a large-diameter steel bucket foundation that is embedded in the local seabed using suction (see Figure 1). Suction caissons have been widely used by the offshore oil and gas industry, particularly for anchoring large floating facilities (Andersen et al., 2005), but also as footings for jacket and jack-up structures.

Suction caissons have only seen use in a few dynamically sensitive structures, such as wind turbines (Frederikshavn wind turbine; Liingaard (2006)) and met masts (Horns Rev 2 offshore wind farm; LeBlanc (2009a)). Offshore wind turbines have stringent serviceability limit states, defining the total rotation and modal frequencies of the structure. This is of particular concern as the mechanical components exhibit a reduced design life when subjected to misalignment or dynamic loading. Accordingly (as in the case of the Thornton Bank wind park) the total structural rotation is limited to 0–25˚ (Peire et al., 2009) and the modal frequencies are required to differ by 10% to that of the periodic wind, wave, current and vortex loads (Det Norske Veritas, 2002). An example of the dynamic forces that an offshore turbine will be subjected to and the design solutions, where the first modal frequency of the turbine can be safely designed, is illustrated in Figure 2. These solutions are often referred to as soft–stiff and stiff–stiff zones; these describe the regions between the 1P and 3P frequencies and the region above the 3P frequency, respectively, where P is the rotational frequency of the turbine. From a design perspective, the risk is that during long-term cyclic loading the foundation stiffness may change, causing the natural frequency of the system to approach either the 1P or 3P frequencies, producing resonance.

Offshore wind turbines are very recent structures, with the first experimental offshore turbines being constructed in 1991 and large-scale projects being developed from the early 2000s (Zaaijer, 2009). As a result there are scant operational data concerning the long-term behaviour of an offshore turbine system. Accordingly a significant amount of recent research has considered the performance of wind turbine foundations (particularly monopiles) when subjected to long-term cyclic loading (Bhattacharya et al., 2012; LeBlanc et al., 2010a). Much of the work on suction caissons has focused on scaled tests, modelling the monotonic caisson response (Feld, 2001; Houlbsy et al., 2005; Villalobos, 2006), and the behaviour of a caisson subjected to a series of cyclic loading regimes (Byrne, 2000; Villalobos, 2006; Zhu et al., 2013). Such tests have been conducted under 1g conditions and require verification at stress levels more typical in the prototype. To address this, some limited tests have been conducted in saturated sands at a prototype scale (Kelly et al., 2006; Liingaard, 2006) and in a centrifuge (Lu et al., 2007; Senders, 2008; Zhang et al., 2007). These were, however, only conducted for a limited number of cycles and were primarily concerned with the accumulation of pore water pressure under extreme loading situations. Despite the quality and rigour of such investigations, there is still a limited amount of knowledge surrounding the long-term applicability of caisson foundations at prototype scale.

The contribution of this paper is a database of centrifuge results from tests conducted in very dense dry sand that focused on the accumulation of rotation and the stiffness changes with increasing number of cycles.

2. Centrifuge test programme
All tests in this programme were conducted in a geotechnical centrifuge to ensure that the stress–strain behaviour (and hence geotechnical behaviour) in the model were the same as in the equivalent prototype (Gaudin et al., 2010). The tests in this investigation were conducted at 200g using the beam centrifuge facility at the Centre for Offshore Foundation Systems at The University of Western Australia. A description of the centrifuge and its capabilities is detailed elsewhere (Randolph et al., 1991).
To ensure that the loads subjected to the model caissons are representative of the forces expected at full scale, a number of prototype cases were analysed. The behaviour of these full-scale structures was then rationalised in the form of non-dimensional groups characterising the key aspects of the systems. The non-dimensional groups considered in this investigation, shown in Table 1, describe the relative flexibility of the model caissons (group 1), the aspect ratio (group 2), the soil stress (groups 3–5) and the strain properties (group 6) of the foundation as a whole. The derivation and explanation of the groups has been covered before and can be found in Doherty et al. (2005), Cox and Bhattacharya (2012) and Foglia and Ibsen (2013).

Three dynamically sensitive offshore structures have been founded on suction caissons: the 3 MW Frederikshavn quasi-offshore wind turbine (Liingaard, 2006), the 5 MW Wilhelmshaven offshore wind turbine (LeBlanc, 2009a) and the Horns Rev 2 met mast (LeBlanc, 2009b). Targeted non-dimensional scaling values have been determined from these structures and are detailed in Table 1, with full notation provided at the beginning of the paper. A description of the material properties, geometry and the load reference point utilised in the non-dimensional groups is provided in Figure 3.

It was also necessary to scale the corresponding caisson response to extract the relevant data; this was achieved using the non-dimensional groupings suggested by Kelly et al. (2006) (and presented in Table 2), which permits comparisons between the results of this study with those conducted at different scales.

In this investigation two model caissons, herein referred to as caisson C2 and caisson C3, were considered to reflect the prototype structures described in Table 1. The aspect ratios (group 2) of the two caissons were $Z/D = 1$ and $0.5$ for caissons C2 and C3 respectively. Each model was designed such that the caisson flexibility (group 1) was $9.35$. This was achieved by choosing a diameter of $80$ mm and manufacturing the caissons from aluminium ($E_s = 70$ MPa), with a wall thickness $t = 1.4$ mm. Assuming a shear modulus, $G_r = 130$ MPa (Hardin and Drnevich, 1972) and an average effective unit weight, $\gamma' = 17.4$ kN/m$^3$, a strain multiplier (group 6) of 476 was achieved, slightly higher than but comparable to the Frederikshavn and Horn Rev 2 caissons. The dimensionless vertical capacity (group 3) was within the quoted offshore range in Table 1, with values of $0.74$ for the C2 caisson and $0.69$ for the C3 caisson. The two model caissons are shown in Figure 4 and measurement details for each caisson are detailed in Table 3.

Tests were conducted in a centrifuge strong box with internal dimensions $650 \times 390 \times 300$ mm (length $\times$ width $\times$ depth) filled with very dense sample of ‘super fine silica sand’ (with $e_{\text{max}} = 0.747$, $e_{\text{min}} = 0.449$, $D_{50} = 0.15$ mm and $\varphi_{\text{ey}} = 31^\circ$ (Liu and Lehane, 2012; O’Loughlin and Lehane, 2003)), and as such the particle size to foundation diameter ratio was greater than...
Each sample was prepared in an identical manner using the sand pluviation technique to give final sample heights of approximately 200 mm. Global measurements of the sample volume and mass gave unit weights in the range 17.5–17.8 kN/m³ (Dr = 86.5 ± 2.5%) at 1g. The available plan testing area in the strongbox permitted three tests to be conducted in each sample, allowing a minimum edge-to-edge clearance of two diameters between test locations. Based on the radial consolidation solution proposed by Randolph and Wroth (1979), it was found that for a range of typical loading frequencies and permeabilities, a prototype foundation would predominantly behave in a drained manner, with partially drained behaviour seen under low-permeability conditions. Accordingly the sand medium remained dry throughout, replicating a fully drained case.

All caissons in this investigation were installed by jacking using an actuator at 1g. Installing a foundation at 1g causes the sand displaced during jacking to dilate more, increasing the corresponding soil plug heave, in comparison to a prototype case (Tran, 2005). This additional heave will have the effect of reducing the observed caisson capacity; as only the transient behaviour is of interest, the installation was not modelled in this investigation. Load was applied to the caisson system using a pivoted loading arm (Dyson, 1999) at a fixed eccentric height on the model tower to replicate the desired horizontal and moment loads (such that the eccentric height, \( h = M/H \)). Using a combined axial and moment load cell, load control could be achieved in both the horizontal axis (by way of axial load control) and the vertical axis (by way of bending load control). As only a horizontal load was desired, the vertical axis of the actuator was continuously adjusted through a feedback loop to ensure that the moment on the load cell maintained zero and that no change in vertical load was applied to the caisson system.

The system response was measured using the motor encoders on the horizontal and vertical axes of the actuator, two laser potentiometers mounted perpendicular to the axis of forcing and the combined axial and moment load cell. The resulting experimental set-up is illustrated schematically in Figure 5.

<table>
<thead>
<tr>
<th>Group No.</th>
<th>Description</th>
<th>Frederikshavn</th>
<th>Wilhelmshaven</th>
<th>Horns Rev 2 Met mast</th>
<th>Test models</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E_s t / G_D )</td>
<td>Caisson flexibility</td>
<td>10-10</td>
<td>7-89</td>
<td>6-73&lt;sup&gt;a&lt;/sup&gt;</td>
<td>9-20</td>
</tr>
<tr>
<td>( Z / D )</td>
<td>Caisson aspect ratio</td>
<td>0-50</td>
<td>1-00</td>
<td>0-50</td>
<td>0-50/1-00</td>
</tr>
<tr>
<td>( V / \gamma D^3 )</td>
<td>Vertical caisson capacity</td>
<td>0-48</td>
<td>0-43</td>
<td>0-99</td>
<td>0-69/0-76</td>
</tr>
<tr>
<td>( H / \gamma D^3 )</td>
<td>Horizontal caisson capacity</td>
<td>0-05</td>
<td>0-05</td>
<td>0-06</td>
<td>Varies</td>
</tr>
<tr>
<td>( M / \gamma D^3 )</td>
<td>Moment caisson capacity</td>
<td>0-39</td>
<td>0-28</td>
<td>0-04</td>
<td>Varies</td>
</tr>
<tr>
<td>( G_s / \gamma D )</td>
<td>Strain multiplier (considers relative stress–strain behaviour)</td>
<td>436-7</td>
<td>392-9</td>
<td>436-7</td>
<td>475-9</td>
</tr>
</tbody>
</table>

<sup>a</sup>Thickness calculated from DNV guidelines (DNV-RP-C202 Det Norske Veritas (2010))

Table 1. Prototype non-dimensional groupings (Det Norske Veritas, 2010)
The following experimental procedures were adopted for each test run.

(a) A centrifuge strong box filled with a sample of very dense silica sand was located on the swinging platform of the centrifuge.

(b) Each sample was spun to 200 g and a cone penetrometer test (CPT), was conducted to characterise the sample. In this case a 7 mm diameter cone penetrometer was penetrated at 1 mm/s.

(c) The sample was spun down to 1 g and the model caisson was carefully jacked into the sand at a rate of 0.25 mm/s at 1 g using the actuator, ensuring verticality, until the caisson was fully installed. Subsequently the air vent in the caisson lid was sealed.

(d) The model caisson was then connected to the dual axis actuator by way of the pivoted loading arm.

(e) The centrifuge was then spun up until an acceleration of 200 g was acting at the mid-depth level of the caisson.

(f) The selected loading regime was then applied to the model and the corresponding response of the system was recorded.

(g) At the end of the experiment the centrifuge was spun down to 1 g and steps (c)–(f) were repeated for further tests at different sites in the strong box.

In this investigation two separate loading regimes were applied to the model system. Each of these regimes was designed to test the changing serviceability state of the caisson system. These loading regimes were as outlined below.

(a) Push-over tests [P]: these tests allowed the quasi-static stiffness of the caisson to be assessed by steadily displacing the model at an eccentric height and measuring the resultant load.

(b) Cyclic tests [C]: the loading in this regime was intended to closely match what would be applied to an offshore wind turbine (a large static wind force augmented by a varying wind and wave load). In these experiments the varying component was assumed to be a sine wave. By adjusting the static and varying components of the loading regime, different load situations could be developed. Using the definitions proposed by LeBlanc et al. (2010b) the load orientation and magnitude can be defined as

\[ \xi_b = \frac{M_{\text{max}}}{M_R} \]

\[ \xi_c = \frac{M_{\text{min}}}{M_{\text{max}}} \]

where \( \xi_b \) specifies the load level as a ratio of the maximum moment and the static moment capacity, and \( \xi_c \) represents the relative directionality (where \( \xi_c = -1 \) represents a symmetrical two-way loading and \( \xi_c = 0 \) represents a purely one-way regime); this is illustrated in Figure 6.

Each experiment was subsequently identified with a unique label describing the test that was conducted. This label denotes

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**Table 2. Summary of experimental non-dimensional groups after Kelly et al. (2006)**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Normalised expression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angular rotation ( [\theta] )</td>
<td>[ \frac{P_a}{\gamma D} ]</td>
</tr>
<tr>
<td>Vertical displacement ( [\psi] )</td>
<td>[ \frac{W}{D} \left[ \frac{P_a}{\gamma D} \right]^{0.5} ]</td>
</tr>
<tr>
<td>Unload stiffness in each cycle ( [k] )</td>
<td>[ \left[ (\gamma P_a)^{0.5} D^{2.5} \right] ]</td>
</tr>
</tbody>
</table>

---

Figure 4. Model caisson used in the centrifuge tests: (a) C3 caisson \((Z/D = 0.5)\) and (b) C2 caisson \((Z/D = 1.0)\).
the size of caisson used, the adopted loading regime and the test number. For example, the second test under a cyclic loading using a C3 caisson would be labelled as C3-C-2.

3. Analysis
From the data recorded during each test it was possible to determine a number of basic responses of the system. Characteristics such as the instantaneous point of rotation, the moment load, the vertical settlement, the change and accumulation of rotation of the caisson system were easily calculated.

Utilising the moment rotation plots it was possible to calculate the rotational stiffness of the foundation throughout the experiment. This was achieved by identifying the maximum and minimum loads and the corresponding rotations within each loading cycle. By considering the difference between the moment and rotation, the unloading rotational stiffness could be calculated in a similar manner to LeBlanc et al. (2010a) and Zhu et al. (2013). This methodology is illustrated in Figure 7.

4. Results
The experimental programme considered here encompasses two push-over tests and thirteen cyclic tests using two different caissons. A summary of the tests conducted in this investigation is provided in Table 4.

4.1 Cone penetration tests (CPTs)
Within each sample two CPTs were conducted to characterise the sample. Utilising these CPT records the repeatability of the sample preparation method could be assessed. From the CPT data it was noted that there was reasonable consistency between the sand samples; the variation between samples can be attributed to minor inhomogeneity. The cone resistance with depth for all these tests is shown in Figure 8.

4.2 Installation
Each caisson was installed in the manner described previously. Full installation was defined as the point at which the caisson soffit contacted the soil plug, identified by a rapid increase in the installation load. These installation records were consistent with an embedment of 88–90% of the total skirt length. A series of typical installation records is given in Figure 9.

<table>
<thead>
<tr>
<th>ID</th>
<th>Description</th>
<th>Diameter, D: mm</th>
<th>Skirt length, Z: mm</th>
<th>Thickness, t: mm</th>
<th>Mass at 1g: kg</th>
</tr>
</thead>
<tbody>
<tr>
<td>C2</td>
<td>5MW model, 1:200</td>
<td>80.0</td>
<td>80.0</td>
<td>1.4</td>
<td>0.68</td>
</tr>
<tr>
<td>C3</td>
<td>5MW model, 1:200</td>
<td>80.0</td>
<td>40.0</td>
<td>1.4</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Table 3. Details of centrifuge model caissons
4.3 Pushover

A pushover test \((M/HD = 3-3)\) was conducted for both caisson geometries by displacing the caisson tower at an eccentric height at a constant rate of 0-05 mm/s. The load measured during the pushover test allowed calculation of the static unit rotation \((\delta_\theta)\) to be made; this is the structural rotation observed when the maximum load in a cyclic series is applied to the system under monotonic conditions. Utilising the relationships described in Table 1 and Table 2 it was possible to non-dimensionalise the results, thus presenting the behaviour in terms of the load level \([M/\gamma' D^3]\) and rotational strain \([\delta\theta/\gamma' D]\). These results are shown in Figure 10 alongside the yield load calculated in the same manner as Zhu et al. (2013).

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Percentage installed: %</th>
<th>Number of cycles: N</th>
<th>(H/\gamma' D^3)</th>
<th>(M/\gamma' D^4)</th>
<th>(\bar{c}_{\text{re}})</th>
<th>(\bar{c}_{\text{sc}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>C3-P-1</td>
<td>90-2</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>0-04</td>
<td>0-12</td>
</tr>
<tr>
<td>C3-C-1</td>
<td>89-1</td>
<td>71</td>
<td>—</td>
<td>—</td>
<td>0-06</td>
<td>0-18</td>
</tr>
<tr>
<td>C3-C-2</td>
<td>88-8</td>
<td>1877</td>
<td>0-08</td>
<td>0-24</td>
<td>0-06</td>
<td>0-20</td>
</tr>
<tr>
<td>C3-C-5</td>
<td>89-3</td>
<td>690</td>
<td>0-02</td>
<td>0-06</td>
<td>0-06</td>
<td>0-20</td>
</tr>
<tr>
<td>C3-C-6</td>
<td>90-1</td>
<td>9329</td>
<td>0-04</td>
<td>0-12</td>
<td>0-06</td>
<td>0-20</td>
</tr>
<tr>
<td>C3-C-7</td>
<td>89-7</td>
<td>2209</td>
<td>0-04</td>
<td>0-12</td>
<td>0-06</td>
<td>0-20</td>
</tr>
<tr>
<td>C2-P-3</td>
<td>88-3</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>0-99</td>
</tr>
<tr>
<td>C2-C-1</td>
<td>89-0</td>
<td>1555</td>
<td>0-16</td>
<td>0-51</td>
<td>0-54</td>
<td>0-54</td>
</tr>
<tr>
<td>C2-C-2</td>
<td>90-2</td>
<td>1587</td>
<td>0-16</td>
<td>0-51</td>
<td>0-54</td>
<td>0-54</td>
</tr>
<tr>
<td>C2-C-3</td>
<td>89-2</td>
<td>2367</td>
<td>0-24</td>
<td>0-77</td>
<td>0-81</td>
<td>0-81</td>
</tr>
<tr>
<td>C2-C-4</td>
<td>89-9</td>
<td>2597</td>
<td>0-24</td>
<td>0-75</td>
<td>0-79</td>
<td>0-79</td>
</tr>
<tr>
<td>C2-C-5</td>
<td>87-8</td>
<td>2187</td>
<td>0-24</td>
<td>0-76</td>
<td>0-80</td>
<td>0-80</td>
</tr>
<tr>
<td>C2-C-6</td>
<td>89-6</td>
<td>1962</td>
<td>0-08</td>
<td>0-25</td>
<td>0-26</td>
<td>0-26</td>
</tr>
<tr>
<td>C2-C-7</td>
<td>90-3</td>
<td>12 121</td>
<td>0-16</td>
<td>0-52</td>
<td>0-55</td>
<td>0-55</td>
</tr>
<tr>
<td>C2-C-8</td>
<td>88-9</td>
<td>6985</td>
<td>0-24</td>
<td>0-73</td>
<td>0-77</td>
<td>0-77</td>
</tr>
</tbody>
</table>

Table 4. Centrifuge tests parameters
It is evident from Figure 10 that the increased caisson skirt length has a significant effect on the moment capacity of the system. Furthermore the failure modes are different: caisson C3 displays a residual strength similar to that of the peak load, whereas caisson C2 continues to gain strength. This observed ductility in caisson C2 must be a direct result of changes in the failure mechanism owing to the increased embedment.

Furthermore it is possible to evaluate the change in non-dimensional stiffness with rotation level, considering secant values from the pushover data in Figure 10; this degradation is plotted in Figure 11. Figure 11 shows a higher stiffness for caisson C2 (also evident from the steeper slope of the push-over data in Figure 10), and is to be expected given the longer skirt length. The non-dimensional stiffness remains tolerably constant over the linear portion of the pushover tests (up to approximately $\theta = 10^{-2}$), before degrading with increasing rotation, qualitatively similar to plots of shear modulus against strain level (Hardin and Drnevich, 1972).

### 4.4 Unloading stiffness

Under cyclic loading the caisson stiffness was assessed as previously described, and the resulting data were non-dimensionalised using the groups proposed by Kelly et al. (2006) and plotted against cycles number in Figure 12. Anomalous behaviour was noted in the first few cycles of test C2-C-4 data set as the caisson was bedding in, and accordingly this behaviour has been omitted from all analyses. It should be noted that during the C2-C-8 test series, there was a temporary issue with the data acquisition system leading to a gap in the data.

It is evident from Figure 12 that the evolution of stiffness with increasing cycle number is quite erratic (as also observed by LeBlanc et al. (2010a) during cyclic loading of monopiles); this is not unexpected as the displacement amplitudes considered are very small. However, for tests involving long-term cyclic loading (i.e. $N > 1000$) an increase or maintenance of the rotational stiffness was observed. Following similar work by Sawicki and Swidzinski (1989) on the stiffness of sand under triaxial loading conditions and by Li et al. (2010) on the stiffness of a model pile system, a logarithmic relationship was used to express the changing stiffness of the foundation. Utilising the following relationship the non-dimensional change in stiffness can be described with increasing number of loading cycles:

\[
\frac{M}{(\gamma D^3)^0.5} = \ln(1000) \cdot \frac{\theta}{\theta_0} + 1
\]
3. \( \tilde{k} = K_0 + A_k \ln(N) \)

where \( \tilde{k} \) is the instantaneous non-dimensional foundation stiffness, \( K_0 \) is the characteristic non-dimensional foundation stiffness and \( A_k \) is the stiffness change parameter. Despite the erratic evolution of the stiffness, Equation 3 was mapped to the data such that the long-term stiffness was adequately described (indicated by the dotted lines on Figure 12); this was achieved using a constant stiffness parameter \( (A_k) \) of 1·52 and values of \( K_0 \) which varied for each test as summarised in Table 5.

Figure 12 and Table 5 show that caisson C2 is approximately twice as stiff as caisson C3, which is also evident from the stiffness derived from the pushover data and shown on Figure 11, and is attributed to the greater skirt depth. The characteristic stiffness \( K_0 \) appears to be fairly consistent between tests, the load level \( (\zeta_h) \) and load directionality \( (\zeta_v) \) appear to have an effect on the resulting behaviour; however, it is minimal. This is not unexpected as all tests are conducted within or close to the elastic range of the soil. A comparison of Figure 12 with Figure 11 suggests that a reasonable approximation of the characteristic stiffness \( K_0 \) can be obtained from the pushover test (considering rotation levels).

The stiffness change parameter, \( A_k = 1·52, \) as determined for the suction caisson data reported here, is much lower than \( A_k = 8 \) as observed by LeBlanc et al. (2010a) for model tests on monopiles. This behaviour can be attributed to the tests in this instance being conducted at 200g with a high relative density (\( ~85\% \)), compared to the test of LeBlanc et al. (2010a) conducted at 1g in a loose sand sample (\( ~35\% \)). It is not unreasonable to see a lower rate of stiffness change in this instance as there is a lower potential for the sand to densify.

4.5 Rotation of the caisson

For each loading condition the accumulation or retention of rotation was assessed. This was achieved by considering the mean rotation observed during a single loading cycle. In all cases an initial rapid accumulation of rotation was observed which slowed down with additional loading cycles (and increased foundation stiffness). The resulting absolute change in rotation could then be normalised with respect to the static unit rotation \( (\theta_s) \) and plotted against the number of cycles, as shown in Figure 13.

A number of experimental investigations on monopiles and caissons have developed relationships to describe the accumulation of rotation with number of cycles. A logarithmic relationship has been proposed by Lin and Liao (1999), Verdure et al. (2003) and Li et al. (2010) for monopiles whereas a power relationship was proposed by Long and Vanneste (1994), LeBlanc et al. (2010a) and Klinkvort et al. (2010) for monopiles, and by Zhu et al. (2013) for caissons. The data collected in this study were best fitted with a power relationship of the form

\[ \frac{\Delta \theta(N)}{\theta_s} = TN^x \]

As fewer tests were conducted here than in previous investigations (owing to the limited testing window available), the two loading parameters \( \zeta_h \) and \( \zeta_v \) (used by LeBlanc et al., 2010a) were combined and considered as a single variable \( T \). The data were best fitted using \( x = 0·3 \) which is comparable to \( x = 0·31 \) reported by LeBlanc et al. (2010a) for monopiles, lower than \( x = 0·39 \) reported by Zhu et al. (2013) but higher than \( x = 0·18 \) proposed by Foglia et al. (2014) also for caissons. The fitting parameters for the tests series are grouped, and the curves corresponding to those fits are plotted in Figure 13 as dotted.
lines. The coefficient $T$ in Equation 4 varied for each test; the fitted parameters are shown in Figure 13 and tabulated in Table 5.

In addition to rotation being observed under one-way loading, rotation was also observed in the two-way loading series ($\xi_b = -1$) against the direction of the first loading cycle, in this case indicated as a negative accumulation of rotation. Intuitively no rotation would be expected in a two-way test, as there is no resultant direction of loading. Similar observations have been made by Rosquët (2004) and Klinkvort and Hededal (2013) for cyclically loaded monopile foundations in a centrifuge. The accumulation of rotation for all the two-way tests can be seen in Figure 14.

It is believed that the rotation in a two-way test is a result of soil displacements after the first loading cycle. This can be demonstrated by considering the horizontal displacement of the caisson tower (as measured by the laser potentiometers at different heights above the caisson for the first four cycles of loading). The caisson system is vertical when these readings are the same. Instances when this occurs are indicated by the
circles for test C2-C-8 as shown in Figure 15(a), alongside that of a typical one-way response for the C2-C-3 test as shown in Figure 15(b). It is evident from Figure 15(a) that the lateral displacement between vertical positions reduces with increasing number of cycles, indicating an increase in horizontal caisson stiffness. The majority of this stiffening (displacement change) appears to happen in the direction of the first loading cycle, indicated by the shifting verticality point. This behaviour has the effect of producing an accumulation of structural rotation away from the direction, of the initial loading cycle.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>$\xi_b$</th>
<th>$\xi_c$</th>
<th>$\theta_c$ rad</th>
<th>$\Delta \theta$ rad</th>
<th>$K_0$</th>
<th>$T$</th>
</tr>
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<tr>
<td>C3-C-1</td>
<td>0.40</td>
<td>0.02</td>
<td>0.00510</td>
<td>0.00410</td>
<td>46.3</td>
<td>0.097</td>
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<td>C3-C-2</td>
<td>0.60</td>
<td>0.34</td>
<td>0.00841</td>
<td>0.00490</td>
<td>24.1</td>
<td>0.024</td>
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<td>C3-C-3</td>
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<td>0.00</td>
<td>0.01330</td>
<td>0.00720</td>
<td>46.3</td>
<td>0.112</td>
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<tr>
<td>C3-C-4</td>
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<td>0.89</td>
<td>0.00242</td>
<td>0.00640</td>
<td>13.7</td>
<td>0.044</td>
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<tr>
<td>C3-C-5</td>
<td>0.40</td>
<td>0.99</td>
<td>0.00510</td>
<td>0.01170</td>
<td>23.7</td>
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<td>0.01033</td>
<td>0.00740</td>
<td>106.8</td>
<td>0.032</td>
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<tr>
<td>C2-C-2</td>
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<td>0.52</td>
<td>0.01033</td>
<td>0.00360</td>
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<tr>
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<td>0.01715</td>
<td>0.01040</td>
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<tr>
<td>C2-C-6</td>
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<td>0.02180</td>
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<td>-0.021</td>
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</table>

Table 5. Summary of centrifuge test results

Figure 13. Non-dimensionalised accumulation of rotation: (a) C3-C $\xi_c = 0$; (b) C2-C $\xi_b = 0.54$; (c) C2-C $\xi_b = 0.80$
Assuming that initially the sand was relatively undisturbed, during the first loading cycle the lateral sand stress will increase in the compression zone and reduce in the tensile zone. This causes a differential sand density either side of the caisson, shown schematically in Figure 16. The initial compression zone will exhibit a greater potential for densification compared to the tensile zone, which will lead to an accumulation of rotation away from the direction of the first loading cycle. It was subsequently possible to map how each loading regime affected the accumulation of rotation. These results are presented alongside those of Zhu et al. (2013) and Foglia et al. (2014) in Figure 17. The results show similar behaviour to that observed previously in 1g experiments, such as large loading regimes causing a greater accumulation of rotation (as illustrated by comparing C3-C-1 to C3-C-5 and C2-C-1 to C2-C-5) and a loading directionality somewhere between one-way and two-way producing the greatest aggregated rotation (shown by test C2-C-3).

4.6 Settlement
Under the application of a cyclic load the caisson system was observed to settle. Although no data fitting was applied to these results, a number of factors affected the magnitude of the observed settlement. In general, caissons subjected to a greater maximum load ($\xi_b$) were observed to settle more, as shown in Figure 18(a). In addition, the more two-way the loading
regime became (i.e. as $\xi_c$ approaches $-1.0$), the greater the settlements that were observed, as illustrated in Figure 18(b) and Figure 18(c).

5. Implications for prototype structures
It is instructive to apply the findings reported here to a hypothetical prototype system. Modelling a typical 5 MW wind turbine (NREL 5 MW reference Jonkman et al. (2009)) in approximately 25 m of water, the maximum load is calculated, considering an extreme operational gust. In this case a design moment, horizontal and vertical loading of 340 MNm, 6 MN, 18 MN respectively were assumed. It may be noted that these values are different from the operational loads.

For a typical European offshore site, the ground conditions can be approximated by a sand with an effective unit weight of

Figure 16. Assumption of local sand movements from observations: (a) undisturbed sand level; (b) sand is displaced in the compression zone and slumps in the tensile zone; (c) the previously displaced sand is allowed to settle and the subducted material is displaced

Figure 17 (a) Load directionality coefficient $T$, for the C3 caisson with the results of Zhu et al. (2013) and Foglia et al. (2014); (b) load directionality coefficient, $T$ for the C2 caisson
\[ \gamma' = 9.7 \text{ kN/m}^3 \] and a friction angle of \( \phi_c = 30^\circ \) (Bhattacharya et al., 2009; Kühn, 2001). Applying a factor of safety of 2 on the ultimate capacity as outlined in API RP 2A-WSD (American Petroleum Institute, 2007) and employing the design procedure described by Schakenda et al. (2011), a single caisson approximately 18 m in diameter with a skirt penetration depth of 18 m would be required. Under normal operation (defined as case 1.1 in British standards (BSI, 2009)), a cyclic moment in the range 140–220 MNm was estimated to act upon the turbine. The maximum moment-carrying capacity has been estimated to be 880 MNm, which leads to a load capacity and direction factors of \( \xi_0 = 0.25 \) and \( \xi_0 = 0.63 \) respectively, obtained from Equations 1 and 2.

Calculating the accumulation of rotation for this case, a rotation coefficient of \( T = 0.0066 \) can be interpolated (obtained from Figure 17(b) considering a loading directionality \( \xi = 0.63 \)). If such a loading regime (cyclic moment of 140 MNm and 220 MNm) were applied to a turbine for a period of 15 years \(( -4 \times 10^7 \text{ cycles})\), a normalised non-dimensional rotation of 1.29 can be obtained (using Equation 4). For the prototype turbine, considering a dimensional static rotation of \( 0.21^\circ \) (obtained from Figure 10), an accumulated rotation of \( 0.30^\circ \) would be expected. Such a rotation would not be within the serviceability limits imposed on the design (Peire et al., 2009). However, for the same loading conditions \((\xi_0 = 0.25 \text{ and } \xi_0 = 0.63)\), a caisson with a diameter and penetration depth of 19 m would experience a reduced total accumulated rotation of \( 0.22^\circ \), which would lie within the serviceability limits.

Considering the same loading regime detailed above (cyclic moment of 140 MNm and 220 MNm), an initial non-dimensional rotational stiffness of 107 has been estimated. This value is expected to increase by 25% after 15 years, based on Equation 3 and a stiffness change parameter \((A_k)\) of 1.52 obtained from this investigation. In practice however this is highly dependent on the current void ratio of the soil and its strain hardening characteristics (Bhattacharya et al., 2013). Using the dynamic model defined by Adhikari and Bhattacharya (2011), this would alter the first modal frequency of such a wind turbine from \( 0.218 \text{ Hz} \) to \( 0.221 \text{ Hz} \). Assuming the turbine is designed as a soft-stiff system (see Figure 2), such a change would have the effect of moving the modal frequency further away from the 1P

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**Figure 18 Caisson settlement with number of loading cycles:**
(a) C3-C settlement \( \xi = 0.0 \); (b) C2-C settlement \( \xi = 0.54 \); (c) C2-C settlement \( \xi = 0.80 \)
zone, causing a long-term benefit. Owing to the consistent stiffening response observed for different caisson geometries and various loading regimes, altering the caisson dimensions would not reduce the strain hardening characteristics of the local sand.

6. Conclusion
Considering the results presented in this paper it is possible to draw a number of conclusions about the investigation and the use of caissons as foundations for offshore wind structures. These conclusions are based on the practical maximum number of cycles that can be applied in a geotechnical centrifuge (given time and usage constraints). The tests in this investigation involved up to 12,000 cycles, which is short of the $1 \times 10^8$ cycles that may potentially be applied to a turbine over its 20-year lifetime. Despite this limitation, clear trends and behaviours were identified from the tests as summarised below.

(a) The aspect ratio of the caisson has a significant effect on the foundation stiffness and its corresponding capacity. In the pushover tests not only was the caisson with a higher aspect ratio stiffer and stronger, but also displayed ductile behaviour. This behaviour can only be attributed to its increased embedment of the caisson changing the failure mechanism of the foundation.

(b) The rotational stiffness of the caisson system increased when subjected to cyclic loading under a fully drained condition. This increase was approximated by a logarithmic relationship, which was effected to some degree by the load orientation and load level. The increase in stiffness during long-term cyclic loading was much lower than previously reported for monopiles (LeBlanc et al., 2010a).

(c) Under the application of cyclic loading the suction caissons experienced an accumulation of rotation with increasing cycles under a fully drained condition. This rotation was dependent on the magnitude and relative directionality of the loading. In general a cyclic regime between one-way and two-way caused the greatest rotation, similar to the observations of LeBlanc et al. (2010a), Zhu et al. (2013) and Foglia et al. (2014).

(d) Some rotation was observed in the two-way loading tests where there was no predominant loading direction. This is believed to be due to the first quarter loading cycle creating differing soil conditions on opposite sides of the caisson, producing the conditions for one side to compact more than the other. This accumulation of rotation was independent of the load magnitude and is a feature that requires further investigation.

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