Centrifuge study on the cyclic performance of caissons in sand

James A. Cox MEng

Postgraduate Student, Department of Civil Engineering, University of Bristol, UK

Conleth D. O'Loughlin PhD

Associate Professor, Centre for Offshore Foundation Systems, University of Western Australia, Crawley, Australia

Mark Cassidy DPhil, FIEAust, FTSE

Professor/Director, Centre for Offshore Foundation Systems, University of Western Australia

Subhamoy Bhattacharya PhD (Cantab)

Professor, Department of Civil and Environmental Engineering, University of Surrey, Guildford, UK

Christophe Gaudin MEng, PhD

Professor/Deputy Director, Centre for Offshore Foundation Systems, University of Western Australia, Crawley, Australia

Britta Bienen DiplIng, PhD

Associate Professor, Centre for Offshore Foundation Systems, University of Western Australia, Crawley, Australia

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.

Notation

A_k	stiffness change parameter	t	caisson wall thickness (m)
D	caisson diameter (m)	V	vertical load on foundation (N)
D_r	relative density of sand (%)	w	vertical settlement (m)
D_{50}	mean sand grain diameter (mm)	Z	caisson skirt length (m)
E_s	Young's modulus of caisson skirt (MPa)	α	rotation coefficient
$e_{\max}$	maximum void ratio	γ'	unit weight of soil (N/m ³)
$e_{\min}$	minimum void ratio	θ	observed rotation (rad)
G_r	shear modulus of soil at a depth of $D/2$ (MPa)	θ_s	static unit rotation (rad)
H	horizontal load on foundation (N)	ξ_b	regime loading level
K	unloading stiffness (Nm/rad)	ξ_c	regime loading directionality
K_0	characteristic non-dimensional foundation stiffness	φ_{cv}	friction angle (°)
k	instantaneous non-dimensional foundation stiffness		
M	moment on foundation (Nm)		
$M_{\max}$	maximum moment applied (Nm)		
$M_{\min}$	minimum moment applied (Nm)		
M_R	static moment capacity (Nm)		
N	number of cycles		
P_a	atmospheric pressure (kPa)		
T	foundation rotation coefficient		

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; Houlsby *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).

Figure 1. Arrangement of a typical 5 MW offshore wind turbine founded on a single suction caisson

Figure 2. Typical loading frequencies and dynamically sensitive regions of a NREL 5 MW turbine structure (after Bhattacharya *et al.*; 2012)

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 (Young's modulus, $E_s = 70$ 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/m³, 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 $e_{\max} = 0.747$, $e_{\min} = 0.449$, $D_{50} = 0.15$ mm and $\phi_{cv} = 31^\circ$ (Liu and Lehane, 2012; O'Loughlin and Lehane, 2003)), and as such the particle size to foundation diameter ratio was greater than

Group	No.	Description	Frederikshavn	Wilhelmshaven	Horns Rev 2 Met mast	Test models
$\frac{E_s t}{G_r D}$	1	Caisson flexibility	10.10	7.89	6.73 ^a	9.20
$\frac{Z}{D}$	2	Caisson aspect ratio	0.50	1.00	0.50	0.50/1.00
$\frac{V}{\gamma' D^3}$	3	Vertical caisson capacity	0.48	0.43	0.99	0.69/0.76
$\frac{H}{\gamma' D^3}$	4	Horizontal caisson capacity	0.05	0.05	0.06	Varies
$\frac{M}{\gamma' D^4}$	5	Moment caisson capacity	0.39	0.28	0.04	Varies
$\frac{G_r}{\gamma' D}$	6	Strain multiplier (considers relative stress–strain behaviour)	436.7	392.9	436.7	475.9

^aThickness calculated from DNV guidelines (DNV-RP-C202 Det Norske Veritas (2010))

Table 1. Prototype non-dimensional groupings (Det Norske Veritas, 2010)

60 (Hoadley *et al.*, 1981). 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 17.4–17.5 kN/m³ ($D_r = 86.5 \pm 2.5\%$) 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.

Figure 3. Notation and load reference point utilised in the non-dimensional groupings following the definitions of Doherty *et al.* (2005)

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.

Parameter	Normalised expression
Angular rotation $[\tilde{\theta}]$	$\theta \left[\frac{P_a}{\gamma' D} \right]^{0.5}$
Vertical displacement $[\tilde{w}]$	$\left(\frac{w}{D} \right) \left[\frac{P_a}{\gamma' D} \right]^{0.5}$
Unload stiffness in each cycle $[\tilde{k}]$	$\frac{k}{\left[(\gamma' P_a)^{0.5} D^{3.5} \right]}$

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

The following experimental procedures were adopted for each test run.

- A centrifuge strong box filled with a sample of very dense silica sand was located on the swinging platform of the centrifuge.
- Each sample was spun to 200g 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.
- The sample was spun down to 1g and the model caisson was carefully jacked into the sand at a rate of 0.25 mm/s at 1g using the actuator, ensuring verticality, until the caisson was fully installed. Subsequently the air vent in the caisson lid was sealed.
- The model caisson was then connected to the dual axis actuator by way of the pivoted loading arm.
- The centrifuge was then spun up until an acceleration of 200g was acting at the mid-depth level of the caisson.
- The selected loading regime was then applied to the model and the corresponding response of the system was recorded.

- At the end of the experiment the centrifuge was spun down to 1g 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.

- 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.
- 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

$$1. \quad \xi_b = \frac{M_{\max}}{M_R}$$

$$2. \quad \xi_c = \frac{M_{\min}}{M_{\max}}$$

where ξ_b specifies the load level as a ratio of the maximum moment and the static moment capacity, and ξ_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

Figure 4. Model caisson used in the centrifuge tests: (a) C3 caisson ($Z/D = 0.5$) and (b) C2 caisson ($Z/D = 1.0$)

ID	Description	Diameter, D : mm	Skirt length, Z : mm	Thickness, t : mm	Mass at $1g$: kg
C2	5MW model, 1:200	80.0	80.0	1.4	0.68
C3	5MW model, 1:200	80.0	40.0	1.4	0.65

Table 3. Details of centrifuge model caissons

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.

Figure 5. Cross-section schematic diagram of the test layout

Figure 6. Definition of cyclic parameters (after Zhu *et al.*; 2013)

Figure 7. Unloading stiffness definition for a single loading cycle

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 (θ_s) to be made; this is the structural rotation observed

Figure 8. CPT data for all sand samples used in this investigation

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 [$M/\gamma'D^4$] and rotational strain [$\theta/(P_a/\gamma'D)^{0.5}$]. These results are shown in Figure 10 alongside the yield load calculated in the same manner as Zhu *et al.* (2013).

Test ID	Percentage installed: %	Number of cycles: N	$H/\gamma'D^3$	$M/\gamma'D^4$	ξ_b	ξ_c
C3-P-1	90.2	—	—	—	—	—
C3-C-1	89.1	71	0.04	0.12	0.40	0.02
C3-C-2	88.8	1877	0.06	0.18	0.60	0.34
C3-C-5	89.3	690	0.08	0.24	0.80	0.00
C3-C-6	90.1	9329	0.02	0.06	0.20	−0.89
C3-C-7	89.7	2209	0.04	0.12	0.40	−0.99
C2-P-3	88.3	—	—	—	—	—
C2-C-1	89.0	1555	0.16	0.51	0.54	0.01
C2-C-2	90.2	1587	0.16	0.51	0.54	0.52
C2-C-3	89.2	2367	0.24	0.77	0.81	−0.32
C2-C-4	88.9	2597	0.24	0.75	0.79	0.34
C2-C-5	87.8	2187	0.24	0.76	0.80	0.03
C2-C-6	89.6	1962	0.08	0.25	0.26	−0.97
C2-C-7	90.3	12 121	0.16	0.52	0.55	−0.99
C2-C-8	88.9	6985	0.24	0.73	0.77	−0.88

Table 4. Centrifuge tests parameters

Figure 9. Caisson installation records conducted at $1g$

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 pushover 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

Figure 10 Non-dimensional pushover results

Figure 11. Calculated non-dimensional stiffness with rotation level

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 (ξ_b) and load directionality (ξ_c) 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 (θ_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

$$4. \quad \frac{\Delta\theta(N)}{\theta_s} = TN^\alpha$$

As fewer tests were conducted here than in previous investigations (owing to the limited testing window available), the two loading parameters ξ_b and ξ_c (used by LeBlanc *et al.*, 2010a) were combined and considered as a single variable T . The data were best fitted using $\alpha = 0.3$ which is comparable to $\alpha = 0.31$ reported by LeBlanc *et al.* (2010a) for monopiles, lower than $\alpha = 0.39$ reported by Zhu *et al.* (2013) but higher than $\alpha = 0.18$ 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

Figure 12. Non-dimensionalised stiffness with number of loading cycles: (a) C3-C stiffness $\xi_c = -1.0$; (b) C3-C $\xi_c = 0.0$; (c) C2-C stiffness $\xi_c = -1.0$; (d) C2-C $\xi_b = 0.54$; (e) C2-C $\xi_b = 0.80$

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 Rosquoët (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

Test ID	ξ_b	ξ_c	θ_s : rad	$\Delta\theta$: rad	K_0	T
C3-C-1	0.40	0.02	0.00510	0.00410	46.3	0.097
C3-C-2	0.60	0.34	0.00841	0.00490	24.1	0.024
C3-C-5	0.80	0.00	0.01330	0.00720	46.3	0.112
C3-C-6	0.20	-0.89	0.00242	0.00640	13.7	0.044
C3-C-7	0.40	-0.99	0.00510	0.01170	23.7	-0.127
C2-C-1	0.54	0.01	0.01033	0.00740	106.8	0.032
C2-C-2	0.54	0.52	0.01033	0.00360	107.5	0.009
C2-C-3	0.81	-0.32	0.01715	0.01400	117.2	0.057
C2-C-4	0.79	0.34	0.01715	0.00580	122.7	0.020
C2-C-5	0.80	0.03	0.01715	0.01040	122.3	0.039
C2-C-6	0.26	-0.97	0.00475	0.00930	81.4	-0.122
C2-C-7	0.55	-0.99	0.01033	0.01570	97.4	-0.037
C2-C-8	0.77	-0.88	0.01715	0.02180	119.6	-0.021

Table 5. Summary of centrifuge test results

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.

Figure 13. Non-dimensionalised accumulation of rotation: (a) C3-C $\xi_c = 0$; (b) C2-C $\xi_b = 0.54$; (c) C2-C $\xi_b = 0.80$

Figure 14. Two-way accumulation of rotation: (a) C2 caisson;
(b) C3 caisson

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 (ξ_b) were observed to settle more, as shown in Figure 18(a). In addition, the more two-way the loading

Figure 15. Individual displacement reading from the two laser
potentiometers: (a) C2-C-8; (b) C2-C-3

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

regime became (i.e. as ξ_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 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

Figure 18 Caisson settlement with number of loading cycles:
(a) C3-C settlement $\xi_c = 0.0$; (b) C2-C settlement $\xi_b = 0.54$;
(c) C2-C settlement $\xi_b = 0.80$

$\gamma' = 9.7 \text{ kN/m}^3$ and a friction angle of $\phi_{cv} = 30^\circ$ (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 $\xi_b = 0.25$ and $\xi_c = 0.63$ 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 $\xi_c = 0.63$). If such a loading regime (cyclic moment of 140 MNm and 220 MNm) were applied to a turbine for a period of 15 years ($\sim 4 \times 10^7$ 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° (obtained from Figure 10), an accumulated rotation of 0.30° 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 ($\xi_b = 0.25$ and $\xi_b = 0.63$), a caisson with a diameter and penetration depth of 19 m would experience a reduced total accumulated rotation of 0.22° , 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 Hz to 0.221 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

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 1×10^8 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.

Acknowledgements

The research presented in this paper would not have been possible without the exemplary help from the World

Universities Network researcher mobility program. Specific thanks should go to Alan Champneys, Manuel Palacois, Dave Jones, John Breen, Shane DeCatilina and Khin Thida Seint without whose help this research would not have been possible. This work forms part of the activities of the Centre for Offshore Foundation Systems (COFS), currently supported as a node of the Australian Research Council Centre of Excellence for Geotechnical Science and Engineering and as a Centre of Excellence by the Lloyd's Register Foundation. Lloyd's Register Foundation invests in science, engineering and technology for public benefit, worldwide.

REFERENCES

- Adhikari S and Bhattacharya S (2011) Vibrations of wind-turbines considering soil-structure interaction. *Wind and Structures* **14**(2): 85–112.
- American Petroleum Institute (2007) *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress Design*. American Petroleum Institute, Washington DC, USA.
- Andersen KH, Murff JD, Randolph MF *et al.* (2005) Suction anchors for deepwater applications. In *Frontiers in Offshore Geotechnics: ISFO* (Gourvenec S and Cassidy MJ (eds)). Taylor & Francis, Perth, Australia.
- Bhattacharya S, Carrington T and Aldridge T (2009) Observed increases in offshore pile driving resistance. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **162**(1): 71–80.
- Bhattacharya S, Cox JA, Lombardi D and Muir Wood D (2012) Dynamics of offshore wind turbines on two types of foundations. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **166**(2): 159–169.
- Bhattacharya S, Nikitas N, Garnsey J *et al.* (2013) Observed dynamic soil-structure interaction in scale testing of offshore wind turbine foundations. *Soil Dynamics and Earthquake Engineering* **54**: 47–60.
- BSI (2009) BE EN 61400-3: 2009. Wind turbines – Part 3: Design requirements for offshore wind turbines. BSI, London, UK.
- Byrne BW (2000) *Investigations of Suction Caissons in Dense Sand*. PhD thesis, University of Oxford, Oxford, UK.
- Carter JMF (2007) North Hoyle offshore wind farm: design and build. *Proceedings of the Institution of Civil Engineers – Energy* **160**(1): 21–29.
- Cox JA and Bhattacharya S (2012) Dynamic stability of suction caisson founded offshore wind turbines. *Proceedings of the Offshore Technology Conference*, American Petroleum Industry (API), Houston, TX, USA, pp. 1–12.
- Det Norske Veritas (2002) *Guidelines for Design of Wind Turbines*. Det Norske Veritas, Copenhagen, Denmark.
- Det Norske Veritas (2010) *Buckling Strength of Shells DNV-OS-C202*. Det Norske Veritas, Høvik, Norway.

- Doherty JP, Houlby GT and Deeks AJ (2005) Stiffness of flexible caisson foundations embedded in nonhomogeneous elastic soil. *Journal of Geotechnical and Geoenvironmental Engineering* **131**(12): 1498–1508.
- DTI (Department of Trade and Industry) (2001) *Monitoring and Evaluation of Blyth Offshore Wind Farm – Projected Capital Costs of UK Offshore Windfarms Based on the Experience at Blyth*. DTI, London, UK, DTI/Pub URN 01/1524.
- DTI (2007) Study of the costs of offshore wind generation. *A Report to the Renewables Advisory Board and DTI*. DTI, London, UK.
- Dyson G (1999) *Lateral Loading of Piles in Calcareous Sediments*. PhD thesis, University of Western Australia, Crawley, Australia.
- Feld T (2001) *Suction Buckets, A New Innovative Foundation Concept, Applied to Offshore Wind Turbines*. PhD thesis, Aalborg University, Aalborg, Denmark.
- Foglia A and Ibsen LB (2013) A similitude theory for bucket foundations under monotonic horizontal load in dense sand. *Geotechnical and Geological Engineering* **31**(1): 133–142.
- Foglia A, Ibsen LB, Nicolai G and Andersen LV (2014) Observations on bucket foundations under cyclic loading in dense saturated sand. In *ICPMG2014 – Physical Modelling in Geotechnics: Proceedings of the 8th International Conference on Physical Modelling in Geotechnics, Perth, Australia* (Gaudin C and White D (eds)). CRC Press, Boca Raton, FL, USA, pp. 667–673.
- Gaudin C, Clukey E, Garnier J and Phillips R (2010) New frontiers for centrifuge modelling in offshore geotechnics. *Frontiers in Offshore Geotechnics II*. CRC Press, Boca Raton, Florida, USA.
- Hardin BO and Drnevich VP (1972) Shear modulus and damping in soils: Design equations and curves. *Journal of the Soil Mechanics and Foundations Division, ASCE* **98**(7): 667–692.
- Hoadley PJ, Barton YO and Parry RHG (1981) Cyclic lateral load on model pile in a centrifuge. *Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Sweden*, pp. 621–635.
- Houlby GT, Ibsen LB and Byrne BW (2005) Suction caissons for wind turbines. In *Proceedings of Frontiers in Offshore Geotechnics: ISFOG, Perth, Australia* (Cassidy G (ed.)). Taylor & Francis, London, UK.
- Jonkman J, Butterfield S, Musial W and Scott G (2009) *Definition of a 5-MW Reference Wind Turbine for Offshore System Development Golden*. National Renewable Energy Laboratory, Golden, CO, USA.
- Kelly RB, Houlby GT and Byrne BW (2006) A comparison of field and laboratory tests of caisson foundations in sand and clay. *Géotechnique* **56**(9): 617–626.
- Klinkvort RT and Hededal O (2013) Lateral response of monopile supporting an offshore wind turbine. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **166**(2): 147–158.
- Klinkvort RT, Leth CT and Hededal O (2010) Centrifuge modelling of a laterally cyclic loaded pile. *Physical Modelling in Geotechnics*. Taylor & Francis, Abingdon, UK.
- Kühn M (2001) *Dynamics and Design Optimisation of Offshore Energy Wind Conversion Systems*. PhD thesis, Delft University of Technology, Delft, the Netherlands.
- Kühn M, Bierbooms WAAM, Van Bussel GJW et al. (1998) *Opti-OWECS; Structural and Economic Optimisation of Bottom Mounted Offshore Wind Energy Convertors. JOULE III Project Vols 0–5*. Delft University of Technology, Delft, the Netherlands.
- Larsen JHM, Soerensen HC, Christiansen E, Naef S and Vølund P (2005) *Experiences from Middelgrunden 40 MW Offshore Wind Farm*. Copenhagen Offshore Wind, Copenhagen, Denmark, Lyngby, Denmark.
- LeBlanc C (2009a) The Monopod Bucket Foundation: recent experiences and challenges ahead. *Proceedings of Hamburg Offshore Wind Conference 2009*. GL Group, Hamburg, Germany. See http://www.gl-group.com/pdf/No12_Bakmar.pdf (accessed 10/12/2014).
- LeBlanc C (2009b) *Design of Offshore Wind Turbine Support Structures*. PhD thesis, Technical University of Denmark, Lyngby, Denmark.
- LeBlanc C, Houlby GT and Byrne BW (2010a) Response of stiff piles in sand to long-term cyclic lateral loading. *Géotechnique* **60**(2): 79–90.
- LeBlanc C, Houlby GT and Byrne BW (2010b) Response of stiff piles to random two-way lateral loading. *Géotechnique* **60**(9): 715–721.
- Li Z, Haigh SK and Bolton MD (2010) Centrifuge modelling of mono-pile under cyclic lateral loads. *Proceedings of the 7th International Conference on Physical Modelling in Geotechnics, Zurich, Switzerland*, pp. 965–970.
- Liingaard MA (2006) *Dynamic Behaviour of Suction Caissons*. PhD thesis, Technical University of Denmark, Denmark.
- Lin S and Liao J (1999) Permanent strains of piles in sand due to cyclic lateral loads. *Journal of Geotechnical and Geoenvironmental Engineering* **125**(9): 798–802.
- Liu Q and Lehane B (2012) The influence of particle shape on the (centrifuge) cone penetration test (CPT) end resistance in uniformly graded granular soils. *Géotechnique* **62**(11): 973–984.
- Long J and Vanneste G (1994) Effects of cyclic lateral loads on piles in sand. *Journal of Geotechnical Engineering* **120**(1): 225–244.
- Lu X, Wu Y, Jiao B and Wang S (2007) Centrifugal experimental study of suction bucket foundations under dynamic loading. *Acta Mechanica Sinica* **23**(6): 689–698.
- O'Loughlin C and Lehane B (2003) *Measure and Prediction of Deformation Patterns Beneath Strip Footings in Sand*. Department of Civil and Resource Engineering, University

- of Western Australia, Perth, Australia, Research report no. C:1751.
- Peire K, Nonneman H and Bosschem E (2009) Gravity based foundations for the Thornton Bank Offshore Wind Farm. *Terra et Aqua* **2009(115)**: 19–29.
- Randolph MF and Wroth CP (1979) An analytical solution for the consolidation around a driven pile. *International Journal for Numerical and Analytical Methods in Geomechanics* **3(3)**: 217–229.
- Randolph MF, Jewell RJ, Stone KJL and Brown TA (1991) Establishing a new centrifuge facility. In *Proceedings International Conference Centrifuge 91, Boulder, CO, USA* (Ko H-Y and Mclean FG (eds)). Balkema, Stockholm, Sweden, pp. 3–9.
- Rosquoët F (2004) *Pieux sous charge latérale cyclique*. PhD thesis, University of Nantes, France (in French).
- Sawicki A and Swidzinski W (1989) Mechanics of a sandy subsoil subjected to cyclic loadings. *International Journal for Numerical and Analytical Methods in Geomechanics* **13(19)**: 511–529.
- Schakenda B, Nielsen SA and Ibsen LB (2011) *Foundation Structure*. United States of America patent application 12/226,255, February.
- Senders M (2008) *Suction Caissons in Sand as Tripod Foundations for Offshore Wind Turbines*. PhD thesis, University of Western Australia, Crawley, Australia.
- Tran MN (2005) *Installation of Suction Caissons in Dense Sand and the Influence of Silt and Cemented Layers*. PhD thesis, The University of Sydney, Sydney, Australia.
- Verdure L, Garnier J and Levacher G (2003) Lateral cyclic loading of single piles in sand. *International Journal of Physical Modelling in Geotechnics* **3(3)**: 17–28.
- Villalobos FA (2006) *Model Testing of Foundations for Offshore Wind Turbines*. PhD thesis, University of Oxford, UK.
- Zaaijer MB (2009) Review of knowledge development for the design of offshore wind energy technology. *Wind Energy* **12(5)**: 411–430.
- Zhang JH, Zhang LM and Lu XB (2007) Centrifuge modeling of suction bucket foundations for platforms under ice-sheet-induced cyclic lateral loadings. *Ocean Engineering* **34(8)**: 1069–1079.
- Zhu B, Byrne B and Houlsby GT (2013) Long-term lateral cyclic response of suction caisson foundations in sand. *Journal of Geotechnical and Geoenvironmental Engineering* **139(1)**: 73–83.

WHAT DO YOU THINK?

To discuss this paper, please email up to 500 words to the editor at journals@ice.org.uk. Your contribution will be forwarded to the author(s) for a reply and, if considered appropriate by the editorial panel, will be published as discussion in a future issue of the journal.

Proceedings journals rely entirely on contributions sent in by civil engineering professionals, academics and students. Papers should be 2000–5000 words long (briefing papers should be 1000–2000 words long), with adequate illustrations and references. You can submit your paper online via www.icevirtuallibrary.com/content/journals, where you will also find detailed author guidelines.