SUCTION CAISSON FOUNDATIONS FOR OFFSHORE WIND TURBINES

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SUMMARY

Suction caisson foundations are being investigated for offshore wind turbine applications. The research programme includes laboratory testing, larger scale field testing and theoretical modelling. This paper concentrates on the experimental results obtained in combined loading tests on monopod caissons. Results obtained from monotonic and cyclic tests on caissons installed either by pushing or by suction are presented and interpreted.

1. INTRODUCTION

The need for increased production of clean and sustainable energy in the near future has resulted in a search for alternatives to fossil fuels as sources of energy, such as nuclear power or a variety of “renewable” sources such as hydroelectric, solar or wind power. Wind energy is one of the most promising options for electricity generation, with optimistic growth forecasts for the near future (Byrne and Houlsby, 2003), particularly in offshore wind energy. The wind speed is typically higher and steadier offshore than onshore, so offshore wind turbines can produce more power. However, capital costs (including installation and cable costs) are around 30-50% higher than onshore. The decision to go offshore can be justified by the higher energy generation of 20% to 40% when compared to onshore wind turbines (Milborrow, 2003). Offshore electricity costs are dropping and, depending on the technological developments, could reduce to a third of present levels. As a result, offshore wind power is becoming more competitive when compared to other power sources.

The UK Government, in the Renewables Obligations (UK Government 2002), is implementing a renewable energy policy to reduce CO_2 emissions. Currently, offshore wind farms are being built along the UK coasts, with the target to supply 10% of UK electrical energy requirements by 2010. At the time of writing, 32 offshore wind turbines were in operation and a further 60 in construction. It has been estimated that about 3000 turbines might be necessary to achieve the 10% target.

In an offshore wind farm project, the cost of the foundations has been estimated to be about 35% of the total installation cost (Byrne and Houlsby, 2003). Two types of foundations have so far been used for offshore wind turbines: gravity bases and piled foundations. The first option is not suitable for large towers, since the size and weight required becomes excessive. The second option of piling can be pursued however as the size of the pile increases the installation time and cost increases at a disproportionate rate. In addition, as the size of the pile and structure increases issues of structural dynamics can start to dominate the design process.

Figure 1a depicts the order of magnitude of the size of an offshore wind turbine, in shallow water with a depth between 5m and 20m. For this sort of installation suction caissons might be a feasible solution to the foundation problem. This type of foundation has been used in the oil and gas industry in the construction of platforms and other offshore facilities (Sparrevik, 2002). However, the loading from a wind turbine structure differs from that for oil and gas structures - for the wind turbine the moment loads are much larger in comparison with the vertical loads than for typical oil and gas applications (Byrne and Houlsby, 2003). Furthermore the total weight of the turbine structure is much lower, and many installations are required within a wind farm.

The arrangement options for the wind turbine foundations could be a monopod, tripod or quadruped (see Figure 1b). For a tripod or quadruped the structural design approach must take into account the fact that the most unfavourable conditions involve the possibility of transient tensile loads in the upwind leg (for a discussion of this problem see Kelly *et al.*, 2004). For monopods the most unfavourable loading condition results primarily in a large overturning moment. Design issues include (i) ultimate capacity of the foundation, (ii) displacements associated with this capacity and (iii) the accumulated deformations that occur under cyclic loading.

This paper discusses a large research project on suction caisson foundations including laboratory testing, large scale field trials and numerical modelling. The paper will describe briefly these three components, and will concentrate on the experimental results obtained in combined loading tests of monopod caissons in sand.

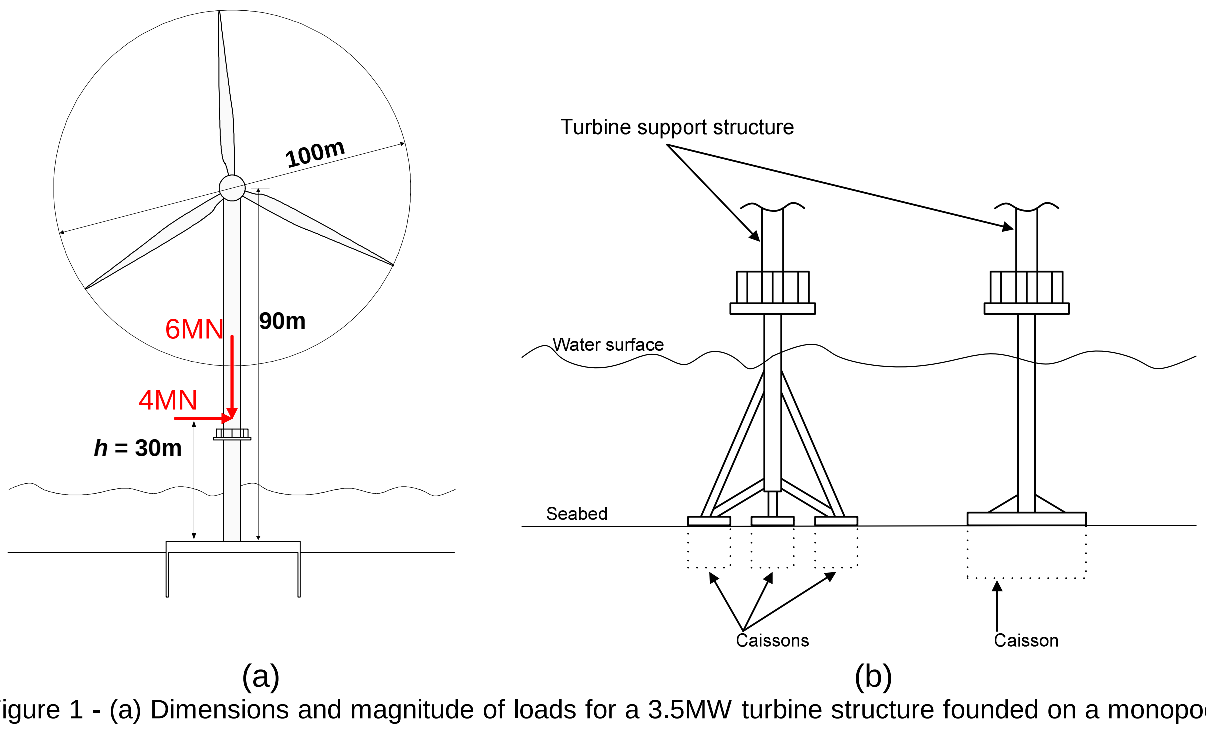


Figure 1 - (a) Dimensions and magnitude of loads for a 3.5MW turbine structure founded on a monopod suction caisson; (c) Different configurations for offshore wind turbines foundations: multiple caissons and monopod caisson (adapted from Byrne and Houlsby, 2003)

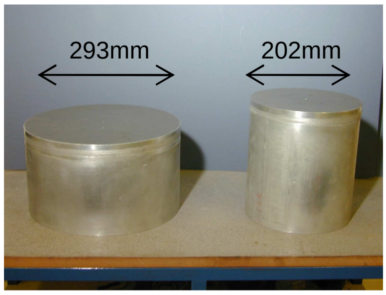
2. THE RESEARCH PROGRAMME

2.1 Laboratory testing

The laboratory tests were designed to provide the necessary data to develop theoretical models for offshore foundations. The experimental results are interpreted within the framework of “force resultant models”. In this approach, a complex soil structure interaction problem is reduced to the analysis of resultant loads applied at a chosen reference point, at which the transfer of loads from the superstructure to the foundation is considered as occurring. The foundation behaviour can then be incorporated with the response of the superstructure in a numerical analysis. The force resultant models are expressed using plasticity theory, and the main aim of the tests was to define yield conditions and the evolution of plastic displacements. A three degree-of-freedom (3DOF) loading rig, designed by Martin (1994), was used to carry out the tests, (Figure 2). This rig can simultaneously apply vertical, rotational and horizontal displacements (w, $2R\theta$, u) to a footing by means of computer controlled stepper motors (Byrne, 2000). Therefore, loads typical of the offshore environment, consisting of gravity, wind, waves and currents can be reproduced with the rig by applying vertical, moment, and horizontal loads (V, $M/2R$, H).

(a)

(b)



(c)

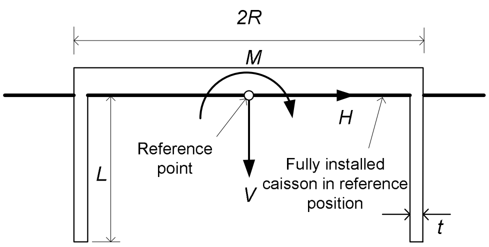


Figure 2 – (a) The three degree of freedom loading rig; (b) two of the caissons tested; (c) reference point and loads and displacements during loading

2.2 Large scale field trials

Despite the versatility and lower cost of laboratory tests as compared to field tests, there are issues of scaling that need to be addressed, including the effect of the much higher stress level encountered for prototype foundations. It is necessary to know how the results obtained in the laboratory will scale to applications involving real foundations. For that reason, large field trials have been conducted using two caissons (see Table 1, last two columns) installed by suction in clay and sand soils (Kelly, 2002). For the clay tests a reaction frame was set up in an excavated rectangular pit 20m by 10m, 2m deep at the Bothkennar test site. The loads were applied to the caissons using hydraulic jacks for compression-tension and moment tests, see Figure 3a and b. A structural eccentric mass vibrator (SEMV) was used to apply large numbers of cyclic moment loads of very small amplitude, see Figure 3b. The field test results are to be used to validate the numerical model, which are initially calibrated against the laboratory results. The field tests will not be discussed further in this paper.

(a)

(b)

Figure 3 - Field trial frame set up in clay showing: (a) the hydraulic jacks over the 1.5m diameter caisson; (b) the hydraulic and SEMV used to test the caisson of 3m diameter

2.3 Theoretical modelling

Force resultant models using work hardening plasticity theory have proved to be well suited to the analysis of the monotonic behaviour of spudcan and flat circular footings under combined loads (Martin, 1994; Houlsby and Cassidy, 2002). However, the response under cyclic loading is not so well modelled by this approach. Houlsby and Puzrin (2000) suggest that models using multiple yield surfaces may be suitable for modelling cyclic loading, and that these can be derived within a relatively compact mathematical framework by adopting the hyperplastic formulation which is based on thermodynamics. In conventional plasticity it is necessary to define four components: the shape of the yield surface, a hardening law, flow rule and elastic behaviour inside the yield surface. Whilst hyperplasticity theory requires the definition of just two scalar functions, these in turn can be established from knowledge of the behaviour in conventional plasticity terms.

3. MOMENT TESTS AND THEIR INTERPRETATION

3.1 Monotonic loading

Moment loading tests were carried out to investigate the response of a monopod caisson under low vertical loads. Two aspect ratios of caisson were tested, $L/2R$ = 0.5 and 1.0, as shown in Figure 2b. A range of aspect ratios is relevant as it is not yet clear which will lead to an optimal design. Due to installation considerations it likely that lower aspect ratios will be appropriate in sand and higher aspect ratios in clay. The soil used in the experiments on dry sand was loose white Leighton Buzzard sand (average relative density, R_d = 30%). Experiments on saturated sand used Baskarp Cyclone sand saturated with 100 centistokes silicon oil. The details of the caissons tested are given in Table 1, and the soil properties in Table 2.

Tests started by pushing the caisson into the ground, at a rate of $\dot{w}$ = 0.5mm/s, until the underside of the lid made complete contact with the soil. At that point the maximum vertical

Table 1 – Geometry of the model caissons

	Laboratory			Field	
Diameter, $2R$ (mm)	293	202	200	3000	1500
Length of skirt, L (mm)	150	200	100	1250	1000
Thickness of the skirt wall, t (mm)	3.4	3.4	1.0	10	10
Aspect ratio, $L/2R$	0.5	1	0.5	0.41	0.67
Thickness ratio, $2R/t$	86	59	200	300	150

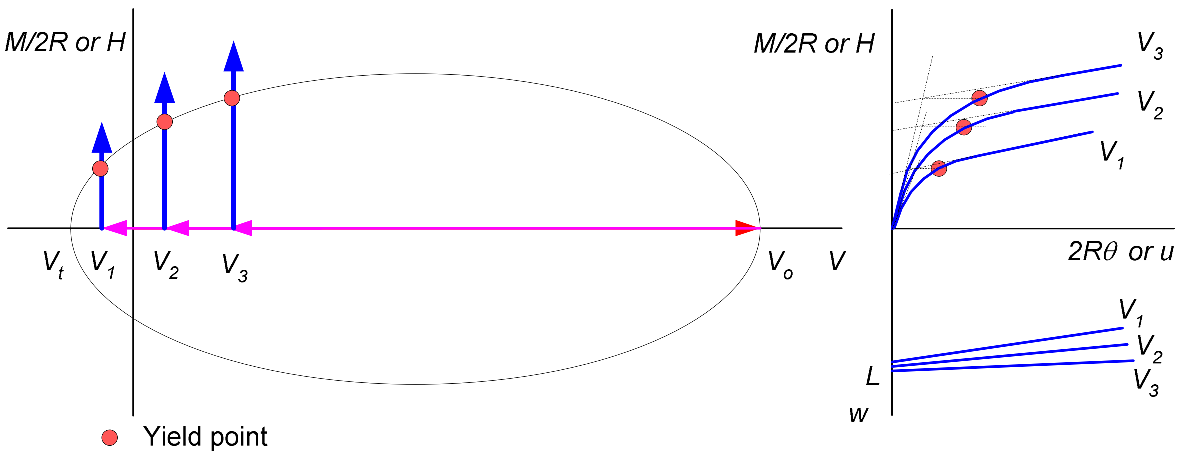
Table 2 – White Leighton Buzzard sand and Baskarp Cyclone sand properties (after Schnaid, 1990 and Byrne, 2000)

	Leighton Buzzard	Baskarp Cyclone
D_{10}, D_{30}, D_{50}, D_{60}: mm	0.63, 0.70, 0.80, 0.85	0.178, 0.377, 0.577, 0.688
Coefficients of uniformity, C_u and curvature C_c	1.36, 0.92	3.87, 1.16
Specific gravity, G_s	2.65	2.69
Minimum dry density, γ_{min}: kN/m^3	14.65	12.72
Maximum dry density, γ_{max}: kN/m^3	17.58	16.85
Critical state friction angle, ϕ_{cs}	34.3º	32.5º

load obtained, V_0 will determine the size of the yield surface. Next the vertical load was reduced to a chosen value at a rate of $\dot{w}$ = 0.01mm/s. Once the target value was reached, it was kept constant whilst the caisson was rotated at a rate of $2R\dot{\theta}$ = 0.01mm/s with a constant ratio between the moment and horizontal load, *M/2RH*. Tests were conducted for a range of vertical loads from V = -50N (tension) to V = 100N, and at *M/2RH* values between -2 to 2. These ranges were chosen by scaling typical prototype values such as those shown in Figure 1a. The ratio *M/2RH* can also be interpreted as the ratio between the height h where the horizontal load is applied, to the caisson diameter $2R$, *i.e.* $h/2R$. The horizontal force is the resultant of the wind, waves and current forces. The low vertical load was held constant to reproduce the self weight of a light structure (wind turbine), whilst rotation is applied to reproduce the environmental loads. Figure 4a shows the load path applied. Yield points were obtained from the curves of: $M/2R$ v. $2R\theta$ and H v. u, as the intersection of the two straight lines, see Figure 4b. On the other hand, incremental plastic displacement vectors were calculated from the slopes curves of: u v. $2R\theta$ and w v. $2R\theta$.

The mathematical formulation adopted for the yield surface is given by an expression that represents an ellipsoid. Such a surface y can be expressed by:

$$y=\left(\frac{H}{h_oV_o}\right)^2+\left(\frac{M}{2Rm_oV_o}\right)^2-2a\left(\frac{H}{h_oV_o}\right)\left(\frac{M}{2Rm_oV_o}\right)-f(V,V_t,V_o)=0 \qquad (1)$$



(a) (b)

Figure 4 – (a) Load paths for monotonic loading tests and yield surface derivation for low vertical loads; (b) Curves of loads (*M/2R, H*) versus displacement (*2Rθ, u*) and vertical displacement versus displacement (*2Rθ, u*)

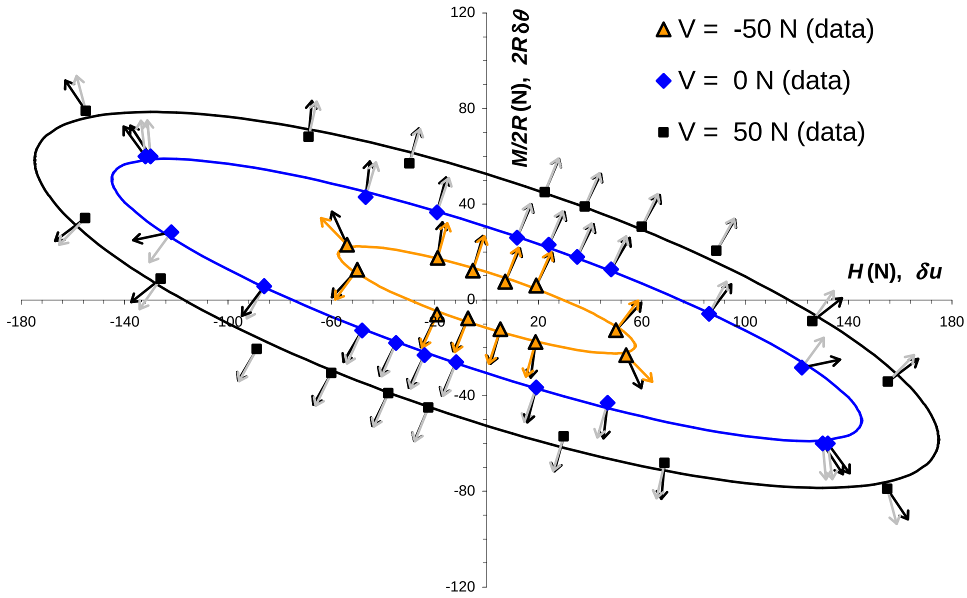


Figure 5 – Yield points fitted with ellipses curves in the M/2R – H plane and experimental and normal flow vectors for *V* = -50N, 0N and 50N. Aspect ratio caisson *L/2R* = 0.5

Where *a* is the eccentricity of the yield surface, V_o is the maximum pure vertical load, V_t is the maximum pure pull-out load and h_o and m_o are the horizontal and moment dimension of the yield surface. The elliptical curves illustrated in Figure 5 were fitted to the experimentally determined yield points, using the least square error method. A separate curve was fitted to each set of tests at a particular vertical load. This fit shows clearly how expression (1) agrees very well with experimental results.

The flow rule can be derived from the yield surface equation (1) using the following form:

$$\begin{Bmatrix} \dot{w}_M^p \\ \dot{\theta}_M^p \\ \dot{u}_H^p \end{Bmatrix} = \lambda \begin{Bmatrix} \partial y / \partial V \\ \partial y / \partial M \\ \partial y / \partial H \end{Bmatrix} \tag{2}$$

Where ($\dot{w}_V^p, 2R\dot{\theta}_M^p, \dot{u}_H^p$) correspond to the increments of the plastic displacements and λ is a positive scalar multiplier that accounts for the magnitude of these velocity vectors. Figure 5 shows the experimental flow vectors that represent the direction of the plastic displacements ($2R\dot{\theta}_M^p, \dot{u}_H^p$) in the *M/2R* v. *H* plane. The direction is similar to the vectors normal to the yield surface, demonstrating that an associated flow rule is valid in this plane. Associated flow in the *M/2R* – *H* plane has been experimentally observed previously (Martin, 1994; Gottardi *et al.*, 1999; Byrne and Houlsby, 2001). However, when experimental flow vectors are plotted in the *M/2R* – *V* plane they do not tend to follow the direction of the vectors normal to the yield surface, as can be observed in Figure 6. Further investigation is required to establish the correct form of non-associated flow rule.

3.2 Cyclic loading

The environmental loads are cyclic, therefore an investigation of foundation behaviour under cyclic loading was conducted under similar conditions to those already explained for the monotonic combined loading tests. The same loading rig, caissons and soil were used (see Figure 2 and Tables 1 and 2). Tests were conducted holding a constant vertical load whilst a

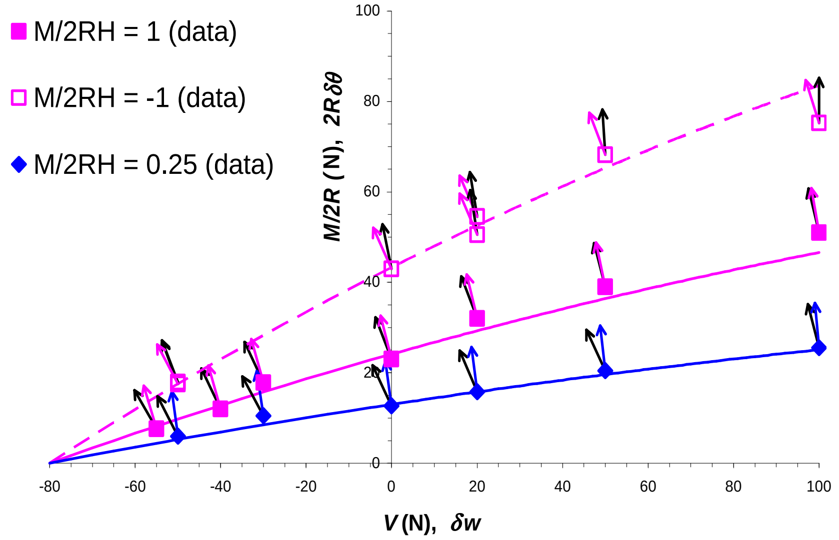


Figure 6 – Yield points in the *M/2R* v. *V* plane and experimental and normal flow vectors for *M/2RH* = -1, 0.25 and 1. Aspect ratio of caisson *L/2R* = 0.5

cyclic rotational displacement of increasing amplitude was applied for ten cycles. Tests were performed for a range of vertical loads from *V* = -50N (tension) to *V* = 400N, and at *M/2RH* values between -2 to 2.

Figure 7 shows ten rotational cycles applied to a caisson of diameter *2R* = 293mm at a rate of $2R\dot{\theta}$ = 0.02mm/s. The response is hysteretic and it is possible to observe stiffness degradation during each cycle. Figure 8 shows proof, however, that the shape of the cycles conforms to the second Masing rule, which states that the shape of unloading and reloading curves is the same as that of the initial curve, but doubled in both dimensions. The first Masing rule is also confirmed by Figure 8. This states that the tangent to the slope of the reloading curves is identical to the tangent to the slope of the initial curve. The confirmation that the Masing rules apply offers the possibility of a relatively simple interpretation of the data, since Masing rules correspond to pure kinematic hardening.

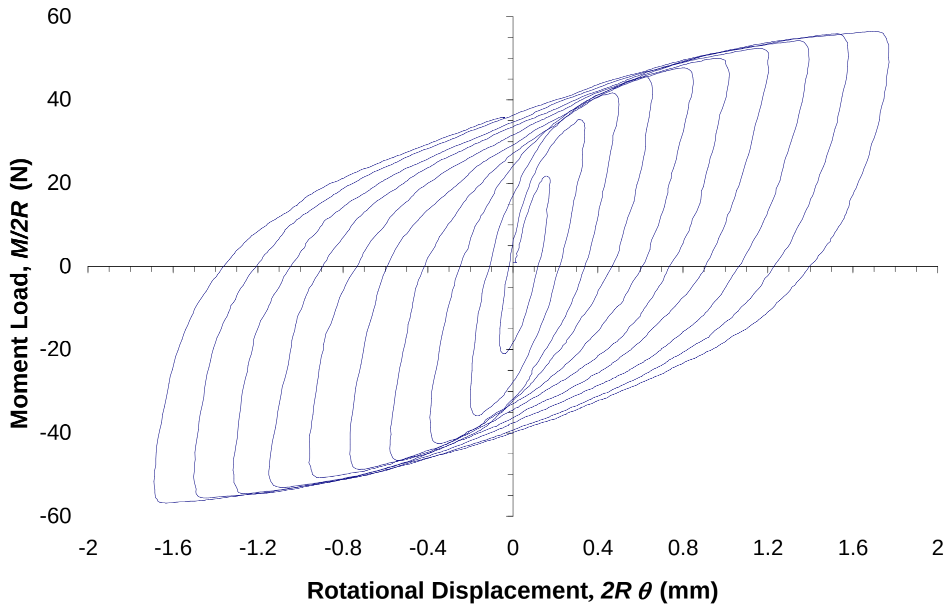


Figure 7- Typical cyclic rotational test. *V* = 50N, *M/2RH* = 1 and *L/2R* = 0.5

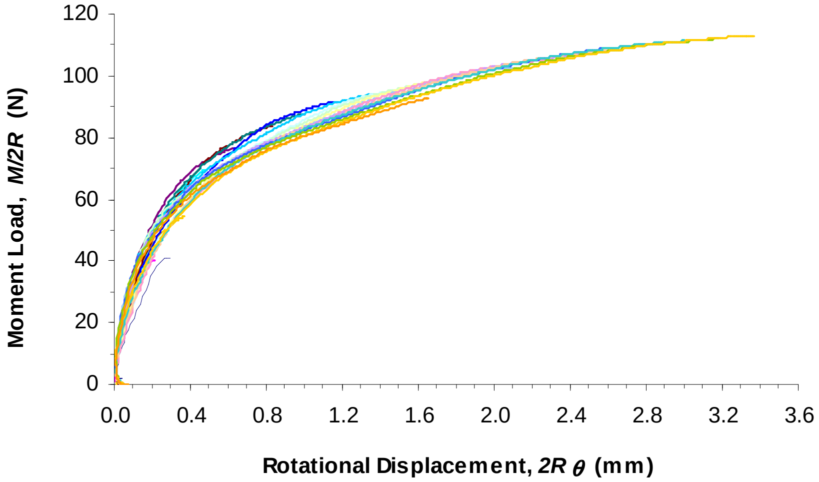


Figure 8 - Second Masing rule. The initial loading is doubled; reversals and re-loadings are relocated.

In tests with $V \leq 0$N the moment resistance approaches an asymptotic value. However, the remainder of the tests (*i.e.* $V > 0$N) show an increase in the moment resistance after each cycle. The moment response increases as V is increased. Furthermore, there was an uplift of the caisson in tests with $V < 100$N. The caisson rotated almost without vertical displacement at $V = 100$N. Settlement occurred for high vertical loads, $V \geq 200$N.

3.3 Moment capacity tests of a suction installed caisson

The tests described above consisted of model caissons installed in loose dry sand by pushing. In the field caissons are installed by suction. A study of the effect of suction installation on moment capacity was therefore performed. Two series of combined loading tests, one using each of the installation methods, were carried out on a model scale suction caisson (4th column in Table 1). Both series of tests were in dense oil-saturated Baskarp Cyclone sand. The properties of this soil are in the last column of Table 2. Using the 3DOF rig (Figure 2) the caisson was first penetrated to 20mm by pushing to form a seal with the soil, and then a constant vertical load was held whilst suction was applied to install the caisson into the ground. Once installed, moment loading tests were conducted using the following sequence:

(a) The footing was vertically displaced until a preset vertical load was reached.
(b) The vertical load was held constant for a period of time, to allow excess pore pressure (measured by a pore pressure transducer under the middle of the caisson lid) caused by the loading in (a) to dissipate.
(c) Rotational and horizontal movements were applied so that a load path in (V, $M/2R$, H) space was followed. A rotational displacement of $2R\theta = 0.5$mm was reached under a rate of $2R\dot{\theta} = 0.0005$mm/s. This corresponded to drained conditions.

The relative density was estimated by driving a small cone penetrometer into the sample at the tested site (Mangal, 1999). The average relative density was $R_d = 69\%$. Figure 9 compares the two moment tests showing that no significant differences in moment capacity are observed between the different methods of caisson installation.

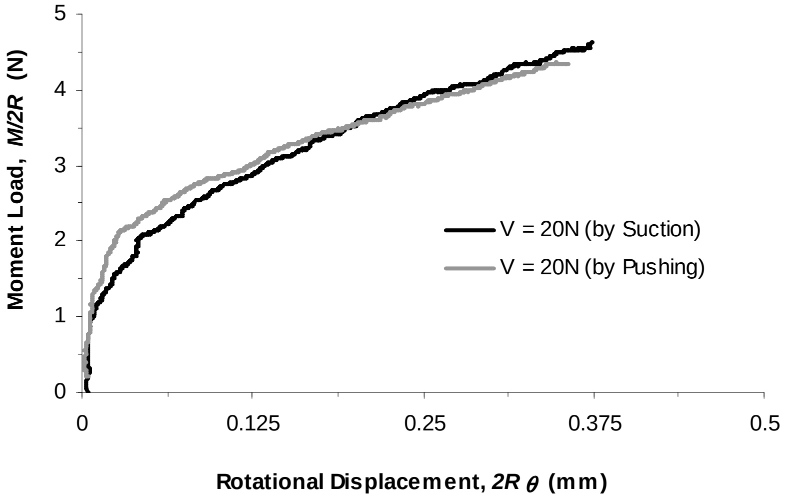


Figure 9 – Comparison of moment capacity for a caissons installed by different methods, *M/2RH* = 0.5 and *L/2R* = 0.5

CONCLUSIONS

A description of the research currently in progress to investigate suction caissons as an alternative foundation for offshore wind turbines has been presented. Laboratory testing has been carried out to provide the necessary data to construct and validate theoretical models. Field trial results are being used to assess the scale effect in the models. A hyperplasticity theory has been used to model monotonic and cyclic caissons response using multiple yield surfaces. This paper has focused mainly on laboratory testing, from which a yield surface and flow rule was determined, for two model caissons of different aspect ratios under low vertical load. The following conclusions are drawn:

- Monotonic and cyclic moment loading tests proved that higher moment resistance was obtained when the vertical load is increased. Furthermore, uplift of the suction caisson was observed under the action of moment loads when the vertical load was below a certain critical value.
- In cyclic tests a reduction of stiffness during each cycle was observed. Furthermore, all the tests obeyed the Masing rules. This makes the numerical modelling more straightforward since the entire response can be reproduced using the first loading part of the cyclic curve.
- Finally, analyses of the effect of the installation method on the moment capacity are in progress. Provisional results indicate that differences in capacity between the two methods are not significant.

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