Tensile Loading of Model Caisson Foundations for Structures on Sand

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ABSTRACT

The viability of multiple footing structures that use suction caisson foundations would be improved if the up-wind leg(s) could resist significant tensile loads. In the particular case of offshore wind turbines, large moments are applied at foundation level by the action of wind and waves on the structure. For a multiple footing structure the applied moment is resisted primarily by vertical reactions on opposing foundations. If a significant tension can be allowed at the upwind side, then the spacing of the foundations, and therefore the overall size of the structure, can be greatly reduced. Bye *et al* (1995) suggest that the tensile capacity of caissons is dependent on the rate of loading, compared to the rate of drainage of excess pore pressure beneath the caisson. To investigate this, model caisson tests were carried out in test beds made of two different gradings of sand. Whilst the ultimate capacity is large in both cases (and controlled by the cavitation of the pore fluid) the displacements required to mobilise the loads are large compared to the diameter of the footing. These displacements are of a magnitude that would cause serviceability problems. The experimental results suggest that the upwind footing should be designed for tensile loading no greater than (at most) the drained friction on the skirt.

INTRODUCTION

Within the next few years a number of wind farms will be constructed around the coast of the UK. In the first instance many of the wind turbine structures will be founded on piles. These foundations, although simple to design, contribute a significant proportion (about 30%) of the overall installed cost for these structures. Various options are being investigated to reduce the costs, and therefore increase the economic viability of offshore wind-farm developments. One possibility is use of skirted shallow foundations, installed by suction either as a single foundation (*i.e.* monopod structure) or as a multiple foundation system (*i.e.* quadruped/tripod structure) (see Houlsby and Byrne, 2000; Byrne and Houlsby, 2002b; Byrne *et al*, 2002; Byrne and Houlsby, 2003). For the monopod, the key issue is the performance of the foundation under the large moments applied by the wind and wave loads on the structure (for a discussion of this problem see Byrne *et al*, 2003). For the multiple footing case the applied moment loads from the wind and waves will largely be reacted as vertical compression and tension loads on the individual foundations. Both these loading cases are being investigated at the University of Oxford in laboratory scale and field scale tests funded by the Department of Trade and Industry (DTI), the Engineering and Physical Sciences Research Council (EPSRC) and industrial participants (SLP Engineering Ltd, Shell Renewables Ltd, General Electric Wind Ltd, Fugro Ltd, Aerolaminates Ltd, HR Wallingford and Garrad Hassan) (see Byrne *et al*, 2002; Byrne *et al*, 2003; Kelly *et al*, 2003). This paper concentrates on the tensile vertical capacity of a caisson foundation embedded in a saturated sand, relevant to the design of multiple footing foundations.

Previous research on the transient tensile capacity of skirted foundations in saturated sand has been conducted by Bye *et al* (1995), Johnson (1999), Byrne (2000), Byrne and Houlsby (2002a) and Kelly *et al* (2003). These studies have shown that significant tensile capacities are possible, under the appropriate loading conditions, and are limited by cavitation of the pore fluid. The displacement of the caissons required to generate the maximum tensile capacity was in the order of 5-10% of their diameter Byrne and Houlsby (2002a), and the stiffness of the response was observed to reduce when the load applied to the foundation changes from compression to tension.

Clearly the economic viability of multiple caisson foundations would improve if the upwind leg(s) of the structure can resist significant tensile loads at moderate displacements. The experimental evidence, as reported above, suggests that large upward movements of the caissons are required to develop significant tensile loads. These movements are likely to be of an order sufficient to prevent a wind turbine from operating. For a caisson foundation on sand the transient loading condition will be critical to understanding the load displacement response. The transient response of a foundation on sand is dependent on the rate of loading, compared with the rate of drainage of the excess pore pressures beneath the caisson. In particular it is necessary to understand which variables control aspects of the load-displacement behaviour, such as the stiffness changes at the tension/compression transition. Model caisson tests have been conducted on test beds made of very fine sand, and are compared with tests conducted using a coarser material. The loading rates have been chosen so that the response spans a range of partially drained conditions.

TEST APPARATUS

A model caisson of diameter 280mm, skirt depth of 180mm and wall thickness of 3.125mm was used in these experiments. The test chamber is shown in Figure 1, and comprises a watertight cylindrical vessel 1m in diameter and 1m in height. A saturated sand sample was constructed inside the test chamber. An Instron actuator was fixed to the lid of the test chamber to provide vertical load to the model caisson. The model caisson and a waterproof load cell were suspended beneath the lid of the test vessel and attached to the actuator via a stainless steel rod, which penetrated through the lid of the chamber via a watertight gland. The load applied by the actuator, outside the test chamber, was also measured by a 100kN capacity load cell fixed to the ram of the actuator. Pressures beneath the lid of the caisson and at the tip of the skirt were measured. Sensors near the top and the base of the cylinder monitored pressure within the chamber. The displacement of the caisson during loading was measured by an LVDT attached to the rod fixing the actuator to the caisson. During installation, the air within the caisson was vented through a tube from the caisson to the outside of the vessel. Once the caisson had been installed, a slight pressure was applied within the chamber in order to force water through this tube and remove all of the air within it. Signals from the instrumentation were filtered and amplified. The data was recorded by a PC-based system.

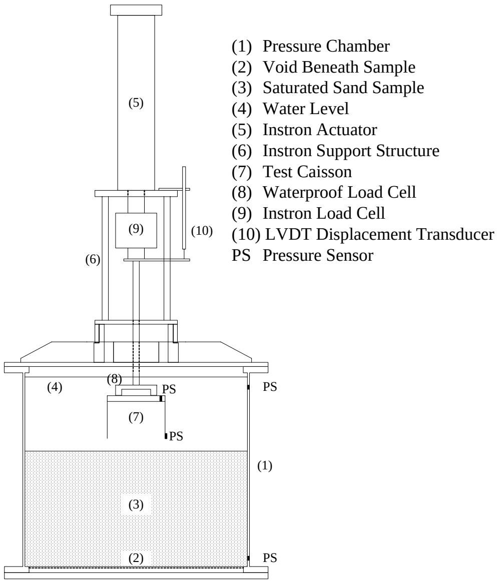

Fig. 1 Test Chamber

SAMPLE PROPERTIES AND PREPARATION

The sands used were Oakamoor HPF5 and Redhill 110. HPF5 is a crushed silica sand with d_{50} of 50µm whereas Redhill 110 is a sieved silica sand (or sand silt) with a d_{50} of about 120µm. For consolidation and drainage dominated problems, as in this paper, the d_{10} size is more critical. In the case of Redhill 110 the value for this parameter is about 75µm whilst the HPF5 is an order of magnitude finer at 7µm. A particle size diagram showing the grading of both sands is presented in Figure 2.

The Redhill 110 sample was prepared by first placing a filter layer of Leighton Buzzard 16-30 sand to a thickness of about 80mm. About 700kg of the Redhill 110 was placed dry on top of the filter layer. The pressure chamber was sealed and a vacuum applied within it. Carbon dioxide gas was introduced to the base of the chamber and allowed to

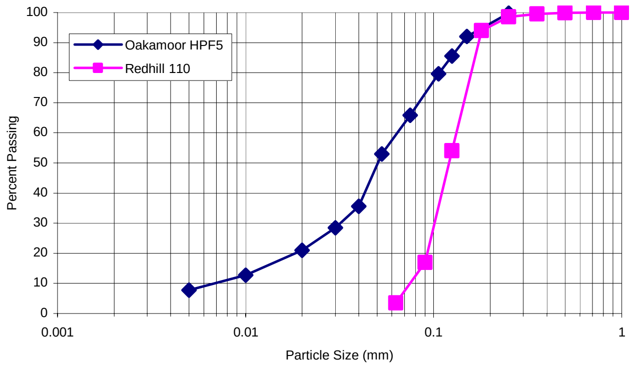

Fig. 2 Particle size distribution

flush through the sand for a period of time. De-aired water was then introduced to the base of the evacuated chamber until the test bed was entirely submerged. Carbon dioxide gas was again flushed through the sample, and the vacuum was gradually released. At this stage any small amount of carbon dioxide was expected to have dissolved in the water.

The HPF5 sample was constructed differently, to minimise hazard from fine silica particles if a dry preparation method was used. The Leighton Buzzard filter layer was again used together with a thin layer of Redhill 110. The chamber was then filled with water and the HPF5 sand pluviated through water into the chamber. The sand was introduced in stages and vibration applied to the walls of the test chamber to remove air trapped within the sand. Both sands were densified prior to the tests by use of a vibrating motor strapped to the side of the test chamber. The Redhill 110 sample was prepared to a relative density of 80% while the HPF5 test bed had a relative density of about 75%.

TEST PROCEDURE

The initial penetration of the caisson into the Redhill 110 was conducted at a rate of 0.2mm/s whilst in the HPF5 the rate was reduced to 0.05mm/s to prevent piping failures from occurring. This procedure was applied until the vertical load on the caisson was 10kN. Subsequently, step loads of 5kN were applied to investigate the rate of pore pressure dissipation beneath the caisson. The step loads were applied until the Instron load cell recorded 35kN. Load cycles were then applied with amplitudes of ±5kN, ±10kN, ±20kN, ±30kN, ±35kN and ±40kN. The cycles were applied at rate of 1Hz in the test reported here using the Redhill 110 sand and at rates of 0.1Hz and 1Hz in the tests using the HPF5 sand. (Other tests where the caissons were pushed into the sand until a load of 35kN was reached, and then rapidly pulled out of the sand at a rate of 100mm/s are not reported here).

The average times for the dissipation of half of the initial excess pore pressures beneath the lid of the caisson during the step changes was 0.04s in the finer Redhill 110 sand and 18.6s in the coarser HPF5 sand. The permeabilities of the samples were estimated from these data using the equation $T_v = c_v t / H^2$ with $T_v = 0.196$ as for 50% drainage in one-dimensional consolidation, and H taken as the depth of the caisson skirt. The permeability is then obtained from $c_v = k/\gamma_w m_v$, with m_v being estimated from the deflection during the load increment. The permeability of the HPF5 sand was estimated as $4\text{x}10^{-7}$ m/s and that of Redhill 110 sand $1.5\text{x}10^{-4}$ m/s.

RESULTS

Data from three cyclic tests are shown in Figures 3, 4 and 5. All of the test data show that as the cyclic load amplitude increased so did the displacement of the caisson into the sand. In the coarser Redhill 110 sand during the attempted ±40kN cyclic load set the caisson was able to sustain a tensile load of only about 1kN, whereas in the finer HPF5 sand, loaded at a rate of 0.1Hz, a tensile load of about 7kN could be sustained. The stiffness of the load-displacement response in tension was, however, much lower than that in compression in both tests. These results are consistent with tests conducted by Byrne (2000), and reported by Byrne and Houlsby (2002a), in extremely low permeability oil saturated fine silica sand. The tests were carried out using a footing of 150mm diameter and 50mm depth. An example of a cyclic loading test is shown in Figure 5. Note that the magnitude of the loads is significantly lower with a mean vertical load of 200N. A feature of all the cyclic tests is the accumulation of vertical (downward) displacement during the cycles, as well as the hysteresis in each cycle.

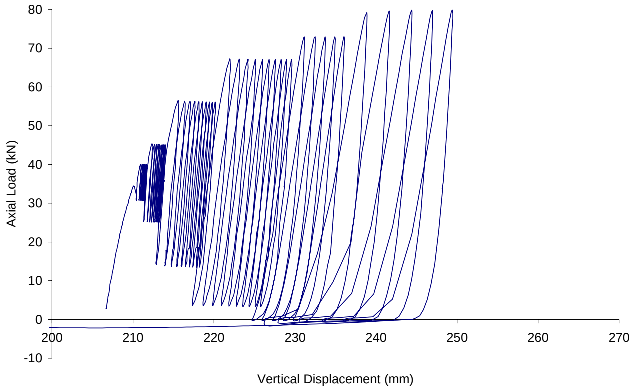

Fig. 3 Load-displacement data: Redhill 110 loaded at a rate of 1Hz

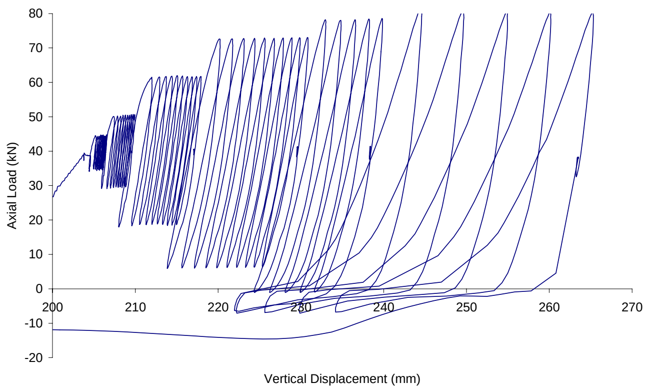

Fig. 4 Load-displacement data: HPF5 loaded at a rate of 0.1Hz

Another method of exploring the limits of the caissons behaviour, other than reducing the permeability of the soil, is to increase the rate of loading. The test conducted in HPF5 sand was repeated with the rate of loading increased to 1Hz. Data from this test is shown in Figure 6. The response of the caisson was not as stable as the other two tests. The mean compressive load appeared to change throughout the test and tensile load was apparently not mobilized although the stiffness of the load-displacement response during the ±40kN cyclic load set reduced significantly as the load approached zero. Extremely high pore pressures beneath the lid of the caisson were recorded during this test. The maximum pore pressure was 372kPa during the compressive half-cycle of the ±40kN load set. This pressure is equivalent to about 23kN, which is about one-third of the total applied load.

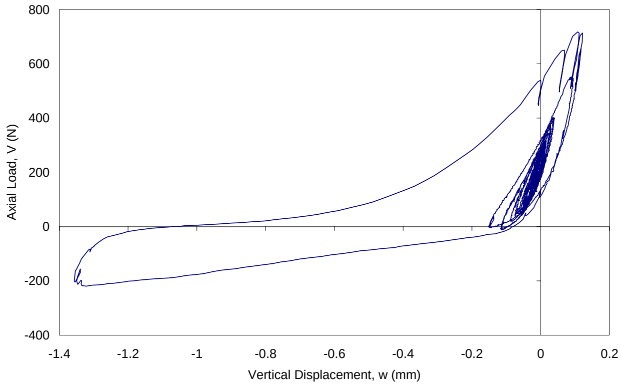

Fig. 5 Load-displacement plot from Byrne (2000).

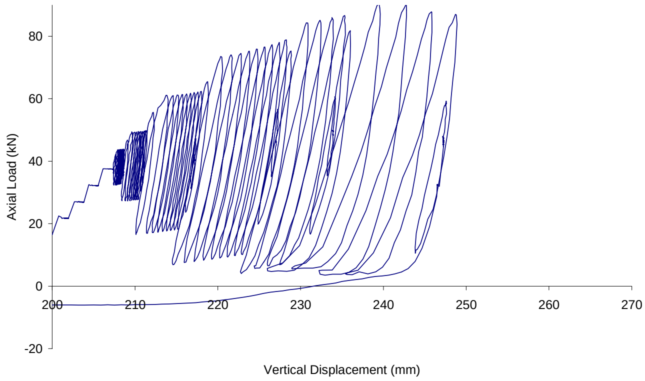

Fig. 6 Load-displacement data: HPF5 loaded at 1Hz

The cyclic tests, followed by pullout, were conducted to assess whether disturbance to the sand during cycling affected its tensile behaviour. A comparison of the results from the two load-unload tests is shown in Figure 7. Tensile loads of about 10kN were achieved in each test. The pore pressure recorded beneath the lid of the caissons in each test, at the maximum tensile load, was about –100kPa. This is the pressure at which cavitation of the pore fluid occurs in water. The tensile load due to the cavitation pressure would correspond to approximately 6.2kN. The difference between this value and the 10kN measured represents an enhanced friction due to the very large hydraulic gradient at the outside of the caisson skirt. The displacement required to mobilise the maximum tensile load was about 10mm in the Redhill 110 sand and about 20mm in the HPF5 sand. These displacements correspond to about 3.5% and 7% of the caisson diameter respectively. It is significant that the stiffness of the load-displacement response remained much less in tension than compression, even when the caissons were pulled out rapidly from low permeability sand, and prior to any disturbance caused by cyclic loading.

DISCUSSION

The transient tensile capacity of a skirted foundation depends on an interaction between the permeability of the soil, the length of the drainage path and the rate of loading. The non-dimensional parameter that incorporates these variables is T_v, the time parameter used in one-dimensional consolidation analysis:

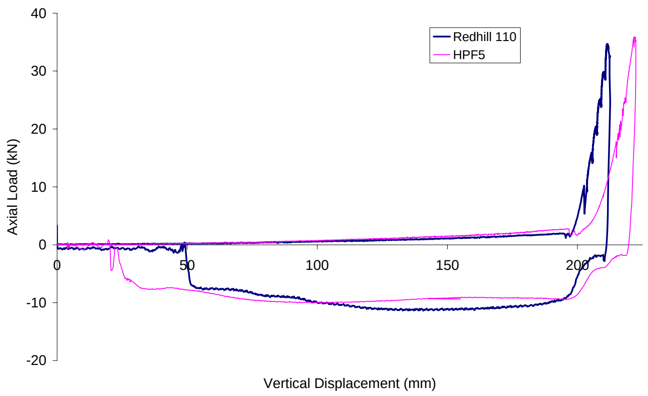

Fig. 7 Pullout of caissons from Redhill 110 and HPF5 sand

$$T_v = c_v \frac{t}{H^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where c_v is the coefficient of consolidation, t is the time taken for a certain degree of pore pressure dissipation and H is the length of the drainage path. This equation can be used to assess whether the loading rates applied in these tests are applicable to a prototype situation. The prototype caisson is assumed to have a diameter of 6m and a skirt length of 5m. It is assumed that the time t in the above equation represents the rise-time during a loading cycle (*i.e.* a quarter cycle), and that the length of the drainage path is equivalent to the length of the caisson skirt. A comparison of calculated T_v values is presented in Table 1. The values of c_v in Table 1 were estimated from the step load data in the model caisson tests. It has been assumed that c_v for the prototype sand is that as deduced for the sand at the Draupner platform site by Bye *et al* (1995), where caisson foundations have been used.

Table 1 Comparison of T_v values

Sand	c_v **(m²/s)**	t **(s)**	H **(m)**	T_v
Redhill 110	0.19	0.25	0.18	1.47
HPF5 at 0.1Hz	0.00039	2.5	0.18	0.03
HPF5 at 1Hz	0.00039	0.25	0.18	0.003
Prototype	0.5	2.5	5.0	0.05

The T_v value in Table 1 for the prototype caisson indicates that it might behave in a similar manner to the model caisson test in HPF5 sand, loaded at a rate of 0.1Hz. Note that in this test the caisson was able to sustain a modest tension during cyclic loading. Note, however, as shown in Figure 4, that the stiffness in tension is significantly lower than in compression. Also note that the footing penetrates into the ground substantially after each tensile excursion. If the loads do not pass into tension the displacement response is less severe.

If it is assumed that the tensile load mobilised is proportional to the cube of the dimension of the caisson (as would be usual for drained capacity problems in sand), then the maximum mobilised tensile load would be about 70MN. If, however, the capacity is proportional only to the square of the dimension (as would be true if cavitation were the dominant factor) then the capacity would only be about 3MN for a 6m caisson.

However, the low stiffness as the loads become tensile may impose a serviceability design limit, that the load on an upwind leg of a multiple caisson foundation must be limited to zero tensile load, or at most the drained friction value on the skirt. This condition may be required in order to prevent the caisson from ratchetting into or out of the ground, depending on the mean vertical load applied to each caisson. The laboratory-scale data suggests that the maximum tensile load possible before a caisson undergoes significant upward displacement is limited to the self-weight of the caisson, the weight of the soil plug and the external skin friction acting on the caisson's skirts.

Note that Figures 3, 4 and 6 indicate that, as long as tensile loading is avoided, there is little difference in the vertical load-displacement response across the wide range of T_v values shown in Table 1.

CONCLUSIONS

Cyclic loading tests of model caissons in dense sand are presented. These data are relevant to the design of multiple caisson foundation systems for offshore wind turbines, in which the design is likely to be limited by the tensile capacity of the foundations. Loading rates were carefully selected so that, when scaled to prototype scale, they represent realistic degrees of partial drainage under wave loading conditions. The results indicate that only small tensions can be permitted on the foundations if excessive vertical movement is to be avoided.

ACKNOWLEDGEMENTS

The authors are grateful to the DTI and EPSRC for the funding of this research. The authors would also like to acknowledge the industrial participants to this research project: SLP Engineering Ltd, Shell Renewables Ltd, General Electric Wind Ltd, Fugro Ltd, Aerolaminates Ltd, HR Wallingford and Garrad Hassan. The second author is also grateful for the support provided by Magdalen College, Oxford.

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