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Present knowledge about Laboratory Testing of Axial Loading on Suction Caissons

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Abstract: Offshore wind turbines are increasing in both efficiency and size. More economical foundations for such light structures are under investigation, and suction caisson was shown to be particularly suitable for this purpose. In multi-pod foundation configuration, the overturning moment given by loads on the structure is resisted by push-pull loads on the vertical axis of each suction caisson. Relevant works where this situation is examined by means of laboratory testing, are summarized in this article, then different conclusions are followed by discussion and comparison. In the initial theoretical section, an overview of phenomena related with the case of study is presented. Drained and undrained condition, liquefaction and suction are examined from the theoretical point of view for mechanisms related to the case of study.

1. INTRODUCTION

Wind turbines are usually founded on piles, these foundations are of simple design but take about 30% of the total budget. Suction caisson foundations are an option that can decrease the overall cost and increase the diffusion of wind turbine. Since wind turbine are dynamically sensitive structures where stiffness requirements have to be satisfied, an alternative design allowing to increase stiffness is multi-pod configuration *(Byrne 2002)*, wherein loading response changes significantly with respect to a monopod. The following work is focused on loading of multi-pod foundation, where very little moment is taken by the suction caisson and the moment load is mainly resisted by push-pull load on the vertical axis of opposite suction caisson. For these reasons, it is important to understand behavior under tensile loading and improve the stiffness of foundation, so a correct design can be established. Among others, multi-pod foundations can be both tripod or tetrapod. Tripod has the advantage that it requires less material and it is easier to construct and install.

This review has the purpose to analyze research on vertical loading of suction caisson installed in sand, focusing on works done in laboratory. Cyclic and monotonic pull-out tests are reported, specifying equipment used and test modality adopted in order to discuss and compare works of different authors. It is recognized that the design of a wind turbine foundation is not driven by the ultimate capacity but it is governed by parameters as stiffness and behavior under cyclic loading, so particular attention has been given to these topics. Important matter is the enhancement in resistance to pull-out load given by pore pressure under the lid of the caisson. This resistance is a consequence of a complex interaction between permeability of the soil, drainage path and rate of loading, and is a resource on which can possibly contribute to peak load resistance. However a study needs to be done to have a more precise model of this phenomenon.

2. THEORETICAL OVERVIEW.

2.1 Laboratory Testing

Laboratory testing is a fundamental step of the assessment of the design procedure, inasmuch allow to test in a controlled environment phenomena of interest, and on which will be based the prototype design. Several types of laboratory setups were designed in order to test offshore foundations. Most of them examine the behavior of models which are about 100 times smaller than the full-scale foundations. Among the best known, are 1g and centrifuge tests which will be compared in this section.

In 1g models body forces cannot be modelled with a scale factor of one, friction angle is higher than the one in real size and Young modulus is lower. Since load-displacement response of sand depends also on void ratio, real condition can be reproduced in a scaled model reducing the density of the sand *(Randolph 2011)*.

Centrifuge testing allows body forces to be modelled properly. In non-centrifuge small scale tests, stress-dependent behavior is modelled at low value of body forces, at which soil can show a different behavior and measurements have to be really accurate in order to get reliable results. Effective stress level in a centrifuge test is equal to the one of the prototype, strength ratio (shear strength over effective vertical stress) and stiffness ratio are scaled with a factor of 1 *(Mangal 1999)*.

Drawback of centrifuge testing is the time scaling factors, that are $\frac{1}{N}$ and $\frac{1}{N^2}$ respectively for dynamic and seepage timing, where N is the acceleration level. To overcome this problem permeability of the soil has to be decreased, by increasing the viscosity or changing the grain size. Darcy describes the velocity for a laminar flow as $v = ki$ where the permeability is given by $k = K\frac{\gamma_w}{\mu}$ where γ_w represent the soil unit weight and μ the dynamic fluid viscosity. Since γ_w

increases linearly with the g level, μ has to be increased in order to reduce the permeability and keep the fluid velocity proportional to the prototype, therefore silicon oil is usually used in order to proper simulate the fluid flow through the soil

The capacity of centrifuges is given in g-tons, calculated as the multiplication of the maximum acceleration for the maximum package mass that fit in the centrifuge. The acceleration level is chosen in proportion to the depth that has to be modelled, dividing the height of the prototype for the height of the model.

Scaling relationship for 1 g and centrifuge tests is schematized in the following image, showing that in the centrifuge test body forces are scaled with a factor of 1 and distance is inversely proportional to the scaling factor.

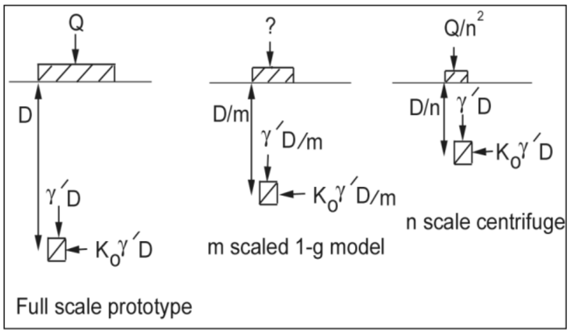

Figure 1. Scaling relationships for 1-g and centrifuge models (Murff 1996).

2.1 Drained and Undrained Condition

According to effective stress principle, after the application of a load, the drained condition occurs when the change in effective stress is equal to the change in total stress while the undrained condition occurs when the change in effective stress is equal to the difference between total stress and pore pressure. The intermediate state, partially drained conditions, occurs when the rate of volume change is greater than the flow rate of the fluid between the voids. Hence a variation of effective stress can be observed during the period of load application.

In a wind turbine foundation, various conditions can occur, depending on the soil permeability, the drainage length, and the rate of loading. When a suction caisson installed in soil with low hydraulic conductivity is pulled out at high rate of loading, the trapped soil has an undrained behavior. In this case, theoretically, pore pressure developed below the lid corresponds to the applied pressure (tensile load divided by the area of the lid) and is limited by cavitation. Therefore the uplift resistance is given by the self-weight of the caisson plus external skirt friction and the weight of the soil plug trapped inside the caisson. Drained behavior, instead, is generated by high sand permeability and low rate of loading. In drained condition, uplift resistance is given by the self-weight of the suction caisson, plus internal and external skirt friction. In undrained condition, the uplift resistance is generally greater than in drained condition.

In dense sand the expected behavior is of partially drained condition. Thus suction can occur below the lid, which increases the resistance capacity. The degree to which sand has a partially drained behavior, depends on the geometry of the caisson, the rate of loading, and drainage and deformation characteristics of the soil.

2.2 Liquefaction

In saturated sand, cyclic loading at relatively high frequencies can bring to an undrained behavior where pore water supports the load causing a decrease of effective stress. If the cyclic loading is rapid enough to not allow complete dissipation of pore water pressure, the latter can cause the effective stress going to zero and bringing the sand to a liquid state with low shear strength. This is the condition of soil liquefaction wherein sand has characteristics similar to those of a liquid. Even if effective stress is not zero, failure can occur because of the reduction in shear strength.

Liquefaction can often occur in loose sand, where cyclic loading creates a contractant behavior of the soil, causing a decrease in volume and an increase in pore pressure that cannot dissipate in undrained conditions. Generally, high void ratio and low confining pressure brings to a more rapid liquefaction. Time required for liquefaction is inversely proportional to the strain caused by cyclic loading, so the more strain is developed during each cycle, the less cycles are required to bring soil in a state of liquefaction.

During installation with suction, an upward flow of fluid is generated, and as a consequence an upward hydraulic gradient is formed inside the caisson. If difference in pressure is high, upward forces can exceed downward forces reducing to zero effective stresses, resulting in a liquefaction of the soil. This condition occurs when the critical hydraulic gradient is reached or exceeded. Critical gradient "i" is defined as the ratio between the effective unit soil weight and the unit weight of water $i = \gamma_s'/\gamma_w$ *(Roy 2010).*

2.3 Suction

On the studied cases, differentiation has to be made between active suction and passive suction. To install the suction caisson, active suction is created by means of pumps, and cannot be increased once the pump is disconnected. Passive suction is build up under the lid of the suction caisson as a consequence of upward displacement caused by loading.

Active suction during installation in sand establishes a flow in the soil surrounding the caisson. This flow reduces the vertical effective stresses of the skirt tip and on the interior of the caisson. Development of the upward hydraulic gradient inside the skirt reduces the side shear between soil and steel, while the downward flow of water outside the skirt increases the side shear, facilitating the penetration.

In model testing, a gradient close to the critical gradient is required to permit suction installation. This reduces the penetration resistance but, if is not correctly evaluated, piping failure can occur preventing a complete penetration. This phenomena in field installation is avoided to some extent using water jetting or dredging pumps.

Installation in laboratory can be done also by pushing. It requires less equipment and do not give problems of active suction installation, such as liquefaction and creation of sand heave below the lid. The latter phenomenon occurs if the penetration resistance is not in equilibrium with active suction pressure, causing a deformation of the soil skeleton of which mechanism is not fully understood, and can cause a not complete installation of the model suction caisson with consequence on the test response *(Tran 2005)*.

During tensile loading passive suction is creating a gradient in the same direction of the one of installation but, since the displacement is upwards, the gradient is acting in favor of resistance on the skirt friction. In drained to partially drained condition, the pressure gradient created between the lid and the bottom of the caisson creates a fully developed seepage flow from outside to inside the caisson. As consequence, the internal skirt friction is lower than the external one because internal effective stresses are reduced by the upward gradient, while external effective stresses are increased by the downward flow. In partially drained to undrained condition, the soil plug remains trapped within the caisson. Dilation occurs on the internal side of the skirt and in the area beneath the caisson, causing negative pore pressure and therefore a downward seepage also inside the caisson. As result, the uplift capacity increases due to the enhancement given by frictional resistance also on the inside of the skirt.

Enhancement of negative pore pressure is given also by dilatancy. If soil is in undrained condition, dilatancy can be fully developed, increasing resistance. This is not the case in most of the loading condition in dense sand, where there is a partially drained behavior instead. Therefore in partially drained condition, dilatancy has a reduced effect on pore pressure, since drainage results in volume deformation. From this consideration is it possible to infer that passive suction is inversely proportional to the degree of drainage and directly proportional to the rate of loading.

3. STATE OF THE ART.

3.1 Investigations of Suction Caissons in Dense Sand (Byrne 2000).

Equipment features

In this work a three degree of freedom loading rig was initially developed to test footings on clay (Martin 1994). At a later stage it was modified in order to cope with greater stiffness and displacement rates required for tests in sand (Mangal 1999). Load or displacement were applied by a computer controlled stepper motor and measured with high accuracy (±2N, ±2μm). *Figure 2* shows the loading rig, details are given in Martin 1994, Gottardi and Houlsby (1995), Mangal (1999) and Byrne (1999).

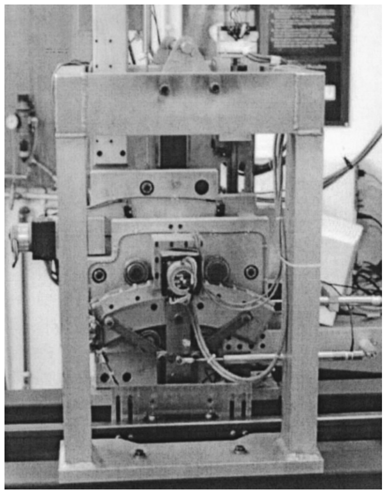

Figure 2. Loading rig (Byrne 2000).

Different loading programs were tested, tensile behavior was investigated in oil saturated sand samples, so in the following only these vertical load cases are discussed.

A tank of 1100 mm diameter and 350 mm depth was used to test dry and oil-saturated dense sand. This diameter has been considered large enough to allow performing multiple tests on the same sample of sand. Several testing of vertical loading behavior were made and are summarized in *Table 4.1* and *Table 4.5* in *Byrne(1999)*, respectively for cyclic and monotonic tests. All tests were carried out with a suction caisson model with a diameter of 150mm and skirt length of 50mm (aspect ratio = 0.33). Pore pressure was measured with one pressure transducer positioned at the center below the lid, and two on the perimeter of the caisson.

Tests have been made in oil saturated samples of Baskarp Cyclone sand *(Byrne 1999 Table 2.3)*, and prepared with a systematic procedure *(Byrne 1999)*. A vacuum was applied at the top of loose sand before to vibrate it, so that full saturation was reached. Then alternating downward gradient and vibration, the wanted density was reached. Density in a range of 80-95% was estimated from CPT test by empirical formula from *Mangal (1999),* and drainage properties were evaluated with consolidation tests. Suction caisson model was installed at a speed of 0.05mm/s, keeping the valve on the top open so no piping failure occurs. Once a preload of 75N has been reached the valve were closed and the sample unloaded to 0N.

Sand samples were not prepared for each test and more tests were carried out on the same sample instead, every time loading with greater mean vertical load and then starting the test. For example in a typical cyclic test the footing was loaded with a sequence as: 100 ± 25N, 100 ± 50N, …. , 100 ± 250N. This means that for most of the test the soil was on the elastic region, and only at the beginning of the sequence it reached the virgin curve. This represents the real physical situation, where extreme events are causing a small plastic deformation and then the loading remains in the elastic region.

Cyclic tests

In cyclic tests, a cyclic load was applied with "Constrained New Wave" method *(Taylor et. al., 1995)* that ensures extreme events to be included in the random simulation. 100 cycles with 4 extreme events were applied at each test. Comparing tests where three different loading programs were applied, respectively "Constrained New Wave", modulated sine wave, and stepped sine wave, it can be seen that there is no substantial difference in results. Despite that, "Constrained New Wave" method was used in most of the tests, because it reproduced the actual physical loading on the foundation.

Large number of tests were carried out, in *Table 4.2*, *Table 4.3*, and *Table 4.4* in *Byrne(1999)* these tests are subdivided by relevance to the study of, respectively, frequencies, loading history and cyclic load ratio. Typical cyclic response is asymmetric, showing vertical load mobilized at greater displacement in tension with respect to compression. The load-displacement response changes gradually to an asymmetric response as the load moves closer to tension, see *Figure 3*.

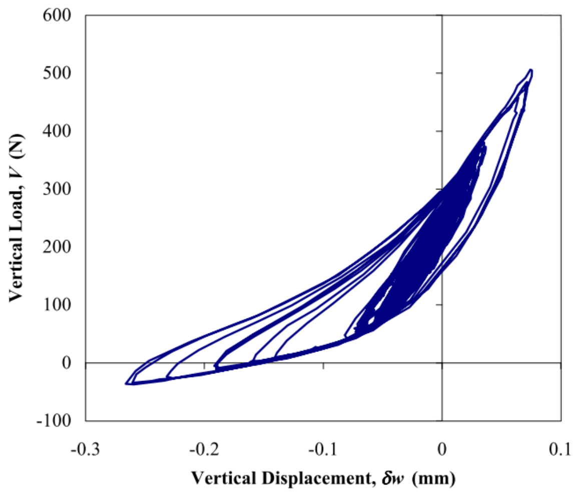

Figure 3. Asymmetrical cyclic response (Byrne 2000).

The range of period tested were from 1s to 30s and in all cases the response did not show any relevant change in behavior. Pore fluid response was relevant in both short and long period test. Longer periods were allowing the control system to have a better control on the loading and measuring devices, which in turn were allowing to reach a greater tensile displacement. In the long period test 1 mm displacement was mobilized at 200N, a displacement that was not reached in tests with faster period.

Monotonic pull-out test

Monotonic tests were carried out prescribing displacement and velocity. Tests were made for small and large displacements.

Effect of loading rate was analyzed applying small displacement, varying the rate of pull-out. A displacement of 1mm was applied at five different pull-out speed from 0.00086mm/s to 5mm/s in a sand sample with a relative density of 79%. The tensile capacity calculated was around 15N. This value is significantly lower than laboratory responses, meaning that, at small displacement, there was a partially drained behavior for all loading rates. In every test, most of the load was carried by pore fluid as shown in *Figure 4* and there was a little variation of pressure applying different load rates.

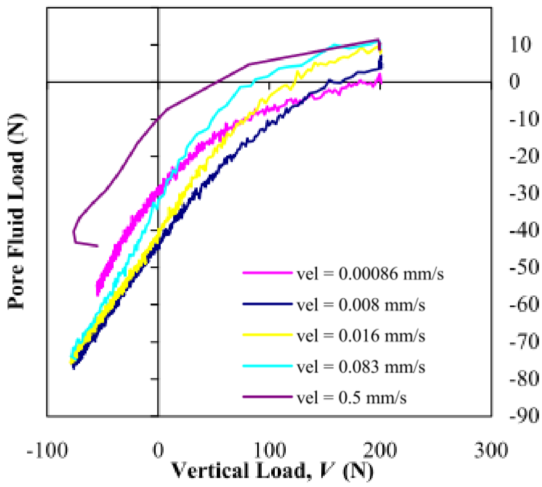

Figure 4. Pore pressure development (Byrne 2000).

Tests at small upward displacement with different loading history were carried out. Repeated pull-out tests on the same sample were showing a gradual decrease of the response, due to the loosening of the sample. An increment of tensile capacity was noticed after that a loading history causing redensification of the soil was applied.

Pull-out tests in loose sand were showing a softening response in the initial stage of loading, followed by a stiffer response. To analyze this softening behavior, small and large displacement tests were carried out in a soil with a density of 94%, with the 150mm caisson, at different rates. Applying repeatedly small displacement (1mm) on the same sample, no degradation of response and no rate dependency appeared. It was noticed that the behavior was partially drained also for low rates of loading. For this reason, the a response was significantly higher than the drained capacity, since partially drained behavior allowed also at the pore pressure to carry the load. Remaining within serviceability requirements, greater displacements were applied repeatedly, and the response is showing a progressive degradation till the drained capacity is mobilized (weight of the soil plugged into the caisson plus contribution from external friction).

Large displacement tests were carried out with constant pull-out speed of 2mm/s. The initial softening behavior was studied applying small displacement, where a response independent from the rate of loading was noticed. As larger displacements were applied, within the limit of softening behavior, the response become rate-dependent showing greater stiffness for high pull-out rates. When total pull-out was reached, after the softening response it was noticed a rate-dependent stiffer response, associated with dilation due to shear. In this latter response the stiffness was controlled by the velocity at which the water moved within the soil matrix to equilibrate the pressure difference created by the volume change that was occurring. The ultimate capacity was mobilized at large displacements, and was limited by cavitation. For this reason was suggested to design the tensile capacity on the initial softening response (Byrne 2000).

As can be seen in *Figure 5*, the total load response was greater than the pore fluid response. This gap is due to external friction, enhanced by a downward hydraulic gradient.

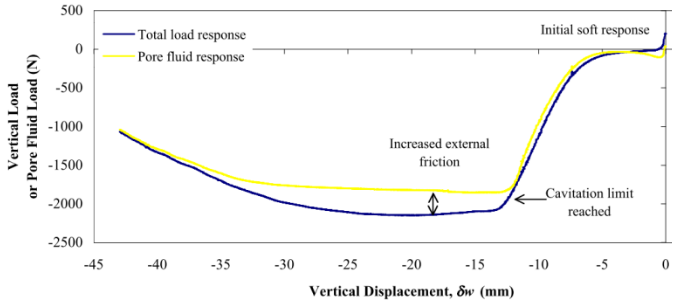

Figure 5. Response of high displacement pull-out (Kelly 2003).

Skirt effect was also analyzed, comparing pull-out tests of two footings with 100mm diameter and aspect ratio of 0 and 0.16. Tests were carried out at velocity of 2mm/s and the footing was preloaded with 100N load. Despite the small skirt, there was a great improvement of tension capacity due to the longer drainage path, and cavitation limit was reached using the 0.16 aspect ratio caisson.

3.2 Pressure Chamber Testing of Model Caisson Foundations in Sand (Kelly 2003).

Equipment features

Tests were carried out in a cylindrical pressure chamber (*Figure 6)*, 1m diameter and 1m high, designed to develop a maximum pressure of 200kPa. Loads or displacements were applied by a hydraulic actuator, installed on the lid of the pressure chamber. The actuator had a capacity of 100kN and a maximum rate of load-controlled cycling frequency of 10 Hz.

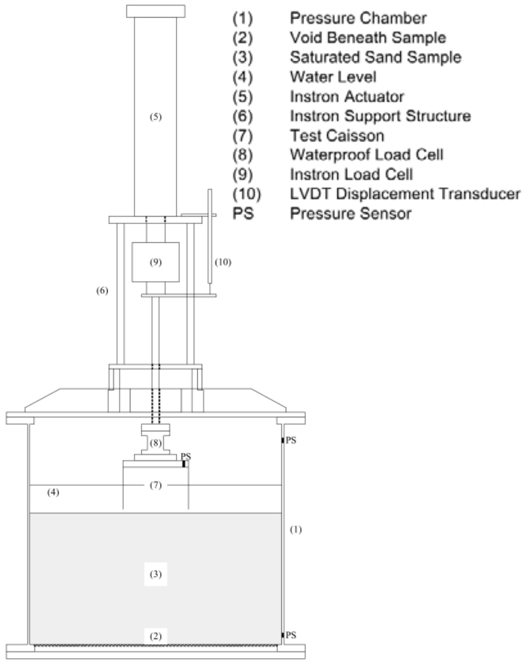

Figure 6. Pressure chamber (Kelly 2003).

A 100kN capacity load cell was used to measure the load. Displacements were measured by a system of Linear Variable Differential Transformer (LVDT). Two pressure transducers were installed in the pressure chamber. One was fitted at the top and the other at the bottom, so comparison could be made and hydraulic gradient could be measured. On the model caisson the pressure was measured by two pressure transducers, installed beneath the lid and at the tip of the skirt.

Model caisson was made of aluminum, it had a diameter of 280mm and a skirt length of 180mm (aspect ratio of 0.64). Caisson's skirt had a thickness of 3mm and the lid was 28mm thick. A vent valve was installed on the lid, in order to prevent water pressure building up during installation phase.

Tests were carried out with sand Redhill 110, sieve test results are shown in *Kelly (2006)*. The sand was vibrated in order to reach a D_r=80%. Sample preparation process is reported in *Kelly et al. (2003).*

Testing

The caisson was installed with a velocity of 0.2mm/s, till a compression load of 30kN. Tests were carried out at a frequency of 1Hz. 10 cycles were applied with amplitudes of ±5kN, ±10kN, ±20kN, ±30kN, then 5 cycles were applied with amplitudes of ±35kN and ±40kN. In between of each set of cycles the pore pressure was allowed to dissipate. After the last set of cycles a pull-out test was carried out at a rate of 5mm/s. Due to the rate of loading and permeability property of the sand, the behavior of the soil was drained to partially drained.

During the cyclic tests, low tensile loads were reached: on the ±40kN amplitude set of cycle, only -1kN was mobilized on a target of -5kN. As the load goes into tension there was a dropping of stiffness as can be seen from *Figure 7*.

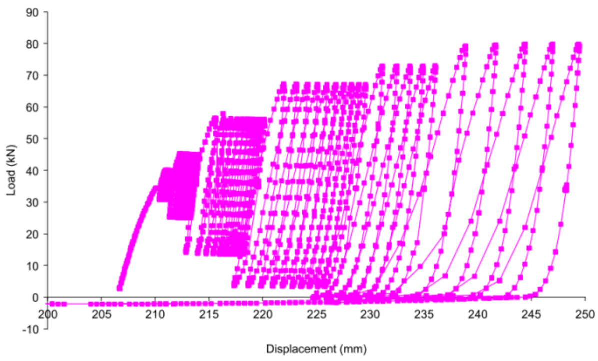

Figure 7. Cyclic loading followed by pull-out (Kelly 2003).

As shown in *Figure 7*, during pull-out tests, maximum tensile load reached was 2.1kN. After the test all the soil remained plugged into the caisson. Maximum tensile load was reached at a vertical displacement of 10-20% of the caisson's diameter. These deformations are too high to satisfy serviceability requirements, so it is suggested that tension limit can be limited to the weight of the caisson plus the weight of the soil plugged inside the caisson and the external skirt friction.

Increasing ambient pressure did not have any effect in these tests, because minimum pore pressure reached under the lid was far from the cavitation limit. The pore pressure may be dependent on the rate of loading and may approach the cavitation limit as rate of loading is increased.

3.3 Transient Vertical Loading of Model Suction Caisson in a Pressure Chamber (Kelly et al. 2006b).

Equipment features

Testing rig and caisson model were the same as utilized by *Kelly et al. (2003)* at Oxford University. Tests were conducted in pressure chamber using two different sands. Redhill 110 silica sand is the more permeable and it was used to investigate behavior in drained to partially drained conditions. Oakamoor HPF5 is an artificially created sandy silt, and it was used to analyze behavior in partially drained to undrained conditions. Different sands were prepared following different methods of which step by step description is reported in *Kelly et al. (2006a and 2006b*). Redhill 110 was vibrated till a relative density D_r*=80%* with $\phi=43.9°$, density of Oakamoor HPF5 was varying from *53%* to *73%* with $\phi=48.4°$.

Testing

Caisson was installed by pushing it into the sand at different speeds depending on the sand. In Redhill 110 the caisson was installed at a rate of 0.1mm/s, in Oakamoor HPF5 installation started with a rate of 0.05mm/s and ended with a velocity of 0.02mm/s. In all tests Installation ended when a preload of 35kN was reached, except for tests number 13 that was preloaded with 15kN.

Each cyclic test consisted of different packets of sinusoidal cyclic loads applied on the vertical axis of the caisson. At the end of the test, the caisson was completely pulled out from the sand. Most of the cyclic tests were made applying a different constant load frequency and varying amplitude, or varying both amplitude and frequency. Two cyclic load tests were carried out with large number of cycles at constant frequency and amplitude, but installing the caisson at different preloading loads. Push-pull tests were carried out pushing the caisson into the sand by steps of 10kN, so dissipation of pore pressure inside the caisson could be investigated, then pull-out displacement was applied at different speeds, varying in a range of 5 - 100mm/s, depending on the test.

Tests carried out in Redhill 110 are summarized on the table below *(Kelly et al. 2006b).*

Test	Date	Ave. Sample Height (m)	Dry Unit Weight (kN/m3)	Relative Density	Pressure (kPa)	Cyclic Frequency Hz	Pullout Rate (mm/s)
1	03/04/2003	0.57	15.7	0.79	0	0.5	5
2	11/04/2003	0.56	15.8	0.81	0	1	5
3	28/04/2003	0.56	15.9	0.82	0	-	100
4	30/04/2003	0.55	16.2	0.88	0	-	100
5		0.55	16.3	0.89	0	10	100
6	25/07/2003	0.56	15.8	0.80	200	1	100
7	30/07/2003	0.56	15.8	0.80	0	Multi	100
8	01/08/2003	0.56	15.8	0.80	200	Multi	100
9	04/08/2003	0.56	15.9	0.81	0	-	100
10	05/08/2003	0.56	15.9	0.82	200	-	100
11	06/08/2003	0.56	15.9	0.81	0	-	5
12	07/08/2003	0.56	15.9	0.82	200	0.5	100
13	08/08/2003	0.56	15.8	0.80	200	0.5	100

Tests done in sand Oakamoor HPF5 are summarized in the table below *(Kelly et al. 2006b).*

Test	Date	Dry Unit Weight (kN/m3)	Relative Density	Pressure (kPa)	Cyclic Frequency Hz	Pullout Rate (mm/s)
14	25/11/2003	15.1	0.53	0	1.0	5
15	26/11/2003	15.2	0.55	0	0.1	10
16	28/11/2003	15.8	0.67	0	-	100
19	16/03/2004	15.8	0.68	200	0.1	25
20	24/03/2004	15.8	0.67	0	1.0	25
21	26/03/2004	16.1	0.73	0	10.0	25
22	29/03/2004	16.1	0.72	200	1.0	25
23	31/03/2004	16.1	0.72	200	-	25
24	01/04/2004	16.0	0.72	0	10.0	25
25	13/04/2004	16.0	0.72	0	0.1	25

Analyzing cyclic tests in sand Redhill 110, total displacement at the end of each cyclic test is downwards, displacement per cycle increases with load amplitude, and it is greater in the first cycle of every set. The tensile capacity reached was small, in fact, on a target tension load of -5kN only -1kN was mobilized (*Figure* 8). If cavitation limit was not reached, tensile capacity was not affected by the ambient pressure. Varying the loading rate did not affect significantly the load-displacement response. For all cyclic load amplitude, the pore pressure increased increasing rate of loading and load amplitude.

In the two long cyclic tests 'shakedown' effect was noticed. This effect is common for cyclic loading on sand and it causes a decrease in displacement for each cycle as the number of cycles in a series increase. Was not present a significant pore pressure accumulation, and comparison with previous cyclic tests showed that large number of cycles do not affect the load-displacement curve.

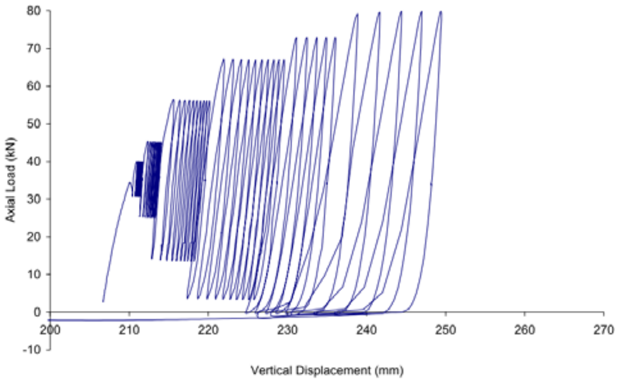

Figure 8. Cyclic loading in Redhill 110 loaded at a rate of 1Hz (Kelly et al. 2006b).

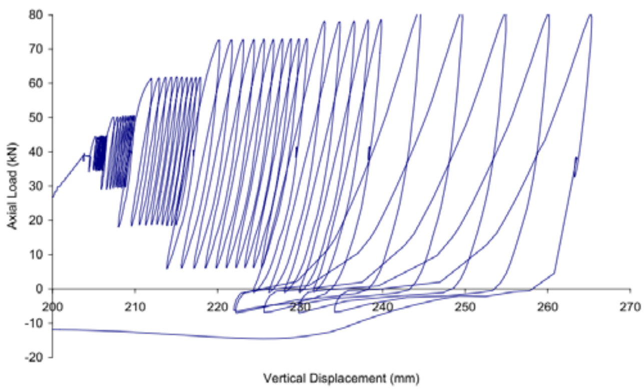

Figure 9. Cyclic loading in Oakamoor HPF5 loaded at a rate of 0.1Hz (Kelly et al. 2006b).

Cyclic tests in sand Oakamoor HPF5 were showing a behavior similar to tests in Redhill 110. Therefore axial stiffness reduced in tension, and ambient pressure did not affect tensile capacity, but the latter reaches a value of -7kN so greater than tests with higher permeable sand. This was due to passive suction developed that was higher than in Redhill 110, and was resisting about 50% of the applied load, against 15% of load resisted by passive suction in tests with sand Redhill 110. Shakedown effect was noticed, as displacement decrease with increasing number of cycles. As for tests in Redhill 110, cyclic amplitude increased as the cyclic load was increasing, and there were greater displacement in cycles where total load were approaching to zero *(Figure 9)*.

Increasing the rate of loading was causing a decrease of downwards accumulated deformation, and an increase in pore pressure development under the lid. The top pore pressure was reached in test 21, at a loading rate of 10 Hz and cyclic amplitude of 25mm/s, where the pore pressure exceeded 350kPa going out of scale.

Push-pull tests in sand Redhill 110, showed that the tension capacity was affected by the rate of loading and was increasing with the increase of ambient pressure. Therefore, unlike cyclic tests, monotonic test was dependent on ambient pressure. An ultimate tensile load of 10kN was mobilized at a displacement of 10mm, corresponding to 3.5% of the diameter of the caisson.

In pull-out tests in Oakamoor HPF5 sand where 5mm/s, 10mm/s, 25mm/s pull-out speed were applied, pore pressure and tensile capacity were increasing with the pull-out velocity. Ultimate tensile load of 10 kN was mobilized at a displacement of 7% of the caissons diameter. The maximum load was related to the rate of pull-out and limited by cavitation, so greater pull-out velocity and ambient pressure were allowing larger loads.

3.4 A Comparison of Field and Laboratory Tests of Caisson Foundation in Sand and Clay (Kelly et al. 2006a).

Equipment features

Same equipment and preparation procedure of *Byrne (2000)* were used in this work. Tests were carried out with sand Redhill 110 and prepared with relative density in the range of 70% to 84%. Two caisson models with aspect ratio of 0.66 and different diameters of 20mm and 15mm were used to carry out tests where vertical cyclic loading was applied.

Testing

Different modality of installation were applied in tests with 15mm diameter caisson. In one test the caisson was installed by suction and in the other by pushing, till a preloading of respectively 0.065kN and 0.062kN was reached. Installation of 20mm model caisson was done by pushing till a preload of 0.152kN. In each test a cycling load package with increasing amplitude was applied.

Since different caissons were utilized, results were converted into dimensionless form in order to allow comparison. Cyclic tests carried out with different caissons dimension and installed pushing, were showing that larger caisson had less accumulated displacement, so increasing the scale brings to a decreasement of total displacement.

As can be seen in *Figure 10*, tests where installation was done by suction, had a significantly higher total downward displacement. Stiffness was decreasing increasing load amplitude, and was remaining constant in sets of cycles with the same amplitude. Hysteresis was increasing with cyclic amplitude, and this increase was more marked when the load become tensile.

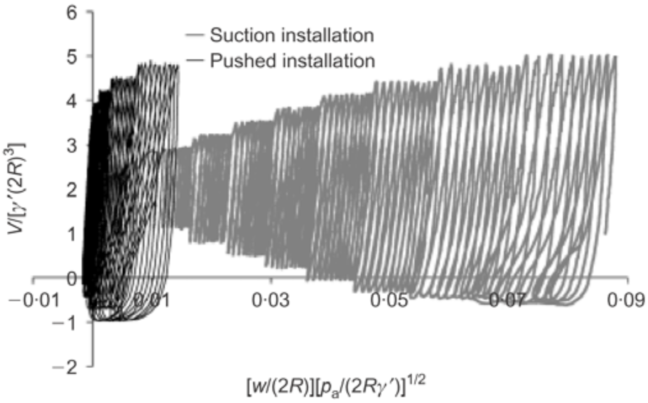

Figure 10. Cyclic loading after suction and pushed installation (Kelly et al. 2006a).

3.5 Centrifugal Experiment Study of Suction Bucket Foundations under Dynamic Loading (Lu X. et al. 2007).

Equipment features.

In this work a 50g-ton centrifuge was used to carry out tests. Sample of fine sand was prepared in a 600mm x 350mm x 350mm (L x W x H) tank. Sand was prepared layer by layer and pore pressure transducers were placed in between each layer, inside and outside the suction caisson, following a defined pattern. Sand was then saturated flushing water inside from the bottom, and applying vacuum. Consolidation was done applying a pressure of 80g, reaching a dry density of 15.69kN/m^3. Displacement measurements were done by means of two LVDT connected at the top of the caisson and another placed on the sand surface. Suction caissons had a diameter of 60mm and different skirt length of 48mm, 72mm, and 90mm. Vertical load was applied by hydraulic-electric system that can develop a maximum force of 0.98kN and a maximum frequency of 20Hz.

Testing

Monotonic tests were carried out with 60x72mm caisson, applying an upward displacement of 10 mm in steps of 0.2 mm. The uplift bearing capacity was mobilized at a displacement of 3.5 mm, corresponding to 2.1% of the diameter of the caisson, reaching a tension load of 0.59 kN. Uplift velocity is not specified in the article.

Cyclic tests were done by applying displacement amplitude of 2mm, 1mm, 0.5mm, and 0.2mm, at a frequency of 0.8Hz. Greater amplitude allowed greater pore pressure, of which peak was reached after 2.5 hours of loading, then the pore pressure was remaining constant with a slight decrease over the time. As a general behavior, great pore pressure was developed below the lid, pore pressure was decreasing with the depth and with distance from the model caisson. Applying an amplitude of 2mm (67% of the static uplift capacity) was bringing to a total liquefaction of the soil, with a reduction of the liquefied layer thickness decreasing the load.

3.6 Experimental Study on the Bearing Capacity of Suction Caissons in Saturated Sand (Lu et al. 2009).

Equipment features.

Tests were carried out in a 500mm x 500mm x 500mm tank made of glass, filled with 400mm of water saturated Mongolia sand that was vibrated in order to reach a dry density of 15.69kN/m^3. Displacements were measured by a LVDT with a range of 0-30mm and loads were measured by a transducer with a range of 0-6kN. Because of the limit of the apparatus, the vertical load was applied by displacement at a rate of 0.0067mm/s.

Two typology of foundation were tested, a monopod with diameter of 40mm and skirt length of 72mm and a tetrapod, composed of four caissons of the same dimension of the monopod, positioned at a distance of 10mm to each other. Each model caisson had a valve on the top that could be closed or opened depending on which test was carried out.

Testing

Monotonic compressive tests were carried out with a target downward displacement of 20mm. The bearing capacity curve had a steep increase during the first 4mm, then the increase become more slight. Single caisson reached a bearing capacity of 240N when the valve at the top was sealed, and a bearing capacity of 210N when the top valve was open. The difference is low because under monotonic loading the behavior of the sand tended to be drained also when the valve was sealed. Response of the tetrapod was nearly 4 times the response of monopod, meaning that bearing capacity was increasing with the same proportion of the numbers of suction caisson installed.

Monotonic tensile tests were carried out with both monopod and tetrapod, applying uplift vertical displacement at different rates (0.016mm/s, 0.16mm/s, 0.32mm/s). Uplift bearing capacity increased with the rate of loading *(Figure 11)*. The bearing capacity of tetrapod was almost 6 times the bearing capacity of the monopod, so there was a high strengthening effect using tetrapod configuration.

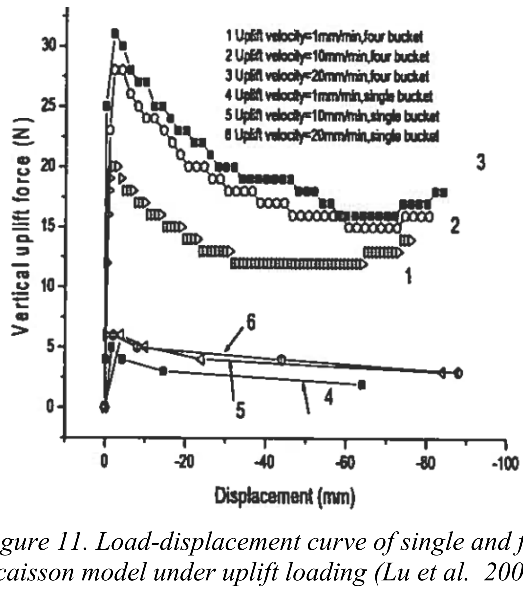

Figure 11. Load-displacement curve of single and four-caisson model under uplift loading (Lu et al. 2009).

3.7 Axial Capacity of Suction Piles in Sand (Jones W.C. et al. 1994).

Equipment features

Hydraulic ram was used for vertical loading. A cylindrical tank, with a diameter of 914mm and 1060mm high, was filled by Oklahoma sand, saturated with de-aired water in test where water saturated sand was used. Displacements were measured by a LVDT and load measurements were done by means of electronic load cell. A double-walled model caisson was used. The caisson was designed so that pore pressures can measured both inside and outside the caisson, by pressure transducers placed in three different positions on both sides of the skirt. Inner diameter of the caisson model was 111mm and outer diameter was 127mm.

Testing

Installation was carried out both by pushing and by suction. In installation by pushing 667N were required to complete the procedure for all the skirt length, and it was calculated that 91% of the installation load was carried by tip resistance. In installation by suction the first step was to let the model caisson to penetrate under its own weight. After self-weight penetration, different active suction were applied in different tests, in order to determine the minimum value of negative pressure required for installation, found to be 3.1kPa. Despite suction was maintained at the minimum value allowing installation, liquefaction of the soil inside the caisson could not be avoided, and an excess soil plug of 50mm was formed not allowing a complete penetration. Force required installing by suction was of 80N, therefore significantly lower than pushing installation. This was due in part to the not complete installation, but mainly to the flow around the skirt occurring as consequence of pressure gradient.

After suction installation, pull-out tests were carried out at a constant pull-out rate of 76mm/s. Tests In drained conditions were carried out in dry sand, keeping valve at the top of the model caisson open. The maximum tensile load was 66N, 50% of which was due to the caisson weight, and was mobilized at a displacement of 0.8mm (0.7% of the caisson diameter). Test in partially drained conditions were carried out in water saturated sand. Maximum tensile load of 244N was reached at a displacement of 25,4mm.

Tension load was causing a decrease on stiffness. Positive and negative pore pressures were increasing in magnitude as the load was going respectively in compression and tension. This increase was not due to the increasing load, but was depending on the velocity that the actuator had to apply in order to reach the target load within a period of 1 second.

3.8 Suction Caissons in Sand as Tripod Foundations for Offshore Wind Turbines (Senders 2008).

Equipment features.

Tests were carried out in a 40g-tones centrifuge, equipped with a sand box of 650mm x 390mm x 325mm (L x W x H). Electrical actuator was used to apply vertical displacement, maximum load capability was of 8kN and it could move in a range of 240mm. Further details are described in *Randolph (1991)*. Loads were measured by a 10kN load cell and pressure was measured by pore pressure transducers connected inside and outside the caisson. A syringe pump was used for suction installation.

Model caissons used in sand had skirt length/diameter measures of 60/60mm, and 60/49mm. Both of them were equipped with two valves: one to let the water going out and the other to apply suction by means of the syringe pump. Tests were carried out in oil saturated silica sand, the sample was then vibrated so a relative density in a range of 90-100% was reached.

Testing

Installation was done both by suction and by pushing at the rate of 1mm/s. Pull-out tests were carried out in both drained and undrained conditions, keeping the valve respectively open and closed, and applying slow or fast rate of loading. Cyclic tests were carried out to analyze partially drained to undrained conditions so valves were kept closed.

Monotonic pull out

Pull-out tests were carried out at 100g, keeping the valve open and applying slow pull-out rate for drained tests, and keeping the valve close and high pull-out rate for undrained tests.

In both drained and undrained tests, pore pressure response was increasing with up-lift velocity, and cavitation was reached with an uplift speed of 5mm/s. Seeing results in *Figure* 12, it was concluded that the uplift resistance increased increasing the pull-out rate, for infinitely slow (valve open), 1mm/s, and 5mm/s uplift speeds, the pull-out resistance was respectively 1.13γ 'D, 1.63γ 'D, 2.45γ 'D, values consistent with findings from *Bye et al. (1995)* and *Houlsby et al. (2005b)*.

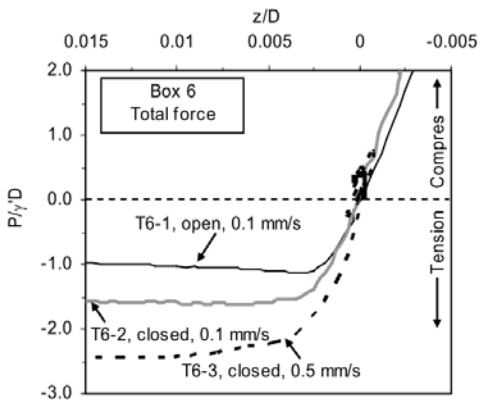

Figure 12. Total resistance (Senders 2008).

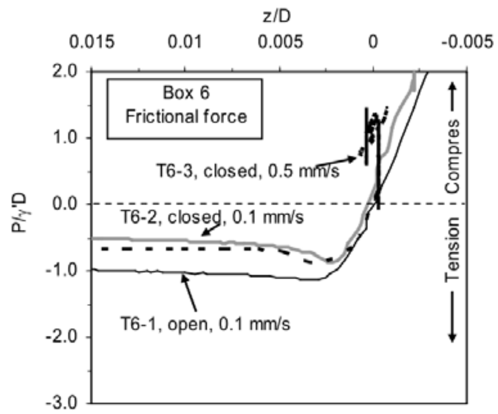

Figure. 13. frictional resistance (Senders 2008).

Figure 13 shows resistance given by friction in undrained tests, calculated subtracting uplift resistance given by pore pressure at the total response. Frictional resistance reached peak almost immediately *(Figure 13)*, with a linear trend, and was greater in drained condition. In undrained condition frictional resistance was decreasing after the peak, till a value that was half of the drained ones. The initial linear behavior of frictional resistance was similar for all tests, so it seemed to be not affected by up-lift velocity.

Figure 14 is showing the force developed by passive suction below the lid. There was a slight difference between tests carried out at different speed, therefore it was concluded that pore pressure was not directly related to the uplift speed. Comparing pore pressure with friction resistance, it was noticed that the latter was mobilized immediately, instead for the former the process was slower.

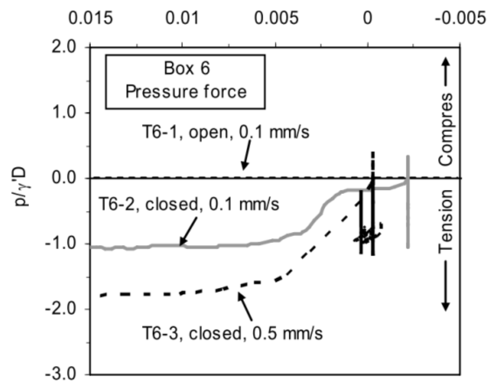

Figure 14. Pore pressure force below the lid (Senders 2008).

Cyclic loading

Cyclic loading tests were carried out keeping the valve on the lid closed and at an acceleration of 100g. Load cycles amplitude and frequency were varying between and within tests. Low frequency tests were carried out in a range of 0.07-0.045Hz, while in high frequency tests, loading frequency was in a range of 1-10Hz. Each cyclic test ended with tensile failure, and frequency, mean load, and load amplitude were varied individually if initial settings of cyclic loading were not critical enough to cause failure *(Figure 15)*.

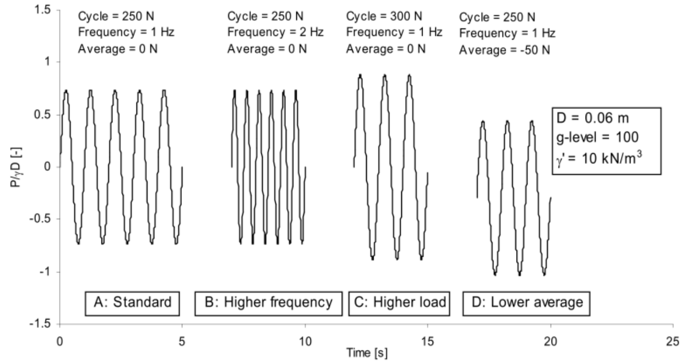

Figure. 15. Example of several cyclic loading patterns (Senders 2008).

Tests where large number of cycles was applied were showing that the number of cycles was not affecting the degradation of resistance, and it was not causing softening of the response. Also when cyclic load of 5N less than the static uplift resistance was applied, number of cycles did not affect degradation of resistance, and a steady state was reached between cyclic differential pressure and number of cycles. Conversely to the current design practice of suction caissons, it was concluded that resistance degradation due to large number of cyclic load does not need to be taken into account for the design.

3. DISCUSSION.

Kelly (2006b) used two different sands to evaluate the behavior under tensile loading.. In both cases, greater load is mobilized at smaller displacement in only pull-out tests with respect to cycling followed by pull-out tests, showing that loading history heavily affects the response. On cycling followed by pull-out tests, greater displacement is required to mobilize smaller load because of the loosening of sand below the lid. *Byrne (2000)* showed that loading history can also bring to an increase of the relative density, affecting positively the pull-out response.

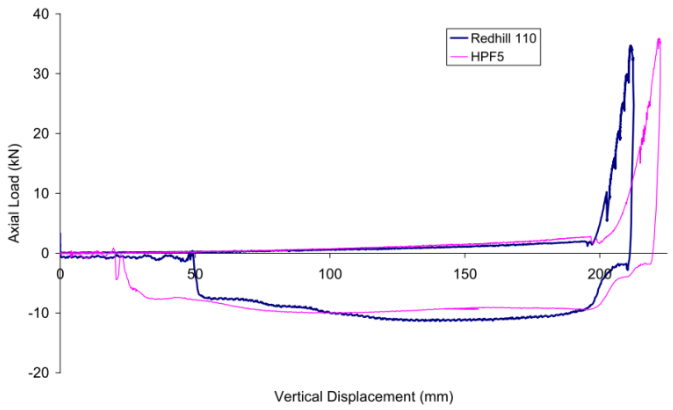

Figure 16. Pull-out response of large displacement test (Kelly et al. 2004).

From monotonic pull-out tests, carried out by *Byrne (2000)*, where rate and displacement magnitudes are varied, it is concluded that there are different phases of the response as can be seen in *Figure 5*. Same different phases of the response can be noticed also in *Figure 16*, where *Kelly (2006b)* analyzed the behavior under rapid pull-out in drained and partially drained soil conditions, respectively with sands Redhill 110 and HPF5. The initial softening behavior of the soil, occurs in this latter work at a greater tension load with respect to *Byrne (2000)*. Comparing tests carried out at atmospheric pressure, in *Kelly (2006b)* and *Byrne (2000)*, the differences are in pull-out speed, caisson diameter, and fluid of saturation. Larger caisson and greater pull-out speed utilized in *Kelly (2006b)* bring to a greater pore pressure development which could be the reason why the softening behavior occurs at greater tension load in this latter work. Since less frictional resistance is expected in the oil saturated sand, softening behavior is expected to occur at less tension load. Greater frictional resistance occurring in *Kelly (2006b)* can confirm the conclusion of *Byrne (2000),* who stated that the softening behavior occurs when the load exceed the skirt friction resistance. *Byrne (2000)* suggested that this softening behavior needs to be studied or in a geotechnical centrifuge , or with larger caisson in a sample hydraulically surcharged, in order to increase total stresses. Tests carried out in centrifuge and in a sand sample hydraulically surcharged using larger caisson were carried out respectively by *Senders (2008)* and by *Manzotti et al. (2014)*. Softening behavior has been noticed only in the latter work, and is more marked in test where overburden pressure is applied. Softening behavior is not present in *Senders (2008)*, despite the fact that drained tensile capacity is greater than *Byrne (2000)*. This suggests that further studies are needed in this topic.

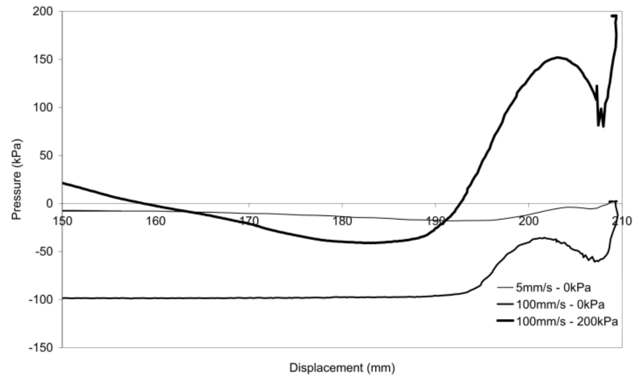

Figure 17. Pressures beneath the lid of the caisson during ultimate tensile loading in Redhill 110 sand (Kelly et al. 2006b).

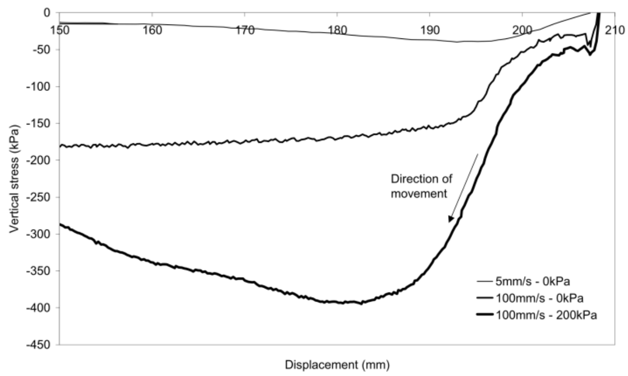

Figure 18. Ultimate tensile loading in Redhill 110 sand (Kelly et al. 2006b).

In *Kelly (2003)* and *Kelly (2006b)* tensional capacity under rapid loading in a pressure chamber was analyzed. At low pull-out rates, the response is drained and the capacity is given by the friction on the skirt. Increasing the rate of loading brings to a partially drained behavior, causing an increase of both stiffness and pore pressure *(Figure 17 and Figure 18)*. The response becomes greater and is limited by cavitation. Therefore, when the ambient pressure increases, the capacity is limited at higher loads, since it is increased the pressure at which cavitation occurs. The ambient pressure affects only the limit of the capacity, not the capacity inside the limit. Ultimate tensile load is dependent on the suction that can be generated under the lid. Hence, in order to have a high tensile load in sand with low permeability, a fast rate of loading and high ambient pressure are needed.

As pointed out by *Senders (2008)* and *Byrne (2000)*, the uplift resistance in drained condition is given by friction on the inside and outside of the skirt. Friction resistance is mobilized with small displacement of the caisson with respect to the passive suction resistance *(Byrne 2000, Kelly 2006b)*. As the behavior become more undrained, less frictional resistance is mobilized and more load is carried by passive suction. In these conditions the ultimate tensile capacity is mobilized with smaller displacement, but always at a greater displacement with respect to the frictional resistance. *Houlsby et al.(2005b)*, shows that high passive suction is mobilized at displacements in a range of 10 to 23% of the caisson diameter, therefore out of serviceability requirements.

Jones (1994) showed that in partially drained condition, suction developed below the lid cause a downward flow outside the skirt, which increases the effective stresses and so the frictional resistance on the outside skirt. In *Kelly (2006b)*, in tests carried out in Oakamoor HPF5 sand, greater load is mobilized with a pull-out rate of 10mm/s than 100mm/s. Since passive suction developed beneath the lid is smaller in the first test, this suggests that less skirt friction is mobilized in the test with a pull-out rate of 100mm/s. Comparing this latter test with test carried out in Redhill 110 sand, maintaining the same pull-out rate and the same caisson, maximum tensile load is mobilized at greater displacement in the less permeable sand Oakamoor HPF5 (*Figure 16*), where a partially drained to undrained behavior occurs. This behavior is in accordance with the conclusion that the enhancement in skirt friction due to the external downward gradient does not have time to occur in totally undrained condition.

It can be concluded that the ultimate tensile load increases proportionally to the uplift speed and the permeability of the soil, as long as the partially drained behavior allows the hydraulic gradient to occur. This is in accordance with Darcy law which linearly related the seepage with pressure differential. Tests carried out in pressure chamber (*Kelly 2003, Senders 2008)* shows that ultimate tensile load is mobilized at a displacements around 10-20% of the caisson diameter, therefore too large to satisfy serviceability requirements.

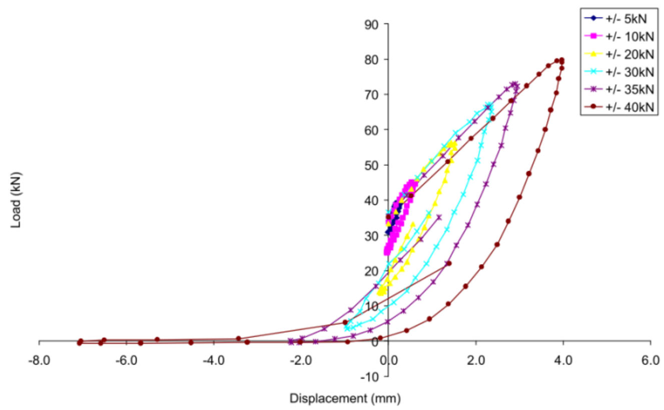

Figure 19. Cyclic loading carried out at different amplitudes (Kelly 2003).

In agreement with *Byrne (2000)*, during cyclic loading in *Kelly (2003)* the vertical stiffness of the caisson is significantly lower in tension than in compression, as can be seen from *Figure 19*, that gives a great representation of a typical load-displacement behavior under cyclic loading. This behavior is noticed also in *Kelly (2006b)*, where a typical trend of results, shows that for small cyclic load amplitude the response is stiff. As the amplitude increases and the load goes into tension, it turns in less stiff response. This brings to an increase of accumulated downward displacement and hysteresis. The physical meaning of increase in hysteresis is the increase in damping. This behavior has to be avoided in stage of design, so traction has to be avoided *(Houslby 2005b)*. *Kelly (2003)* found that the boundary of this dropping, rather than the transition into tension, is when the drained frictional capacity of the skirts is exceeded. Cyclic tests in *Byrne (2000)* and *Kelly (2006b)* are confirming these findings, showing tests where, despite the tension load is not reached, there is a significant drop in stiffness close to 0kN. In the less permeable sand, where less friction is mobilized, this drop occurs around 3kN as shown in *Figure 20.*

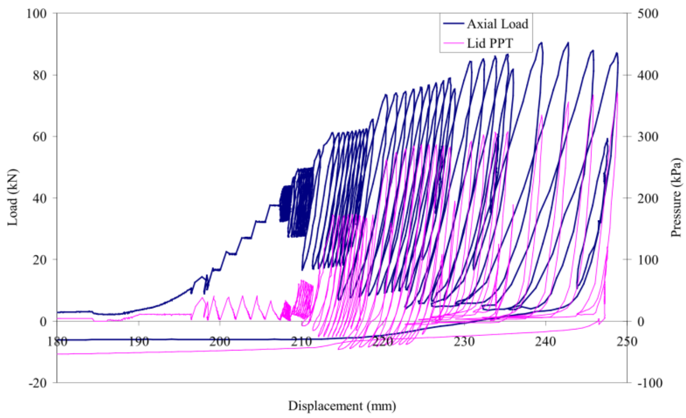

Figure 20. Load-displacement curve of test 14 (Kelly 2006b).

As general behavior in cyclic tests, ultimate tensile load is mobilized at displacement that compromise the serviceability of the structure, and in *Kelly (2006b)* it is stated that the low stiffness reached when tension load is applied can impose serviceability design limit.

This fast decrease in stiffness could be a reason to limit the design tensile load on an up-wind leg of a multi-pod foundation to the self-weight of the caisson plus the internal and external skirt friction, otherwise, due to the low stiffness, there could be ratcheting into the soil *(Kelly 2003)*. Avoid tension in a multi-pod foundation can be done adding ballasting or increasing the spacing between legs. Since these solutions are affecting the cost of the structure, to reduce conservatism *Senders (2008) and Kelly (2003)* suggest that tension could be allowed under extreme condition.

According with *Bye (2005)*, in *Senders (2008)*, a faster and higher development of pore pressure was noticed in high frequency cyclic tests with respect to low frequency tests. This is in contrast with founding in *Byrne (2000), Mangal (2000) and Johnson (1999),* who stated that the influence of loading rate is negligible. Rate dependency is evident also in *Kelly (2003)*, where decreasing the rate of loading in partially drained conditions brigs to a decrease in pore pressure development. Pore pressures are increasing with the rate of loading also in drained tests, but are not affecting the load-displacement behavior (Test 7 in *Kelly et al. 2006b*). This response could be explained by the less relevance that passive suction has in a drained response**.** *Kelly (2003)* found that pore pressure is linearly related to velocity when the load remains in compression. As a tension load is applied, the increase of differential pressure with the increase of loading frequency is not linear anymore and is found to be inversely dependent on the soil permeability.

In cyclic tests, total displacement is downward and increases when the load goes into traction. This behavior has been confirmed in various articles *(Kelly 2006b, Byrne and Houlsby 2002a and Byrne 2000)*, and is attributed by *Byrne (2000)* to the loosening that occurs during tensile load, that brings to greater displacement when load becomes compressive. It has to be noticed that in these studies the mean load is compressive also when traction is applied. In *Kelly (2006b)* tests carried out in a pressure chamber are showing that ambient pressure does not affect load-deformation response in cyclic tests, but affects only the limit of pull-out tension capacity.

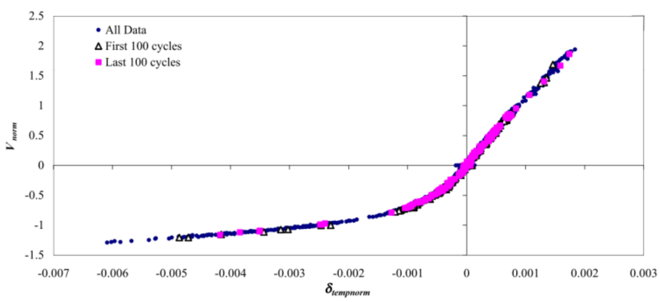

Figure 21. Long cycle response (Byrne 2000).

Lu (2007) established that cyclic response goes gradually in a steady state, where pore pressure fluctuates around a constant value and displacement does not develop any further. Similar behavior is noticed also in *Byrne (2000)* where, in order to evaluate how many cycles are necessary to carry out cyclic tests, a test with 2000 cycles was carried out. In this latter test, no significant difference in response is noticed between the first and the last 100 cycles as can be seen in *Figure 21,* showing that 100 cycles are enough to reach the steady state mentioned by *Lu (2007).* It is also noticed that when a cyclic load is applied, in between cycles with the same amplitude the stiffness remains constant *(Byrne 2000)*. A more close analysis on stiffness in long cyclic tests is done by *Kelly (2006a)*, where stiffness is noticed to increase, slightly and with a decreasing rate, with the number of cycles till a steady state is reached. These considerations are true as far as the load does not approach 0kN, at which point the stiffness drops. It is important to conclude that during cyclic loading there is no degradation of the response, but a little recovery on stiffness occurs instead.

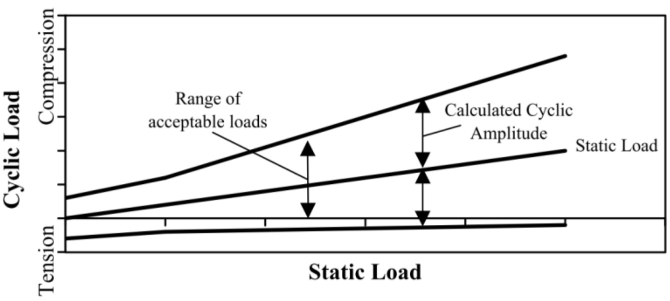

Figure 22 cyclic bearing capacity (Bye et al. 1995).

Figure 22 shows a graph extrapolated from field tests by *Bye et al (1995)*, where value on axis was not shown since results of the study are confidential. This graph suggests that there are boundaries limiting cyclic load amplitudes that can be sustained, once these boundaries are exceed a rapid degradation occurs, so extreme events have been inserted to study this behavior in *Byrne (2000)*. This clear threshold was not present in tests summarized in the present work*,* where in cyclic loading tests there is a gradual transition from stiff symmetric response to an asymmetric response as the load approaches tension, and, even after a tension load, the degradation is still gradual. The tensile boundary suggested by *Bye et al. (1999)* in *Figure 22* may be placed between the initial soft response and the rate-dependent response of the pull-out loading shown in *Figure 5 (Byrne 2000)*.

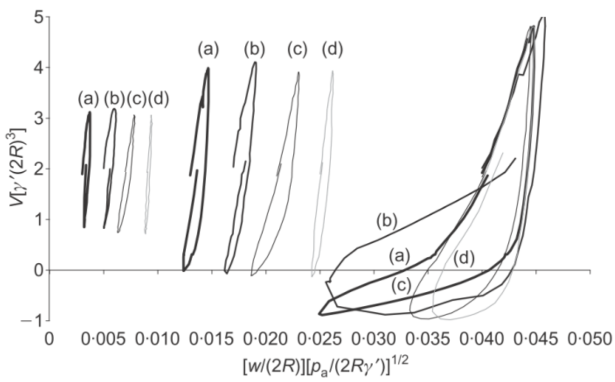

Figure 23. Comparison suction and push installation: installations by suction a) field test, b) 150 mm diameter caisson installed by suction, c) 200 mm diameter caisson installed by pushing, d) 150 mm diameter caisson installed by pushing. Kelly (2006a).

Responses of loading on caisson installed by suction and by pushing are showed in *Figure 23*, where results are normalized in order to allow a comparison. Tests where installation is done by suction have higher total downward displacement and a more steep decrease on stiffness. This behavior is due to loosening of sand that occurs along the skirt during installation, causing a reduction on the frictional capacity. Since these disturbances are localized, they have more relevance in small scale tests, causing greater displacement, in proportion with larger scale models. This latter consideration is true also for caisson of different dimension installed by pushing, as noticed in *Byrne (2000)* where normalized displacements are larger for smaller diameter caisson. *Jones (1994)* found that, applying suction installation, the penetration resistance is reduced to about one third with respect installation by pushing. In this latter work it is concluded that frictional capacity during pull-out loading of the caisson is reduced by suction installation, but has to be noticed that complete penetration into the soil cannot be done, because of formation of a soil plug inside the caisson. These considerations about how the modality of installation in laboratory affects the pull-out response, need to be further investigated, since phenomena that can act in favour of tension resistance, as consolidation of the soil occurring time after installation, has not been considered. In order to have, in pushing-installation test, a tensile behavior more similar to suction installation, it is suggested to study a loading history that cause a disturbance of the soil along the skirt similar to the one caused by suction installation, relying on passive suction to activate the flow mechanism near the skirt. Keeping in mind that to reach a steady state long cyclic test is not necessary, a cyclic test that ends with a steady state that induce a degradation comparable with the one of suction installation is possible.

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