Laboratory Test Setup for Cyclic Axially Loaded Piles in Sand

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ABSTRACT

This paper presents a comprehensive description and the considerations regarding the design of a new laboratory test setup for testing cyclic axially loaded piles in sand. The test setup aims at analysing the effect of axial one-way cyclic loading on pile capacity and accumulated displacements. Another aim was to test a large diameter pile segment with dimensions resembling full-scale piles to model the interface properties between pile and sand correctly. The pile segment was an open-ended steel pipe pile with a diameter of 0.5 m and a length of 1 m. The sand conditions resembled the dense sand conditions found several places in the North Sea. To simulate various vertical effective stress states, an elastic rubber membrane was placed on the soil surface and connected to a vacuum system, thus, increasing the effective stresses in the sand. A custom-made CPT devise was used to confirm equal soil conditions for all tests. For verifications purposes six static tension tests conducted at three different vertical effective stress levels of 0, 35 and 70 kPa. The load-displacement curves showed that the test setup provides repeatable test results. A preliminary comparison between the unit shaft friction determined from the API RP 2GEO standard and from the test results indicated overconsolidation of the sand. Two initial one-way cyclic loading tests provided results of effects on pile capacity and accumulated displacements in agreement with other researchers' test results.

KEYWORDS: Piles in sand, laboratory testing, axial tension loading, cyclic tension loading.

INTRODUCTION

Because of today's focus on the need for renewable energy, several offshore wind farms are under construction and more are planned for future installation. Much activity of this kind can be seen in the North Sea near the coasts of Denmark, Germany and Great Britain.For many of the existing and future sites, the soil conditions are dense to very dense sands. Some of these offshore wind turbines are installed on three or four legged jacket structures. The oil and gas industry have used jacket foundations for a long time and the design methods for piles loaded in compression are well examined. These design methods are now used when determining the bearing capacity of offshore wind turbine foundations. However, wind turbines are very light structures which can result in piles loaded in tension. The situation of piles cyclically loaded in tension is not well examined and pull-out of the piles may be a risk for the wind turbines. Pull-out of some of the piles in a foundation may result in irrecoverable tilt of the wind turbine. Therefore, it is of great interest to examine the behavior of piles cyclically loaded in tension.

The effect of cyclic loading on axially loaded piles in sand has been studied by means of both full-scale and small-scale testing. Jardine and Standing (2000,2012) as well as Beaβler et al. (2013) performed full-scale cyclic loading tests. Small-scale testing is less expensive than full-scale testing and, thus, has been conducted to a greater extent than full-scale testing. Amongst others, Chan and Hanna (1980), Le Kouby et al. (2004), and Thomas and Kempfert (2011) used calibration chamber tests to study the effect of one-way and two-way cyclic loading with different mean loads and load amplitudes on the pile capacity. The common characteristics of these test setups are the size and design of the test segments more similar to a CPT device than to a full-scale open-ended pipe pile. Moreover, the tests are conducted at 1G with an overburden pressure applied to the sand surface to increase the effective stresses by means of water or air filled pressurized membranes under fixed plates. At Aalborg University another type of test setup is used for small-scale testing at 1G. Instead of applying an overburden pressure, an elastic rubber membrane is placed on the soil surface and suction is applied. Foglia et al. (2012) and Vaitkunaite et al. (2014) used this method for experimental testing of horizontally and axially loaded suction buckets, respectively.

The intension was to construct and use a similar test setup to conduct medium-scale testing of an axially, cyclically loaded pile segment to investigate the pile-soil interaction at one-way cyclic tension loading. Medium scale implies a pile segment with a diameter of 0.5 m, close to the diameter of full-scale piles. As the test setup cannot accommodate a full pile with this diameter, the segment was only 1 m long. By simulating one meter of soil and pile at a time, it was an aim to enable analysis of a distinct *t-z* curve for each simulated depth.

This paper focusses on a detailed description of the test setup and test procedures. The following section presents the factors influencing the axial pile capacity when conducting small-scale testing. A discussion of these factors leads to argumentations for the chosen design. The result sections present the results of six static tests and two cyclic loading tests to verify the usability of the test setup. Advantages and limitations of the test setup are discussed. Finally, conclusions are drawn and recommendations for future work with the test setup are given.

OBJECTIVES OF TEST SETUP

Design of a proper test setup necessitates discussion of the factors influencing the targeted test results. For small-scale testing of axially cyclically loaded piles, the influencing factors include:

- Scaling effects.
- Pile design (dimensions, material).
- Test setup design (boundary effects, stress variation).
- Soil conditions (soil characteristics, relative density, saturation).
- Installation method.
- Load conditions (compression/tension, two-way and one-way cyclic loading, load/displacement controlled, amplitude, mean load, frequency, number of cycles).

The objective of the test setup described in the following was to enable investigation of the effect of cyclic loading on the pile shaft resistance of a pile segment which has a diameter comparable to full-scale, i.e. a diameter of at least 0.5 m, at conditions corresponding to service mode of an offshore wind turbine. Thus, at the time of testing, the pile segment should be unaffected by the installation procedure e.g. by preparation of the soil to match the targeted soil properties after installation. A pile segment with a length of 1 m was considered and as such, the principle in the test is not to model the full pile but only part of it. By increasing the effective stresses within the soil in the test setup, the idea wasto simulate 1 m pile at different soil depths. Moreover, it should be possible to run both load and displacement controlled tests. During the tests, the applied loads, the pile head displacement and the effective stresses at the soil surface should be recorded.

Pile Segment

The pile segment illustrated in Fig. 1 had the dimensions L_{pile} = 1 m and D_{pile} = 0.5 m. According to Randolph and Gourvenec (2011) the smallest offshore piles have a diameter of around 0.76 m and vary in diameter to wall thickness ratios between 25 and 100. Thus, the diameter of the pile segment was much closer to full scale than piles normally used in small-scale tests. The pile segment was made of steel, which resulted in a corroded surface of the pile, thus, giving realistic properties of the pile roughness and, thereby, the pile–soil interface friction. With a wall thickness, t_{pile} = 3 mm, the diameter to wall thickness ratio was 167.

The wall thickness was as thin as possible without risk of instability and buckling of the segment to reduce any pile base resistance that may occur during cyclic loading even though only one-way cyclic tension tests were planned.

The pile lid, which was used to connect the pile segment to the hydraulic piston, had four large holes, so the pile acted as a pipe pile during tests. Five pore pressure transducers placed on the pile wall, measured the pore pressure at positions of 1/3 and 2/3 L_{pile} inside and outside the pile wall as well as at the pile tip.

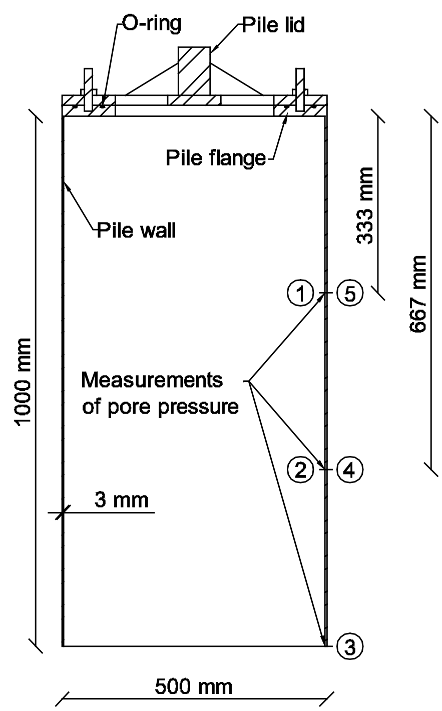

Figure 1: Cross-sectional view of the pile segment.

Test Setup

Fig. 2 shows the test setup whose main features is the sand box in which the pile segment was installed and a load frame where the loading equipment was attached.

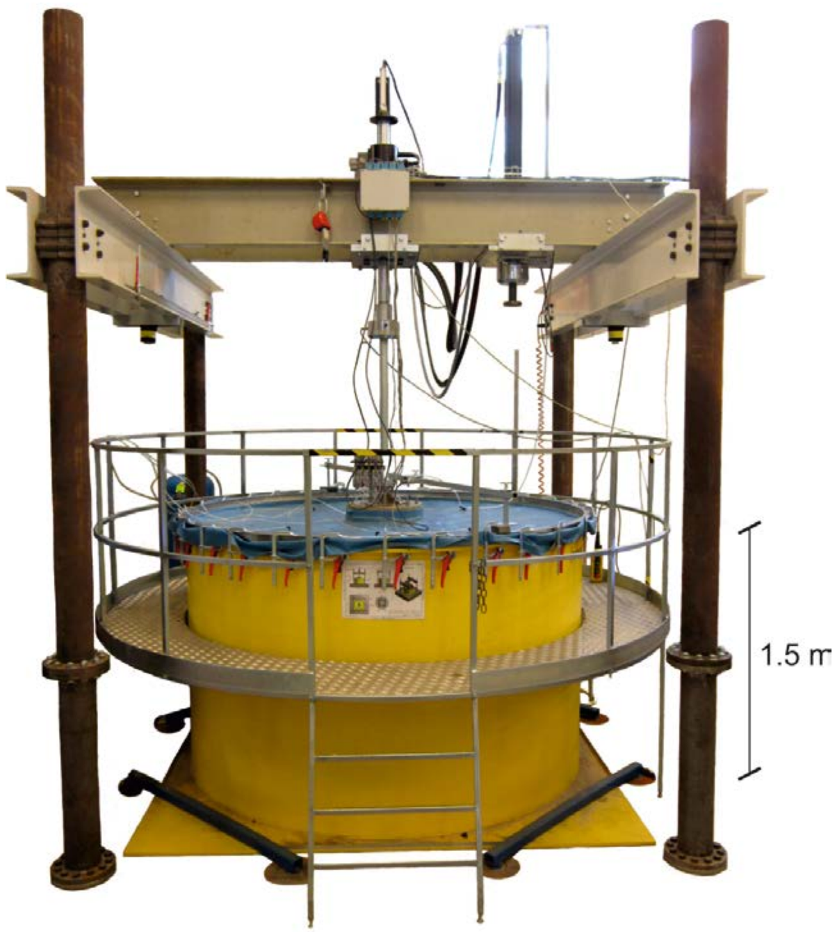

Figure 2: Sand box and load beam system of the test setup.

Fig. 3(*a*) shows the test setup layout while Figs. 3(*b*) and 3(*c*) show cross-sectional views of the test setup. The inner dimensions of the sand box were D_{box} = 2.5 m and H_{box} = 1.5 m. The sand layer had the thickness H_{sand} =1.2 m with a subjacent layer of gravel serving as a part of a drainage system. The system consisted of perforated pipes placed in 0.3 m gravel covered by a felt cloth ensuring homogenous water flow in the sand and preventing sand from getting into the drainage system. The system was coupled to a water outlet, a water tank and an ascension pipe. The water tank placed above the sand container allowed introduction of a hydraulic gradient which was monitored by means of the ascension pipe.

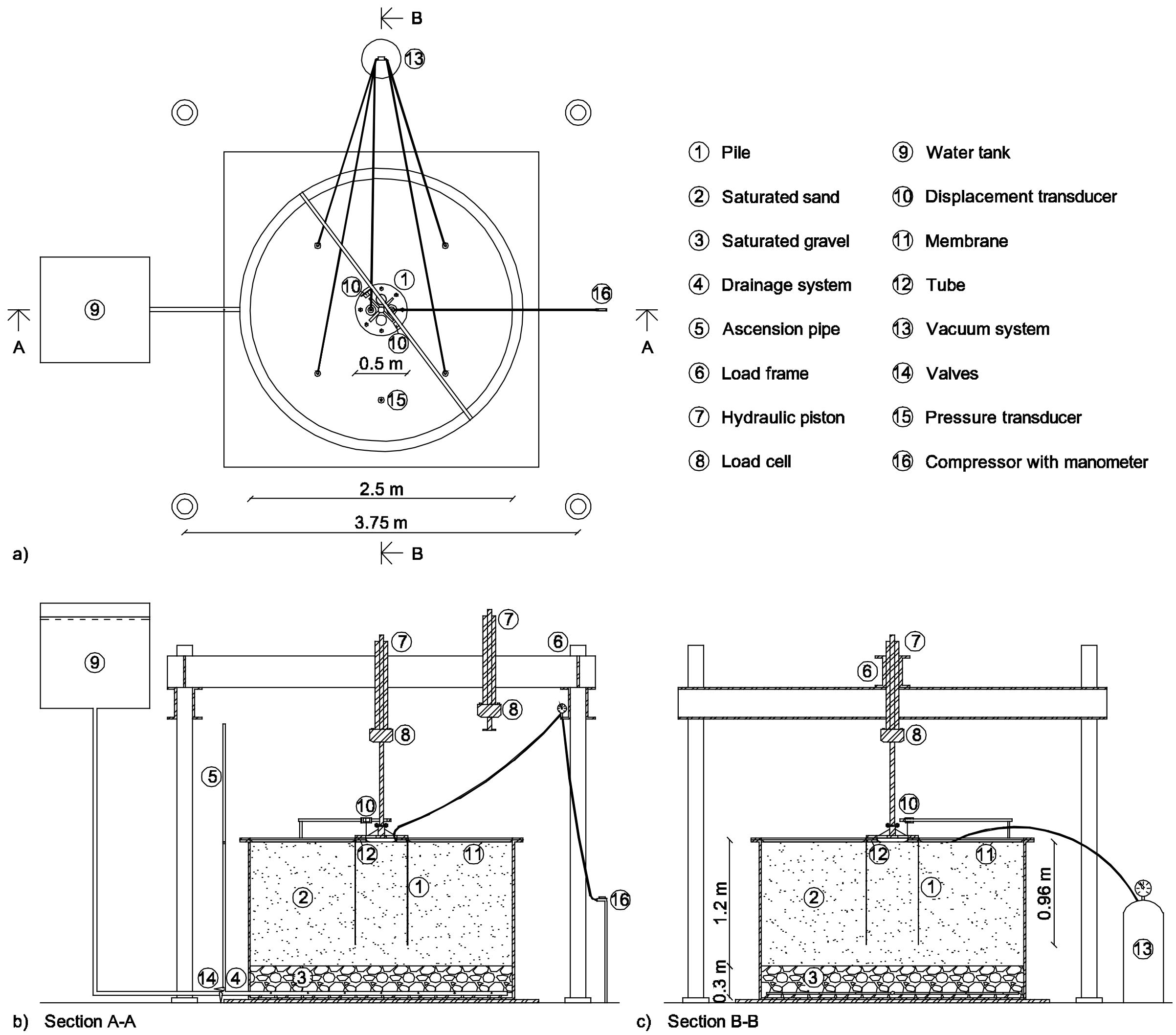

Figure 3: (*a*) Layout of the test setup; (*b*) Section A–A cross-sectional view of the test setup; (*c*) Section B–B cross-sectional view of the test setup.

Loading system

Two hydraulic pistons were attached to the load frame and could be moved in the longitudinal direction of the load beam. The 250 bars hydraulic piston to the right was used to install the pile and to conduct CPTs (Fig. 3(*b*)). The pump pressure was regulated according to the displacement rate of the piston which is measured by an ASM WS10ZG position transducer. The manually operated hydraulic piston was displacement controlled. A 250 kN load cell of the type HBM U10M measured the load. An ASM WS17KT displacement transducer with a range of 2500 mm measured the displacement o the piston. The equipment was connected to a HBM Spider 8 and measurements were recorded with the HBM program Catman Professional with a sample rate of 1 Hz.

The hydraulic piston in the middle was used when running tests (Fig. 3(*b*)). This hydraulic system can be both displacement and load controlled and is operated through the computer program MOOG Integrated Test Suite. A 250 kN load cell of the type HBM U10M measured the load during displacement controlled tests and controlled the pressure in the hydraulic cylinder during load controlled tests. By means of this setup it was possible to make a variety of tests, such as: Displacement or load controlled tests; static tests; and cyclic tests with different mean loads, cyclic amplitudes, wave shapes, and frequency. Two ASM WS10 displacement transducers with a 0–125 mm range placed opposite each other on the pile lid 50 mm from the pile edge measured the pile head displacement. The equipment was connected to an HBM MGC Plus and recorded by Catman Professional with a sample rate of 2 Hz.

Boundary Conditions

The effects of the boundary conditions on the results of laboratory tests depend on the ratio between the chamber radius and the pile radius, $R_{chamber}/R_{pile}$. Rimoy (2013) reported results of two-way cyclic loading test in a calibration chamber with $R_{chamber}/R_{pile}$ = 33. During the tests, radial, vertical and hoop stresses were measured within $2 < R_{chamber}/R_{pile} < 8$ range of the pile. The sensors all showed stress reductions under sustained loading.

Neither soil stresses nor the stresses at the boundary were measured in the presented test setup. Because $R_{chamber}/R_{pile}$ = 6 it must be assumed that stress reductions may take place during the cyclic loading tests. However, the effect of the boundary could not be analyzed with the present test setup.

Increase of Effective Stresses

At low stress levels the soil parameters vary strongly with the stresses. This is a problem when conducting tests at 1G. To avoid this in the present test setup, the effective vertical stress was increased by an elastic rubber membrane placed on the sand surface, sealed at the sand box edge and at the pile (Fig. 4), and connected to a vacuum pump. Common practise for increasing the effective vertical stress in calibration chamber tests is to use a rigis plate on the soil surface. By not using a rigid plate in the present test setup, the soil failure at the soil surface was not restricted.

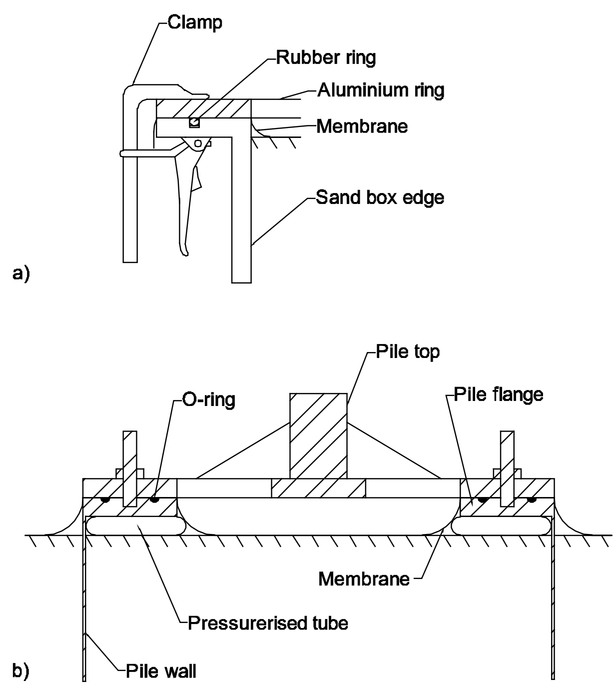

Figure 4: (*a*) Sealing of membrane at sand box edge; (*b*) sealing of membrane at pile flange.

The vacuum system was attached to the membrane by means of hoses and five quick couplings—four at the soil surface outside the pile and one on the soil surface inside the pile. As part of the vacuum system, a 200 L water tank collected the water outflow emerging as air trapped in the system expanded during suction and pressed the water out through the hoses. A felt cloth placed between the membrane and the sand surface restricted the sand from being sucked into the hoses and ensured a homogeneous distribution of the suction on the soil surface. A pressurized tube placed under the pile flange prevented the membrane from being sucked underneath the pile flange and get torn or unwantedly stretched. The excess pressure in the tube is of the same magnitude as the suction on the membrane so that the pressure on the soil surface under the pile flange was the same as on the remaining soil surface. A LISAB-NS-5B pressure sensor (0–5 bar) measured the suction on the membrane.

The membrane and vacuum system is not used for tests without increase of the vertical effective stress. However, the remaining setup was the same as for test with increase of the vertical effective stress.

Sand Specifications

Aalborg University Sand No. 1 is used when doing laboratory testing of offshore foundations at Aalborg University. The material properties of the sand are well-defined from classification tests and triaxial tests at Aalborg University (Hedegaard and Borup 1993; Ibsen and Bødker 1994). Fig. 5 presents the grain size distribution which shows a uniform grading making it possible to get a homogeneous compaction of the sand. Table 1 presents the material properties of the sand found from the classification tests. e_{max} and e_{min} were found according to Danish practice (Lund et al. 2001).

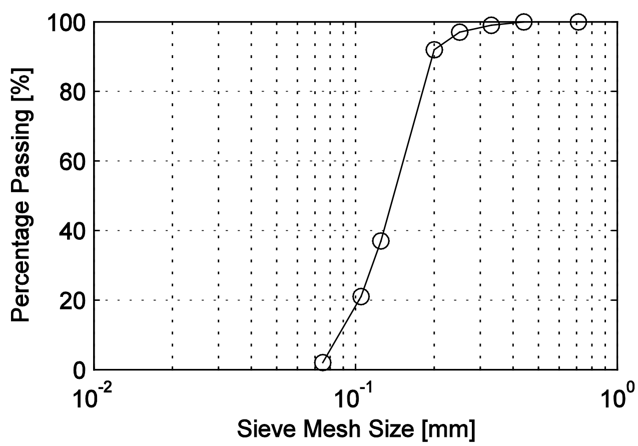

Figure 5: Sieve analyses for Aalborg University Sand No. 1 (Hedegaard and Borup 1993).

Table 1: Material properties for Aalborg University Sand No. 1 (Hedegaard and Borup 1993).

Specific grain density	d_s, g/cm^3	2.64
Maximum void ratio	e_{max}, -	0.854
Minimum void ratio	e_{min}, -	0.549
50%–quantile diameter	d_{50}, mm	0.14
Uniformity coefficient	$c_u = d_{60}/ d_{10}$, -	1.78

SOIL PREPARATION

At many wind farm locations in the North Sea, very dense sand is found in the upper soil layers, and it is at these sites the shake-up problem is observed. Existing tests were mostly conducted in medium dense to dense sands e.g. (Chan and Hanna 1980; Le Kouby et al. 2004, Jardine et al. 2009). Thus, tests in very dense sand are desirable; therefore, the sand in the presented study should reach a relative density, D_r, of around 85%. A relative density of around 85% was achieved by using the following soil processing procedure which is similar to that used by Foglia (2013) and Vaitkunaite (2014). Initial soil preparation before any tests:

1) Sand was placed in the sand box.
2) The sand was loosened by a hydraulic gradient.
3) The sand was vibrated with a rod vibrator.
4) Cone penetration tests (CPTs) were conducted and the was calculated from the CPT cone resistance
5) Step 2-4 were repeated until the average relative density converged at D_r = 85%.

Soil preparation between each test (the sand is not replaced between tests):

1) The sand was loosened by a hydraulic gradient.
2) The pile segment was installed while the hydraulic gradient is still applied.
3) The sand outside and inside the pile was vibrated with a rod vibrator.
4) CPTs were conducted to verify and average relative density of 85%.

Hydraulic Gradient

The applied hydraulic gradient loosened the sand and enabled vibration with a rod vibrator and installation of the pile segment. The hydraulic gradient had the magnitude $i = 0.9i_{crit}$ to avoid piping channels in the sand. The gradient was monitored by observing the water level in the ascension pipe.

Soil Vibration

The water level was 5 cm above the sand surface to avoid air flow into the sand during vibration. The vibration was done with a Wacker Neuson IRFU45 rod vibrator. A wooden plate with equally spaced holes ensured a uniform compaction of the sand by leading the rod vibrator through these holes (Fig. 6). The wooden plate was placed in brackets 10 cm above the sand surface and had no influence on the achieved soil conditions.

In the initial soil preparation phase, every hole of the wooden plate was vibrated once before conducting CPTs and reapplying the hydraulic gradient. The rod vibrator was lead 1.1 m into the sand and pulled up again in approximately 1 min at a constant speed. A vibration study established that the desired relative density could be reached by vibrating every second hole instead of every hole. This vibration procedure was therefore adopted between tests. Before a test every second vibration hole (marked with black) was vibrated and before the next test the other holes (white) were vibrated and so on for the following tests. The sand was vibrated at two positions inside the pile.

The loosening and vibration procedures relocated the sand grains preventing the same sand grains to be polished or crushed in all tests and, thereby, avoiding changes of the soil characteristics. Sieve analyses of sand stuck to the pile wall after eight static tests support this assumption as it shows no changes in grain distribution compared to sieve analyses conducted on a sample of sand prior to any tests (cf. Fig. 7).

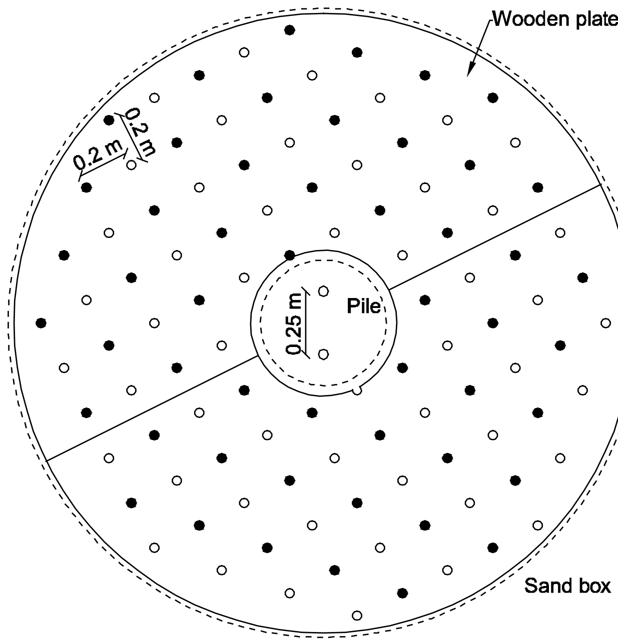

Figure 6: Sketch of the wooden plate (top view), the black and white circles indicate the holes vibrated before every second test.

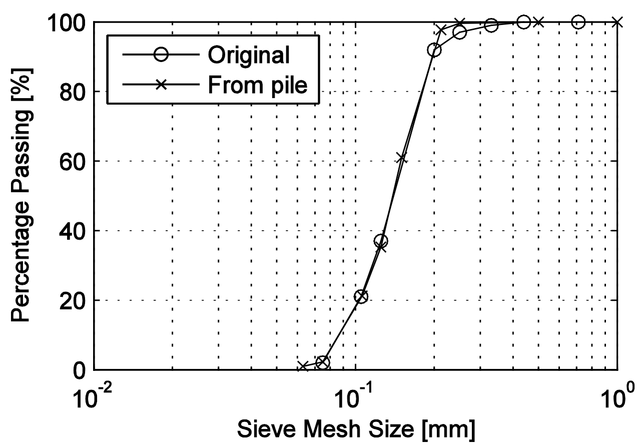

Figure 7: Sieve analyses conducted on the unused sand prior to the tests and sand scraped off the pile wall after eight static tests.

Installation

During pile installation, the surrounding soil experiences failure. Over time however, set-up will result in soil capacity gain. When conducting cyclic tests it is necessary to account for set-up or any other change in load history on the results. One way is to install the pile in a sand sample by jacking or pushing and then find the static capacity of the pile right before and right after a cyclic test, whereby, it is possible to compare these values of the capacity to one another (e.g. Tsuha et al. 2012). Another way is to deposit the sand around the pile, this way any effects of installation are avoided (e.g. Le Kouby et al. 2004). An objective of the herein presented test setup was to model a pile in service mode; hence, at a time after installation where maximum set-up was achieved. Thus, the soil was prepared to the desired relative density after pile installation. The pile was installed to an embedded depth of 0.96 m—leaving room for the pressurized tube under the pile flange—with and installation rate of 5 mm/s while applying the hydraulic gradient. Even though the soil was very loose during installation, plugging of the segment is observed during installed. Thus, indicating that plugging is likely to occur during tests as well.

Analyses of Soil Conditions

CPTs were conducted after vibration with a penetration rate of 5 mm/s at the six positions in the sand box illustrated in Fig. 8. The CPTs validated that the sand is approximately homogeneous throughout the box and that the density was identical from test to test. The CPT device was custom-made with a cone diameter of 15 mm and a cone inclination of 60°. The CPTs were conducted in the saturated sand before the tests; thus, the soil conditions for the CPTs were the same whether or not the following test was made with applied suction. The CPT penetration depth was 1000 mm i.e. the distance from the CPT cone to the gravel layer was 200 mm with an additional distance of 300 mm to the rigid bottom boundary. The cone resistance was therefore not affected by the rigid boundary.

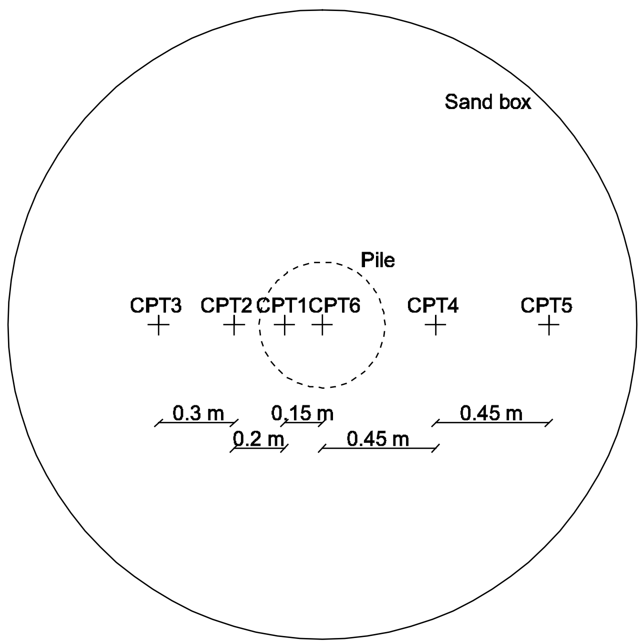

Figure 8: Position of the six CPTs (top view) conducted prior to each test.

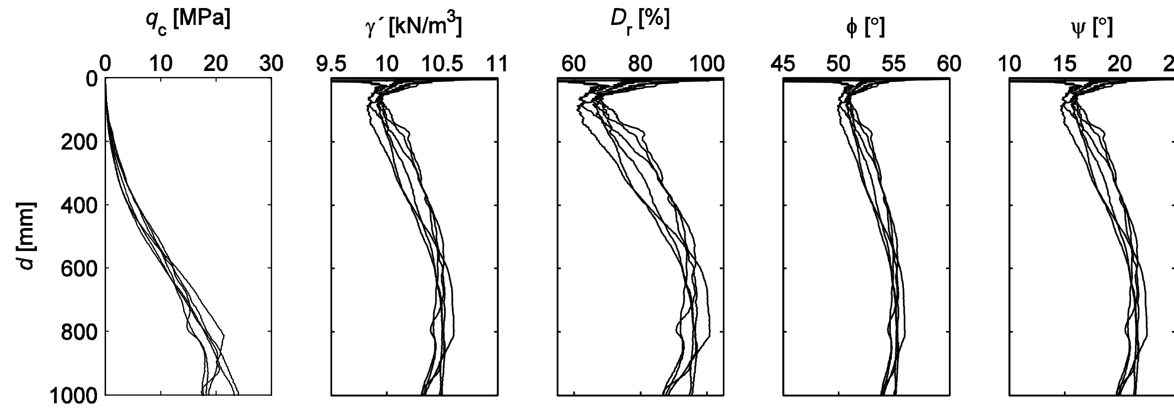

Figure 9: Example of CPT cone resistances and the determined values of γ', D_r, φ_{tr}, and ψ_{tr} versus penetration depth prior to a test.

An iterative procedure involving the following equations determines the friction angle, dilation angle, relative density, and effective unit weight from the CPT cone resistance. The expressions for the internal angle of friction and the dilation angle are based on results from triaxial tests on Aalborg University Sand No. 1 (Ibsen et al. 2009, Ibsen and Bødker 1994). Fig. 9 shows the parameters achieved from the CPT results. The relative density varies between 60–100% with depth but this variation was repeatable from test to test. The mean value of the relative density for the first twelve tests—nine static and three cyclic—was 85.9 % with a standard deviation between tests of 1.7%. The relative density showed no systematic increase or decrease.

$$\gamma' = \frac{d_s - 1}{1 + e_{\text{in-situ}}}\gamma_w \tag{1}$$

$$\sigma_1' = \gamma' d \tag{2}$$

$$D_r = c_2\left(\frac{\sigma_1' / \sigma_{1,\text{ref}}'}{\left(q_c / q_{c,\text{ref}}\right)^{c_1}}\right)^{c_3} \tag{3}$$

$$D_r = \frac{e_{\max} - e_{\text{in-situ}}}{e_{\max} - e_{\min}} \tag{4}$$

where γ' is the effective unit weight of soil, $e_{\text{in-situ}}$ is the in-situ void ratio, γ_w is the unit weight of water, σ_1' is the effective vertical stress, d is the depth below soil surface, D_r is the relative density, $\sigma_{1,\text{ref}}'$ is the reference effective vertical stress, 1 MPa, (c_1, c_2, c_3)= (0.75, 0.0514, -0.42), q_c is the CPT cone resistance, $q_{c,\text{ref}}$ is the reference CPT cone resistance, here chosen as 1 MPa.

$$\varphi_{tr} = a_1 D_r + a_2(\sigma_3' / \sigma_{3,\text{ref}}')^{-0.2807} + a_3 \tag{5}$$

$$\psi_{tr} = b_1 D_r + b_2(\sigma_3' / \sigma_{3,\text{ref}}')^{-0.09764} - b_3 \tag{6}$$

where φ_{tr} is the friction angle, (a_1, a_2, a_3) = (15.2°, 27.39°, 23.21°), D_{r} is the relative density, -, $\sigma_3^{'}$ is the confining pressure, $\sigma_{3,\text{ref}}^{'}$ is the reference effective horizontal stress, 1 kPa, ψ_{tr} is the dilation angle, (b_1, b_2, b_3) = (19.5°, 14.86°, 9.946°).

TEST RESULTS

Results of six static loading tests conducted at three different surcharge levels—0, 35, and 70 kPa—are given in the following. Moreover, two examples of cyclic loading tests without membrane are given.

The tests with applied suction were saturated throughout the test. For the tests with applied suction, the vacuum system sucked out most of the water leaving the sand almost unsaturated during the tests. Most calibration chamber tests were conducted in dry sand under the assumption of drained conditions and the effect of saturation is, therefore, negligible (e.g. Tsuha et al. 2012). Thomas and Kempfert (2011) reports excess pore pressure build-up in saturated sand and observed a negative effect of the bearing capacity for piles in saturated sand compared to dry sand. The effect was higher for two-way than one-way cyclic loading. However, the tests were run at a relatively high frequency of 1 Hz. The frequency of the herein presented tests was 0.1 Hz and it was assumed, that no excess pore pressure would build-up at this frequency.

The applied suction (surcharge) level has the same effect on the vertical effective stress as an applied overburden pressure. In the following the surcharge level is defined positive. The total measure tension force, Q_{T}, is defined negative and so is the upward displacement of the pile segment, w.

Results of Static Tests

The purpose of the tests was to examine the sleeve friction of an axially loaded pile, thus, only tension tests were conducted to avoid influence of the tip resistance on the bearing capacity. Another desired outcome of the static tests was the maximum load necessary to pull out the pile. The maximum force was then used to determine the mean value and amplitude of the subsequent cyclic loading tests. The static tests were displacement controlled and performed at a velocity of 0.002 mm/s to ensure drained conditions. A total displacement of 50 mm of the pile top was applied for the static tests. Figs. 10 and 11 shows the load–displacement curves for six static tests conducted at surcharges of 0, 35, and 70 kPa, respectively. The graphs show that the load needed to move the pile segment increased with increasing suction on the membrane, as expected. Furthermore, they show that the maximum loads were reached at displacements of 3–4 mm.

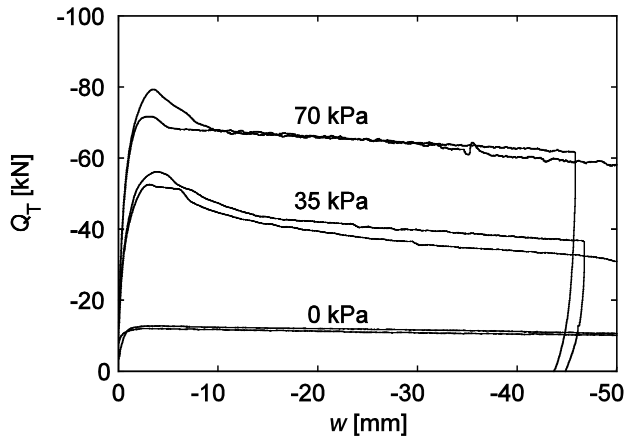

Figure 10: Load–displacement curves for six static tests with surcharges of 0, 35, and 70 kPa, respectively.

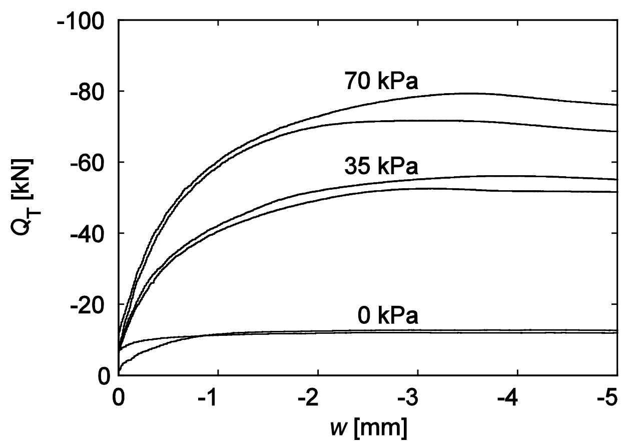

Figure 11: Initial part of the load–displacement curves displayed by Fig. 10.

Because of the short length of the pile segment the unit skin friction, f_s, is calculated directly from Q_T. Soil surface elevation of approximately 30 mm was observed inside the pile after the tests which means that the pile plugged during loading. Hence, f_s is calculated as

$$f_s = \frac{Q_T - W_{pile} - W_{plug}}{A_o} \tag{7}$$

where W_{pile} is the weight of the pile segment and equipment below the load cell, W_{plug} is the plug weight, and A_o is the outer pile shaft area.

Fig. 12 shows f_s versus the pile displacement while Fig. 13 displays the maximum unit skin friction plotted against the effective vertical stress in sand half way down the pile shaft. API RP 2GEO (2011) recommends the following determination of f_s:

$$f_s = \beta\sigma_v^{'} \tag{8}$$

where b is the shaft friction factor which is 0.56 for very dense sand (D_r > 85%), and σ'_v is the effective vertical stress at the depth in question. f_s from the test results are a bit higher than f_s determined from the API method. This is in good agreement with pile test database studies showing that the API method under-predicts the tension capacity of short piles in dense sand (Lehane et al. 2005).

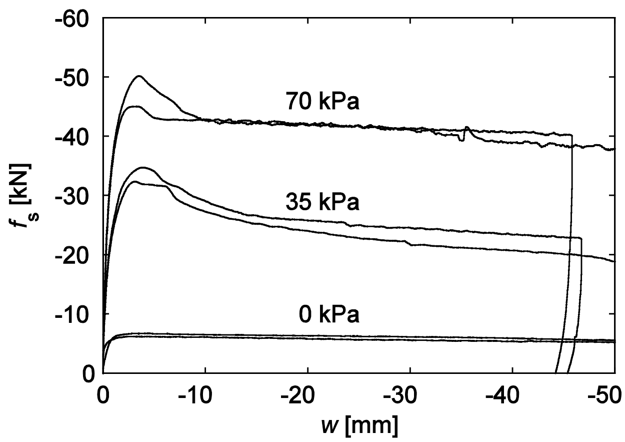

Figure 12: Unit skin friction for the six static tests with surcharges of 0, 35, and 70 kPa, respectively.

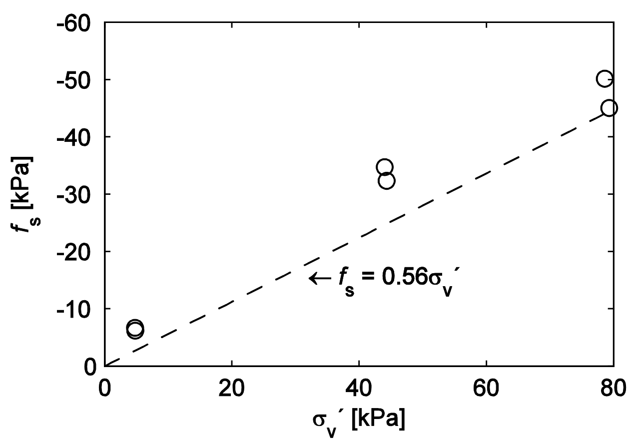

Figure 13: Maximum unit skin friction, $f_{s,max}$, (circles) versus effective vertical stress at the middle of the pile shaft compared to the API (2011) recommendations.

Results of Cyclic Tests

The cyclic tests were conducted after the described soil preparation procedure. Thus, no previous static or cyclic loading tests influenced the test results. The cyclic tests were constructed of three steps: Firstly, the mean load was reached with the same speed as was found from the load–time curve of the static tests. Secondly, cyclic loading was performed for two days with a frequency of 0.1 Hz. Thirdly, if the pile had not failed after the two days, a static test ran in continuation of the cyclic loading to a total displacement of the pile top of 50 mm. This was done to examine the effect of the cyclic loading on the static bearing capacity of the pile segment. that the test was terminated if the displacement of 50 mm was reached before the second day of cyclic loading.

Figs. 14 and 15 show the results of two cyclic loading tests conducted without membrane. The mean load, Q_{mean}, and the cyclic amplitude, Q_{cyclic}, illustrated by Fig. 16 are chosen based on the average maximum pull-out force, $Q_{max\ static}$ = 12.1 kN, obtained from the static tests. The first test illustrated, had a mean load of 0.4 $Q_{max\ static}$ and cyclic amplitude of 0.2 $Q_{max\ static}$.. The cyclic loading did not result in accumulated displacement and the maximum static pull-out force reached in the pre-cyclic static test was 13.7 kN. Thus, the cyclic loading resulted in a bearing capacity gain of 13 % compared to results of the static tests with no prior cyclic loading. Fig. 16 displays the result of a test with a mean load of 0.4 $Q_{max\ static}$ and cyclic amplitude of 0.2 $Q_{max\ static}$. The graph shows that the maximum displacement of 50 mm was reached before the two days of cyclic loading ended, after no more than 177 cycles.

These preliminary results resemble findings of other researchers well. For one-way cyclic loading tests, small amplitudes can increase the static capacity (e.g. Jardine and Standing 2012; Jardine et al. 2006; Le Kouby et al. 2004). Increase of the cyclic amplitude decreases the number of cycles to failure (e.g. Thomas and Kempfert 2011; Chan and Hanna 1980).

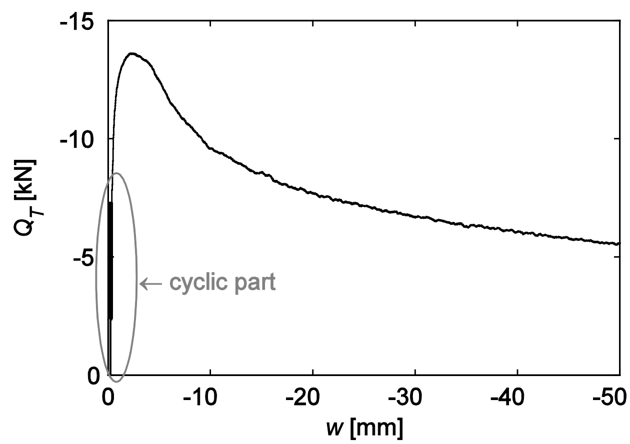

Figure 14: Load–displacement curve for cyclic test at 0 kPa with Q_{mean} = 0.4 $Q_{max\ static}$ and Q_{cyclic} = 0.2 $Q_{max\ static}$.

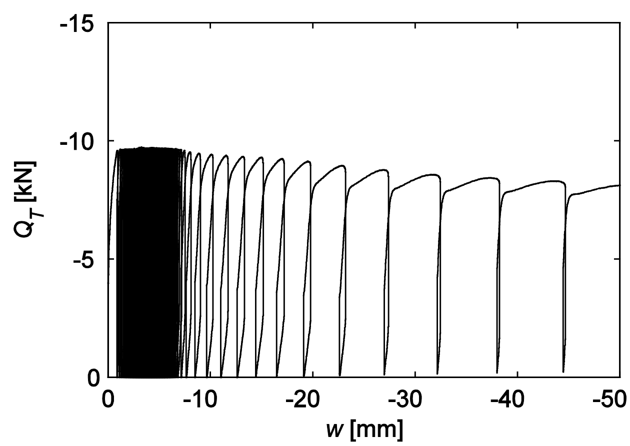

Figure 15: Load–displacement curve for cyclic test at 0 kPa with Q_{mean} = 0.4 $Q_{max\ static}$ and Q_{cyclic} = 0.4 $Q_{max\ static}$.

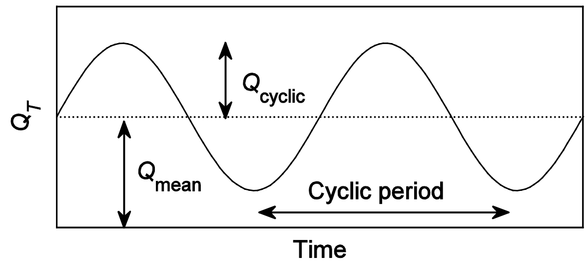

Figure 16: Illustration of the cyclic loading parameters.

Advantages and Disadvantages of the Test Setup

One of the advantages of the described test setup is that the design of the pile segment, such as the pile diameter and that it is open-ended, gives a much closer resemblance to full-scale piles compared to the pile segments usually used in laboratory testing. Hence, the scaling of the sand grains compared to the pile surface roughness is not an issue when interpreting the test results.

Because of the short length of the pile, the conditions are relatively homogeneous for the entire length and the test results can be interpreted as a single *t-z* curve along a pile at the depth corresponding to the increase of the effective stresses in the soil. A disadvantage of the short length is that the influence of the base resistance increases. However, only one-way cyclic loading test of piles in tension were conducted and the base resistance are in these cases negligible.

Because of the size of the sandbox and the way of saturating the sand it is not possible to completely avoid air presence in the sand. Due to the applied suction, the air expands and presses the water out through the suction tubes. Thus, the water level drops to about 0.9 m below the sand surface leaving the sand only partially saturated. The effect of saturated, partially saturated or dry sand on the cyclic axial capacity of piles in sand was not explored. It is assumed that the tests are carried out at a frequency preventing excess pore water pressure development and, therefore, it is assumed that the saturation degree of the sand has no impact on the test results.

CONCLUSION AND RECOMMENDATIONS

This paper presented a new laboratory test setup for testing axially loaded piles in sand. The purpose of the test setup was to gain knowledge about the behavior of piles used in jacket foundations for offshore wind turbines which, due to the low self-weight of the structure, are often loaded in tension. Therefore, the test setup was constructed to examine piles subjected to one-way cyclic axial tension loading. The diameter of the test segment was close to that of full-scale piles. The pile wall thickness was small to reduce the influence of base resistance to a minimum. The length of the pile segment was 1 m which enabled analyses of the skin friction at a given depth below the soil surface. To simulate different soil depths, the effective stresses in the sand were increased by placing a rubber membrane on the sand surface and applying a maximum of 70 kPa suction to the sand box.

The static tests showed that the test setup produces repeatable test results. The preliminary cyclic test results showed good agreement with results found in the literature. The static test results are currently being analyzed and compared to the CPT-based methods suggested by API RP 2GEO (2011), while more cyclic tests are being conducted and analyzed.

Recommendations for improvements of the test setup include strain or stress measurements at the rigid vertical boundary of the sand box to monitor the stress changes and estimate the boundary effects.

Another improvement would be to refine the vibration procedure to ensure a more uniform sand deposit with depth. In the presented sand preparation procedure, the rod vibrator moved with a constant speed both up and down. Perhaps a more uniform deposit could have been made by a slower penetration rate in the top of the sand layer and a faster rate at the bottom.

ACKNOWLEDGEMENTS

The research is funded by the Danish Advanced Technology Foundation via the program “Cost-effective deep water foundations for large offshore wind turbines”. The financial support is sincerely acknowledged.

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Editor's note.
This paper may be referred to, in other articles, as:

Ms. Kristina Thomassen, Prof. Lars B. Ibsen, and Dr. Lars V. Andersen "Laboratory Test Setup for Cyclic Axially Loaded Piles in Sand" *Electronic Journal of Geotechnical Engineering*, 2017 (22.03), pp 1089-1106. Available at ejge.com.