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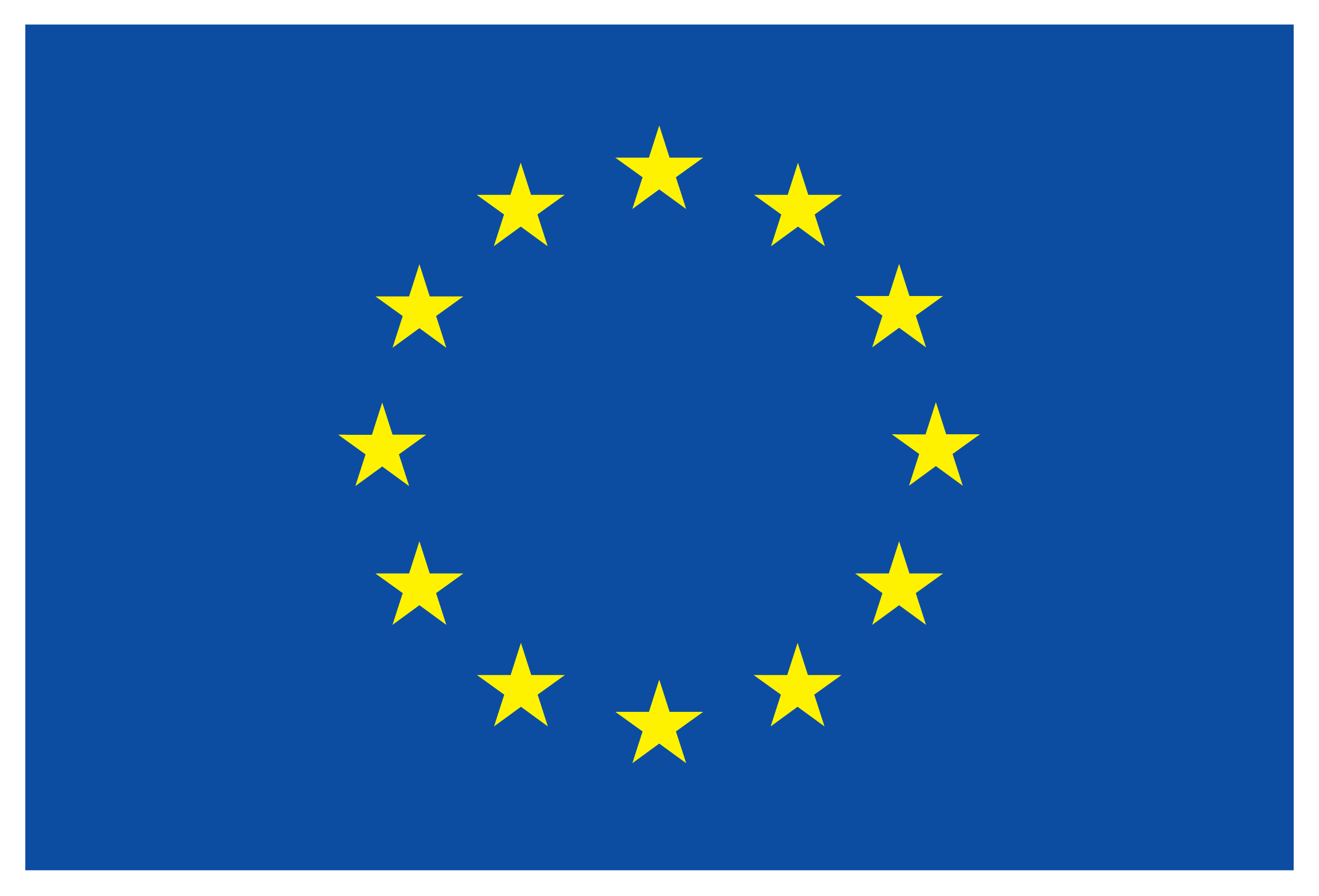
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| Author(s) information (alphabetical): | | |
| Name | Organisation | Email |
| Aligi Foglia | Fraunhofer IWES | aligi.foglia@iwes.fraunhofer.de |
| Tomas Gintautas | Aalborg University | tg@civil.aau.dk |
| Clemens Hübler | ForWind-Hannover | c.huebler@isd.uni-hannover.de |
| Kirill Schmoor | ForWind-Hannover | schmoor@igth.uni-hannover.de |

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| --- | --- | --- |
| Acknowledgements/Contributions: | | |
| Name | Name | Name |
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#### Definitions

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Table of Contents

[Executive Summary 1](#_Toc490745938)

[Introduction 1](#_Toc490745939)

[1. Test campaign 2](#_Toc490745940)

[1.1 Time schedule 2](#_Toc490745941)

[1.2 General notes to the tests 2](#_Toc490745942)

[1.3 Soil preparation and characterization 4](#_Toc490745943)

[1.4 Installation of the piles 7](#_Toc490745944)

[1.5 Geotechnical testing of the piles 7](#_Toc490745945)

[1.6 Dynamic tests 9](#_Toc490745946)

[2. Tests for soil characterization 12](#_Toc490745947)

[2.1 Soil samples 12](#_Toc490745948)

[2.1.1 Test procedure 12](#_Toc490745949)

[2.1.2 Test results 13](#_Toc490745950)

[2.2 CPTs 14](#_Toc490745951)

[2.2.1 Test procedure 14](#_Toc490745952)

[2.2.2 Test results 14](#_Toc490745953)

[2.3 Development of probabilistic models for relevant soil strength parameters 16](#_Toc490745954)

[2.3.1 General stochastic model 16](#_Toc490745955)

[2.3.2 Estimation of model uncertainty according to EN1990, annex D 18](#_Toc490745956)

[2.3.3 Results 20](#_Toc490745957)

[2.3.4 Probabilistic model 24](#_Toc490745958)

[2.4 Wave propagation tests 25](#_Toc490745959)

[2.4.1 Wave propagation basics 25](#_Toc490745960)

[2.4.2 Test procedure 26](#_Toc490745961)

[2.4.3 Test results 26](#_Toc490745962)

[3. Dynamic tests 33](#_Toc490745963)

[3.1 Basics in system identification 33](#_Toc490745964)

[3.2 Test procedure 33](#_Toc490745965)

[3.3 Test data 34](#_Toc490745966)

[4. Geotechnical tests 36](#_Toc490745967)

[5. Conclusive remarks 38](#_Toc490745968)

[6. References 39](#_Toc490745969)

[7. Appendix 40](#_Toc490745970)

Executive Summary

This report aims to give a comprehensive summary of the geotechnical tests with model structures that have been performed in work package 7.2 task 7.2.2 within the IRPWIND project. The large-scale tests are intended to determine soil-structure interaction effects in order to support probabilistic calculations of the reliability of offshore wind turbine support structures. These calculations are mainly performed in work package 7.4 of the IRPWIND project. However, this report already includes the development of a probabilistic model for the axial bearing capacity / resistance that is obtained using a CPT-based approach and the data obtained by large-scale tests. The main part of this report summarizes the test procedures, first test results, and in some cases the theory behind the tests or the data analysis.

Introduction

In task 7.2.2, experimental geotechnical tests with model structures on a large scale have been designed and performed to support probabilistic calculations and to enable the evaluation of the reliability of offshore wind turbine support structures and their foundations (performed under WP7.4). The experimental tests have been performed in a well-defined, sandy and water saturated soil resembling the North Sea environment. The experimental test program has been performed in a large geotechnical test pit (10 m depth, 14 m large and 9 m wide) in the new Test Center for Support Structures in Hannover. The tests are intended for the evaluation of the maximum static bearing capacity of mainly axial loaded piles. The gained data is used to get better understanding of the model error of API and CPT-based methods. For this purpose, a probabilistic model was developed that models the axial bearing capacity / resistance based on CPT data and test results of the axially loaded piles.

The report has the following structure. In the first chapter, the relevant phases and investigations of the experimental campaign are described. The following three chapters of the report are dedicated to the presentation and interpretation of the data gathered within the different investigation phases. The second chapter is about soil characterization and focuses on the CPTs. It also includes the probabilistic model that is based on CPT data and the results of the geotechnical tests that are described in section 4 in more detail. The third chapter deals with the dynamic testing of the piles. The fourth chapter is about the geotechnical testing of the piles for the determination of the maximum static bearing capacity. The relevant achievements of the entire experimental campaign are summarized in the conclusive fifth chapter.

# Test campaign

The main focus of the performed tests is the determination of the maximum static bearing capacity of mainly axially loaded piles. In addition to that, complementary investigations were conducted for a total of three experimental arrangements. Firstly, there were some tests to examine the soil parameters (soil tests). Secondly, there were tests to determine the maximum static bearing capacity of the piles (geotechnical tests). Thirdly, dynamic tests to investigate the modal properties of the piles and the damping of the soil-pile combination were conducted (dynamic tests).

## Time schedule

The tests have been conducted in the following order:

1. Planning of the tests (March 2014 – February 2015);
2. Ordering the piles and detailed planning of the time schedule (March 2015 – October 2015);
3. Preparation of the test pit and densification of the sand in layers (November and December 2015);
4. Taking soil samples at different depths during the preparation of the soil (November and December 2015);
5. Installation of a geophone at about 6 m depth during the preparation of the soil (27th November);
6. Conduction of a CPT campaign after the whole soil volume had been prepared (20th January);
7. Installation of the first pile with a double-acting pile hammer (26th January);
8. Installation of the five other piles (28th January – 15th February);
9. Measuring of the wave propagation in the soil (28th January – 15th February);
10. Dynamic tests with accelerometers and strain gauges during the installation of piles 4 to 6 (8th February – 15th February);
11. Dynamic tests with high sensitive accelerometers during installation breaks (28th January – 15th February);
12. Dynamic tests with high sensitive accelerometers on installed piles (28th January – 15th February);
13. Maximum static bearing capacity tests of all piles (2nd March – 21st March).

## General notes to the tests

The six piles have the following geometry: D is the diameter of the pile, L the embedded length, Ltot the total length of the pile, L/D the slender ratio, t the wall thickness and Di the inner diameter of the pile (see Table 1). The different dimensions were deliberately chosen in order to test piles with slender ratios (L/D) similar to those of jacket piles in the North Sea (L/D=15-25). By gathering more test results regarding the reliability of existing design methods for the geotechnical axial bearing capacity, a more suitable model error for each design method can be determined. Figure 1 depicts the slenderness ratio of already published pile load tests in the literature and of the pile load tests executed within this project. Altogether 12 pile load tests are available (double test on L/D=34 and L/D=18.8). As it can be seen, a model error on basis of all 12 pile load tests can be estimated which leads to a significant improvement within the reliability based design of the support structures. In addition, the range of slenderness ratios of L/D=15-25 is more emphasized. The estimation of the improved model errors and its consequences will be described in detail within WP7.4, since this is part of this WP.

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| *Figure 1: Slenderness ratio of published and chosen pile load tests* |

The preparation of the sand sample was carried out in a systematic manner which ensured a minimal horizontal variability of the soil parameters. The sand is supposed to be close to realistic conditions in the North Sea. The results of the soil tests showing the soil conditions are presented in Chapter 2.

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| Table 1: Pile dimensions |
| |  |  |  |  |  |  |  |  | | --- | --- | --- | --- | --- | --- | --- | --- | |  | D [mm] | L [m] | Ltot [m] | L/D | t [mm] | D/t | Di [mm] | | Pile 1 | 273.0 | 5.7 | 6.9 | 20.9 | 5.0 | 54.60 | 263.00 | | Pile 2 | 273.0 | 6.7 | 7.9 | 24.5 | 5.0 | 54.60 | 263.00 | | Pile 3 | 355.6 | 5.7 | 6.9 | 16.0 | 6.3 | 56.44 | 343.00 | | Pile 4 | 355.6 | 6.7 | 7.9 | 18.8 | 6.3 | 56.44 | 343.00 | | Pile 5 | 355.6 | 5.3 | 6.5 | 14.9 | 6.3 | 56.44 | 343.00 | | Pile 6 | 355.6 | 6.7 | 7.9 | 18.8 | 6.3 | 56.44 | 343.00 | |

The installation order and the exact positions of the installed piles in the test pit are shown in Figure 2. Since a second project was running in parallel, only half of the test pit was available for the tests. The distances between the piles are chosen to be at least six times the diameter of the piles so that there should be insignificant interaction effects between the piles both during installation and static loading phases. Figure 3 depicts the foundation specimens stored outside the laboratory before their installation in the sand pit. As it can be evidently noticed in Figure 3, all the piles were provided with two symmetrically pierced holes at pile head. This constructive feature allowed the piles to both be moved around the test hall and be statically loaded as in details elaborated in Section 1.5.

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| *Figure 2: Layout of the test campaign with positioning of CPTs, piles and geophones* |
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| *Figure 3: Piles stored outside the laboratory upon arrival* |

## Soil preparation and characterization

The soil used for the experiments was uniformly graded siliceous sand. The relevant physical characteristics of this medium are summarized in Table 2where *D*10 is the diameter for 10% finer by weight (and accordingly for 30%, 50% and 60%), *Cu* the coefficient of uniformity, *Cc* the coefficient of curvature, *Gs* the specific gravity of the soil and *n* the soil porosity. The sand sample was prepared from a height of 2 m (depth 8 m) until a height of 9.65 m (depth 0.35 m) in a fully controlled and coordinated manner in order to minimize possible non-uniformities. Throughout the report, depth and height inside the sand pit are relative to upper edge and bottom edge of the sand container. Embedded depth or embedded length refers to the distance between sand surface and a given point in the sediment (see Figure 4).

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| *Figure 4: Definition of depth, height and embedded depth/length in the geotechnical sand pit* |

Since the pile embedment lengths are limited to a maximum of 6.7 m it was deliberately chosen to prepare the new sand layers from a height of 2 (i.e. let in the sand pit the last two meter of soil from previous experiments). As a result of that, around 964 m3 (9 m x 14 m x 7.65 m) of sand were prepared in layers of approximately 30 cm thickness. For each layer, dry sand was initially distributed across the surface, watered to reach the optimal water content for compaction and eventually compacted with directional plate compactors. Four passages with the directional plate compactors were envisaged for each and every sand layer.

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| Table 2: Properties of the sand adopted for the experiments |
| |  |  |  | | --- | --- | --- | | **Property** | **Unit** | **Value** | | *D*10 | mm | 0.22 | | *D*30 | mm | 0.29 | | *D*50 | mm | 0.36 | | *D*60 | mm | 0.40 | | *Cu* | - | 1.82 | | *Cc* | - | 0.96 | | *Gs* | - | 2.65 | | *nmin* | - | 0.31 | | *nmax* | - | 0.46 | |

In order to extrapolate proper parameters for the dynamic, geotechnical and probabilistic studies, the soil was investigated by means three different methods:

1. Soil samples to determine soil compaction state and saturation degree;
2. CPTs to classify the soil and obtain basic soil properties;
3. Measurements of the wave propagation in the soil with geophones over and inside the soil specimen.

The soil samples were taken during the whole preparation of the soil. The first sample was taken at a depth of 7.65 m (height of 2 m) and the last one at the soil surface (depth 0.35 m, height 9.65 m). For each compacted soil layer five to eight samples have been taken at different positions. More information on the soil sampling is given in section 2.1.

The CPT campaign took place on January 20th, 2016. Seven CPTs were conducted. The positions of the CPTs are shown in Figure 2 and a photo of the test rig is displayed inFigure 5. Since the specimen preparation was systematic, it can be assumed that the soil conditions are quite similar in the whole test pit. Still, the conduction of several CPTs can proof the homogeneity of the prepared test pit. Additional CPTs after the installation of the piles or even after the extraction of the piles in order to determine changes caused by the installation process are not part of this project, but are worth investigating in following projects.

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| Figure 5: CPT rig adopted for the characterization of the soil sample |

The investigation of soil properties with the aid of CPTs is a state-of-the-art procedure. However, the process is relatively time consuming and special equipment is needed which can lead to fairly high costs in some cases. Therefore, alternative investigations methods are studied. One of these methods is the measurement of the wave propagation, as the wave velocity and other soil parameters like the dynamic shear modulus are correlated with the CPT measurements [1]. For the present test campaign, wave velocities were measured with four geophones. One was placed during the sand preparation at an embedded depth of 6.29 m, three more devices were fit on the sand surface upon sand preparation completion. The positions of the geophones are shown in Figure 2. During the ramming, the vibration velocity of the soil was measured with the geophones. By analyzing the time delay between the signals of the four geophones, given that the distances to the piles are known, the wave velocity can be calculated.

## Installation of the piles

The piles have been rammed into the soil with a double-acting pile hammer (Menck SB120, data sheet in the appendix). Special attention has been paid to the inclination of the piles, as it is very important for the following geotechnical tests. To ensure pile verticality and provide a robust support to the pile, a pile guide was used for the first half of the installation (Figure 6a) and the inclination was checked during installation (Figure 6b). The entire installation procedure was recorded with video camera to enable the counting of the hammer blows each 10 cm (see Figure 37*)*.

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| Figure 6a: Installation phase: Pile 1, pile guide and hammer | Figure 6b: Verticality check on the pile surface during installation |

## Geotechnical testing of the piles

The predominant objective of the geotechnical tests is the determination of the maximum static tensile bearing capacity of the piles. The entire load bearing curve, and in particular the initial stiffness, is also of interest. The data will be used in WP7.4 by ForWind-Hannover to evaluate the model error of API and CPT based methods for axially loaded piles in tension.

The tensile loading tests consisted of displacement controlled pull out tests until soil failure. The piles were pulled out from the soil for about a tenth of the pile diameter. The tests were carried out by designating a pulling velocity (i.e. load-controlled). To ensure substantially drained conditions, a very moderate velocity of 0.01 mm/s was adopted. The loading rig consisted of an actuator, a gantry and additional steel frame to connect the elements. All the elements involved in the static tests can be seen in Figure 7a and Figure 7b.

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| Figure 7a: Pile 1 during static uplifting | Figure 7b: Particular of Pile 1 during the tensile loading test. |

The actuator, used for all the tests, was a fully controllable hydraulic jack with a maximum capacity of 500 kN. Ad-hoc designed steel frames including long screws, plates of different size and a large pin were fit between actuator and pile head to provide an ideal hinged joint by means of which axial force could be transferred from the actuator to the pile. A massive steel gantry was used as firm support to counteract the forces coming from the application of the uplift action on the pile. All the piles were instrumented with a loading-rig-independent displacement transducer. Additionally, Pile 2 and Pile 5 were instrumented with half-bridged strain gauges to measure the differential deformation during installation and static test. The sensor layout and a picture of the sensor installation is shown in Figure 8a andFigure 8b respectively.

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| Figure 8a: Layout of the strain sensor | Figure 8b: Picture of the material placed on the pile surface to protect the strain sensor cables |

To make sure that set-up effects did not affect differently the piles the same time interval (35-36 days) was kept between pile driving and installation.

## Dynamic tests

The dynamic tests concentrated on the dynamic behavior of the piles at different embedded lengths and examine the damping of the soil-pile combination. For this purpose there were two types of excitations. Firstly, the excitation of the pile hammer during the ramming process was examined. The pile hammer excitations bring high energy inputs into the system. However, the disadvantage is that single excitations without interactions with previous or subsequent blows are not possible. Furthermore, the exact embedded pile length for each hammer blow is not known, as the pile is moving after every blow. For these reasons, in a second attempt, excitations induced with an impact hammer were used. These excitations have much lower energy inputs. On the other hand, the embedded length of the pile is known precisely and interactions among different excitations can be excluded.

Piles 1, 3 and 4 had sensors only above the soil, whereas piles 2, 5 and 6 had sensors below the surface as well. For piles 2 and 5, there were only strain gauges below the surface. Pile 6 had accelerometers in addition. As the geometry is the same for pile 4 and 6, a potential influence of the accelerometers and their protection (steel boxes and cable channels) on the geotechnical test for pile 6 is to be expected.

Above the ground all six piles have been equipped with four high sensitive triaxial accelerometers. The data sheets of these accelerometers can be found in the appendix. The accelerometers have been attached to the piles with pile clamps as they cannot be attached during the ramming. The measurements with these accelerometers have taken place during installation breaks and after the pile installation.

After 3 meters of ramming the process has been stopped, the accelerometers were attached to the piles and dynamic tests have been performed. Then the piles have been driven in one further meter before the next dynamic test was conducted. This procedure was repeated until the pile had reached its final depth.

The tests consisted of several excitations with an impact hammer in all three directions and at different heights. The positions of the accelerometers are shown in Figure 9*.* Since some positions were not reachable, when the pile has been driven further in, the sensors were then moved to a higher position.

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| Figure 9: Sensor positions (in red) above the ground |

After the installation had been completed, further dynamic tests with excitations by an impact hammer and sensor positions, as shown in Figure 9 on the right hand side, were conducted.

Pile 6 was equipped with strain gauges at three different heights. At each height two strain gauges were located 90° to each other. Furthermore, low sensitive uniaxial accelerometers (2000 g, data sheets in the appendix) at the same three heights have been mounted, too. Again, two accelerometers at each height were placed 90° to each other. As the accelerometers shall be reused they cannot be glued to the pile. Therefore, they were placed in small steel boxes that have been welded to the pile. The cables were passed through two cable channels that have been welded to the pile as well. This installation is shown in Figure 10a*.* The positions of the strain gauges and the accelerometers can be taken from Figure 10b. For piles 2, 5 and 6, measurements during the ramming and below the surface were possible with the strain gauges and the low sensitive accelerometers.

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| Figure 10a: Installed strain gauges and accelerometers on Pile 6 | Figure 10b: Positions of strain gauges and accelerometers at pile 6 (purple) |

# Tests for soil characterization

To ensure a high quality estimation of the soil parameters, which are the basis of all further analyses, three different kinds of tests are conducted. Firstly, soil samples are taken during the preparation of the tests. These tests are suitable for experiments like this, but are only partly applicable for field tests where the soil is not prepared layer wise. For field tests, the second kind of test is more common. CPTs can be performed easily with the right equipment. CPTs are the basis of many soil models. Thirdly, wave propagations tests are conducted. The correlation between the wave velocity in the sand and the CPT data is examined. This chapter is split up into three sections, one for each type of tests. In each section, the tests are described and test results are given. If data is further used, the results of the subsequent studies are given in the corresponding sections as well.

## Soil samples

As the soil is prepared layer wise in layers of about 30 cm, taking soil samples during the construction of the sediment is a quite straightforward approach to characterize the soil. Hence, the samples can be taken at different positions and depths without further effort. The water content, the dry density (ρdry) of the sand and the porosity of the soil (n) are measured for every sample. With this data, the void ratio (e) and the relative density (Dr) can be calculated.

### Test procedure

The soil is prepared layer wise. At the beginning of the preparation, the soil height is 2 m. This starting point results from previous experiments. It was not necessary to renew the whole soil. At this depth, six soil samples were taken. Afterwards a new layer of 30 cm was condensed. Eight samples were taken at this depth (2.3 m). This procedure has been continued until the final overall height of 9.65 m was reached. The samples were taken at different locations at every depth. For example, at 2 m they were taken from positions in C4, C1, D2, A3, B1 and B4 according to the plan in Figure 11*,* where the plot represents the same test pit as before so that the piles are located in C and D. 186 samples have been taken in total.

|  |
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|  |
| Figure 11: Soil sampling plan |

### Test results

Using the determined values of the porosity of the soil (n), the void ratio (e) can be calculated as follows:

(1)

The relative density (Dr) is

(2)

with and for the utilized sand. The results for the relative density are summarized in Figure 12. This figure shows the relative density over all samples. Sample 1 to 6 are at a soil height of 2.3m and sample 180 to 186 are at the final soil height of 9.65m. Furthermore, the mean value is marked with a red line and the one and the three sigma intervals are marked with blue and green lines respectively. It becomes apparent that the relative density is quite constant over all layers and all positions. The mean value is 0.736 and the standard deviation is 0.04. This means that according to the API [2] the sand is “dense” for nearly all samples, as “dense” sand has relative densities of 0.65 to 0.85.

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| *Figure 12: Relative density of all samples* |

## CPTs

CPTs are the basis of many soil models, as they can be conducted quite easily everywhere and their results are correlated to a number of soil parameters. Hence, a combination of the soil samples taken during the installation process together with CPTs can be very useful.

In this task of the IRPWIND project, a CPT campaign of seven CPTs took place. The reason for several tests is that spatial deviation can be analyzed if existing.

The results of the tests are firstly used for the characterization of the soil for further studies. Secondly, the CPT data is the basis of the model development of AAU in this task (Section 2.3).

### Test procedure

The CPT campaign took place on January 20, 2016. Seven standard CPTs have been conducted. The positions of the CPTs are shown in Figure 2 and a photo of the tests is displayed in Figure 5. It can be assumed that the soil conditions are quite similar in the whole test pit. Still, the conduction of several CPTs can proof the homogeneity of the prepared test pit.

Additional CPTs after the installation of the piles or even after the extraction of the piles in order to determine changes caused by the installation process are not part of this project, but are worth investigating in following projects.

### Test results

For all seven locations measurements for the cone resistance (qt), sleeve friction (fs), the friction ratio (Rf) and pore pressure are available every centimeter. The relative density can be calculated with the aid of this data with several different empirical approximations. One example of many is the approximation by Puech and Foray [3]:

(3)

The raw data of the CPTs is available for all IRPWIND partners and has been distributed among the partners in this task for further use.

Some summarizing results are presented here. In Figure 13, the cone resistance for all seven measuring points over the whole depth is shown. It can be seen that the soil in the test pit is horizontally quite homogeneous. Vertically the condensation is not that uniform. The soil is a bit less dense at about 7 meter depth. The value of P5, at about half a meter, is just an outliner because of the measurement or a contamination of the sand. Figure 14 shows the relative density of the soil calculated with the aid of the approximation of Puech and Foray [3]. The mean values of all positions are marked with vertical lines. It becomes apparent that the mean values are quite close to each other and are fairly close to the mean value of the relative density calculated from soil samples. However, it has to be mentioned that different approximations can lead to quite different results. Figure 15 shows exemplary all results for P6.

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| Figure 13: Cone resistance of all CPTs | Figure 14: Relative density calculated with CPT data |

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| Figure 15: All CPT results for measuring position P6 |

## Development of probabilistic models for relevant soil strength parameters

This section presents probabilistic models for relevant soil strength parameters.

### General stochastic model

Following the general guideline for modelling of load bearing capacities in ISO 2394:2015 the load bearing capacity / resistance, Y is assumed to be estimated by the following model:

|  |  |
| --- | --- |
|  | (4) |

where

 vector of random variables (soil strength parameters)

 set of deterministic parameters

 model for the resistance

*b*  bias associated with the resistance model

 model uncertainty associated with the resistance model – modeled as Lognormal distributed with unit mean and coefficient of variation equal to 

The soil strength parameters are assumed in general to be modeled as Lognormal stochastic variables obtained by discretization of the stochastic field  which is assumed homogeneous in horizontal and vertical directions and is characterized by

* Mean value as function of depth: , e.g. 
* Standard deviation (vertical):  which could be depth dependent
* Standard deviation (horizontal):  which could be depth dependent
* Correlation length (vertical): 
* Correlation length (horizontal): 

Each of these parameters is subject to epistemic uncertainty (statistical and model) uncertainty.

The parameters in the model for the stochastic field can for example be estimated on basis of data obtained from e.g. CPT tests (Section 2.2). The soil strength data is in general obtained from CPT data by the following model:

|  |  |
| --- | --- |
|  | (5) |

where

is a function transferring CPT data to the considered strength parameter

is model uncertainty connected to the function . It is assumed Normal distributed with mean value and standard deviation

 is measurement uncertainty connected to CPT measurements. It is assumed Normal distributed with mean value  and standard deviation. The statistical dependency between each CPT measurement data could vary between being fully correlated (if all data is influenced by the same measurement uncertainty) and independent.

Alternatively, the soil strength parameters (e.g. undrained shear strength and modulus of elasticity) can be modeled by

|  |  |
| --- | --- |
|  | (6) |

where

 stochastic field modeling soil strength parameter no *i*

 stochastic field modeling the physical uncertainty related to the soil strength parameter no *i*. The statistical dependency between  for different strength parameters should be assessed.

 model uncertainty connected to soil strength parameter no *i* modeling uncertainty in assessment of the strength parameter, e.g. based on CPT data.

 could also include statistical uncertainty. Alternatively, the statistical uncertainty can be modeled by treating the statistical parameters in  as stochastic variables with mean values, standard deviations and correlation coefficients estimated by the e.g. Maximum Likelihood method or by bootstrapping.

 could be Lognormal distributed with mean value  and standard deviation  . The statistical dependency between  for different strength parameters should be assessed.

Note: the uncertainty due to the physical uncertainty (represented by the stochastic field ) will in some cases be unimportant because the load bearing capacity is dominated by averaging the strength parameters over a larger volume / area. Since the model (and statistical) uncertainty is typically fully correlated over the volume / area considered, the resulting uncertainty contribution from the strength parameters will be dominated by this uncertainty.

Example: undrained shear strength 

|  |  |
| --- | --- |
|  | (7) |

where

 stochastic field modeling undrained shear strength

 stochastic field modeling the physical uncertainty related to the undrained shear strength.

Lognormal stochastic field:

* + - Mean value as function of depth: , e.g. 
    - Standard deviation (vertical): 
    - Standard deviation (horizontal): 
    - Correlation length (vertical): 
    - Correlation length (horizontal): 

 model uncertainty

Lognormal distributed with mean value  and standard deviation 

NB: the total ’point’ uncertainty in vertical and horizontal direction can be represented by  and .

### Estimation of model uncertainty according to EN1990, annex D

According to Eurocode EN1990, Annex D (2002) [4] the model uncertainty can be modelled as a Lognormal stochastic variable with bias and coefficient of variation which are determined following EN1990.

Step 1: Develop a design model

Develop a design model for the theoretical resistance of the component considered, represented by the resistance function:

|  |  |
| --- | --- |
|  | (8) |

where  represent the basic parameters (modelled as stochastic variables) which should be measured for each test specimen *i*.

Step 2: Compare experimental and theoretical values

The actual measured properties (values of the basic variables) are substituted into the resistance function to obtain the theoretical values which can then be compared with the experimental values from *n* tests, and shown as illustrated in Figure 16.

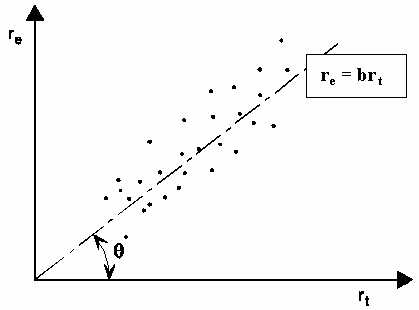


Figure 16: Illustration of experimental and theoretical values .

If the resistance function is exact and complete, then all the points will lie on the line with *b* = 1.

Step 3: Estimate the mean value correction factor *b*

The probabilistic model of the resistance *r* is written:

|  |  |
| --- | --- |
|  | (9) |

where *b* is the “Least Squares” best-fit to the slope, given by .

The mean value of the theoretical resistance function is estimated using the mean values of the basic variables:

|  |  |
| --- | --- |
|  | (10) |

where models the model uncertainty (modelled as a Lognormal stochastic variable).

Step 4: Estimate the coefficient of variation of the errors

The error terms and for each experimental value are determined from:

|  |  |
| --- | --- |
|  | (11) |
|  | (12) |

Estimated values for the mean and variance of are obtained from:

|  |  |
| --- | --- |
|  | (13) |
|  | (14) |

Finally, the coefficient of variation of the model uncertainty is estimated from:

|  |  |
| --- | --- |
|  | (15) |

In addition to the physical uncertainties modelled by  and the model uncertainty modelled by , also statistical and measurement uncertainty should be included when relevant.

Statistical uncertainty should be included when the number of test is limited (smaller than approximately 30). Assuming no prior knowledge, and applying Bayesian statistics implies that becomes Student-t distributed with mean and standard deviation determined from (13) and (14) and with number of degrees of freedom equal to *n*-1. If prior knowledge is available this can easily be included, see JCSS PMC (2002) [5].

Measurement uncertainty may be included as a stochastic variable multiplied to with coefficient of variation representing the measurement uncertainty.

### Results

In this subsection, the above approach for estimating the model uncertainty (and statistical uncertainty) is applied to the test data for the vertical load bearing capacity / resistance (section 4) where theoretical models for the resistance are based on measured CPT values (section 2.2). In general, the theoretical resistance models depend on

* Measured CPT = cone tip resistance values at depths *z*
* *L/D* = ratio where *L* = embed pile length and *D* = pile outer diameter
* *D/t* = ratio where *t* = wall thickness
* = unit weight of soil

This implies that for the CPT-based approach only measurement uncertainty related to the CPT measurements may be relevant to include in a probabilistic model. The measurement uncertainties are considered insignificant compared to other uncertainties, especially the model uncertainty, see below.

This implies that the theoretical model (according to the EN1990) can be written:

|  |  |
| --- | --- |
|  | (16) |

where

 = ()

 = (CPT, *L/D, D/t*)

The following data sets are available:

* 6 data sets (Pile 1-6) from the geotechnical tests described in this report
* 6 additional data sets described in [6]: EURIPIDES I, CLAROM CS, LAROM CL, GOPAL R1, HOOGZAND I, HOOGZAND III

The following theoretical models are used, see [2]:

* API
* ICP
* UWA
* FUGRO
* DNV

The following data are used with = 9.5 kN/m3:

Table 3: Data for axial bearing capacity / resistance.

|  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- |
|  | Measured | API | ICP | UWA | FUGRO | NGI | L/D | D/t |
|  | **[kN]** | **[kN]** | **[kN]** | **[kN]** | **[kN]** | **[kN]** | **[-]** | **[-]** |
| **Pile 1** | 157.2 | 65.5 | 131.9 | 122.9 | 151.1 | 299.7 | 20.9 | 54.60 |
| **Pile 2** | 123.6 | 86.0 | 150.0 | 138.9 | 143.3 | 353.2 | 24.5 | 54.60 |
| **Pile 3** | 187.1 | 87.2 | 186.4 | 174.6 | 230.7 | 391.0 | 16.0 | 56.44 |
| **Pile 4** | 252.2 | 114.2 | 213.8 | 199.7 | 223.9 | 460.6 | 18.8 | 56.44 |
| **Pile 5** | 152.8 | 76.7 | 164.0 | 155.8 | 199.1 | 348.5 | 14.9 | 56.44 |
| **Pile 6** | 219.3 | 114.2 | 213.8 | 199.7 | 223.9 | 460.6 | 18.8 | 56.44 |
| **EURIPIDES I** | 2900.0 | 2805.8 | 3833.5 | 3518.5 | 5060.5 | 4086.5 | 40.0 | 21.19 |
| **CLAROM CS** | 396.0 | 280.5 | 370.3 | 345.3 | 400.3 | 605.3 | 34.0 | 25.51 |
| **CLAROM CL** | 459.0 | 280.5 | 370.3 | 345.3 | 400.3 | 605.3 | 34.0 | 25.51 |
| **GOPAL R1** | 1450.0 | 1079.4 | 1006.7 | 848.7 | 845.7 | 1729.7 | 42.2 | 33.85 |
| **HOOGZAND I** | 830.0 | 135.1 | 648.1 | 671.1 | 732.1 | 808.1 | 19.7 | 22.25 |
| **HOOGZAND III** | 550.0 | 80.4 | 425.2 | 458.2 | 604.2 | 517.2 | 14.9 | 22.25 |

Table 4 shows the estimated bias and coefficient of variation of the model uncertainty for the 5 models and for the data from Pile 1-6 (*n*=6) and for all data (*n*=12). Also shown in the table in brackets is 1/*b* which ‘corresponds’ to the mean of the model estimate divided with the measured resistance. Figure 17 to Figure 25 show plots of the data in diagrams together with the least squares fit. D/t and L/D combinations of the tests can be found in Figure 1. It is seen that if data from Pile 1-6 is used:

* The API model has a relatively large COV but also a large bias
* The other models have COVs approximately equal to 0.15 and bias between 0.9 and 1.1.
* The NGI mean model error is greater than 2 and thus the NGI model is deemed not appropriate for this case. Therefore, bearing capacities using NGI model values for piles 1-6 were not used in further analysis.

If all data is used then:

* The API model has a quite large COV
* The other models have larger COVs compared to those obtained using Pile 1-6; approximately around 0.20.
* The bias values vary a lot with values from 0.6 to 1.1.

Table 4: Estimated bias (and mean value in brackets) and coefficient of variation (COV) of model uncertainties.

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
|  |  | API | ICP | UWA | FUGRO | NGI |
| **Pile 1-6** | Bias *b* (1/*b*) | 2.00 (0.50) | 1.02 (0.98) | 1.10 (0.91) | 0.93 (1.08) |  |
| COV | 0.18 | 0.14 | 0.15 | 0.15 |  |
| **All** | Bias *b* (1/*b*) | 1.10 (0.91) | 0.82 (1.22) | 0.90 (1.11) | 0.62 (1.61) | 0.75 (1.33) |
| COV | 0.63 | 0.20 | 0.20 | 0.27 | 0.20 |

Figure 17: Experimental and theoretical values using API model and Pile 1-6 data.

Figure 18: Experimental and theoretical values using ICP model and Pile 1-6 data.

Figure 19: Experimental and theoretical values using UWA model and Pile 1-6 data.

Figure 20: Experimental and theoretical values using FURGO model and Pile 1-6 data.

Figure 21: Experimental and theoretical values using API model and all data.

Figure 22: Experimental and theoretical values using ICP model and all data.

Figure 23: Experimental and theoretical values using UWA model and all data.

Figure 24: Experimental and theoretical values using FURGO model and all data.

Figure 25: Experimental and theoretical values using NGI model and all data.

### Probabilistic model

In this subsection, a probabilistic model is summarised assuming that the axial bearing capacity / resistance is obtained using a CPT-based approach where the resistance is estimated using equation (16), i.e. the input is

* Measured CPT = cone tip resistance values at depths *z*. Measurement uncertainty is assumed insignificant
* Mean value / best estimate of unit weight of soil
* *L/D* = ratio where *L* = embed pile length and *D* = pile outer diameter
* *D/t* = ratio where *t* = wall thickness

The probabilistic model for the resistance  is written:

|  |  |
| --- | --- |
|  | (17) |

where

model uncertainty: Lognormal distributed with mean value = 1 and COV, according to Table 4. In addition, statistical uncertainty is modelled assuming that is Student-t distributed with mean and standard deviation determined from (13) and (14) and with number of degrees of freedom equal to *n*-1 where *n* is the number of tests.

bias according to Table 4.

unit weight of soil. Following JCSS PMC (2002), [5], the dry unit weight can be modelled as Normal distributed with COV = 0.05-0.10 corresponding to the volume of the testing device, with characteristic dimensions of some centimeters for laboratory test devices up to a few decimeters for field test devices. As these characteristic dimensions are small relative to characteristic dimensions of affected surfaces or volumes, these standard deviations may be interpreted as standard deviations of unit weight fluctuations “from point to point”.

It is noted that by application of the CPT-based approach no specific uncertainty model is needed for CPT.

## Wave propagation tests

Taking soil samples and conducting CPTs in order to determine soil properties is state of the art. However, there are other methods to estimate soil properties. One of these methods is the consideration of wave velocities in the soil. As the wave velocity is correlated to other soil properties, it can eventually be used to replace CPTs.

This section deals with the conducted wave propagation tests. As these tests are not so common, a short introduction to ground waves, wave velocities in sand and in the CPT-wave velocity correlations is given. Subsequently, the tests that have been performed are described. A third section deals with the outcomes of the test, with problems that have occurred and finally with conclusions that can be drawn.

### Wave propagation basics

Waves travelling through the soil are so called seismic waves. They can be a result of events like earthquakes if they occur in large scale. In small scale, pile driving can be the reason for their appearance. Talking about seismic waves, one has to distinguish between so called body waves and surface waves. Body waves can be further divided into longitudinal P-waves and transverse S-waves. All body waves travel into the soil, whereas surface waves mainly travel along the surface. The most important type of surface waves in this context is the Rayleigh wave. Rayleigh waves are a combination of longitudinal and transverse wave parts.

In order to measure the speed of these waves, the vibration velocity of the soil can be measured with geophones at different distances to the source of the wave (in this case to the rammed pile). With the time delay between the signals and the distances of the geophones to the source the wave velocity can be calculated.

The velocity of the wave in the soil depends on the type of soil and on the type of the wave. The Rayleigh wave velocity in sand is about 10% lower than the S-wave velocity [7]. In Table 5*,* according to [8], some examples for the S-wave velocities for different soil types are given. However, it can be seen that even for the same type of soil and wave type, for example for S-waves in sand, the wave velocity differs a lot. This is a result of different soil conditions (i.e. denser sand results in increased wave velocity). Therefore, it is possible to correlate the wave velocity with soil properties, especially with CPT data ( [1] and [9]).

|  |
| --- |
| Table 5: S-wave velocities for different soil types |
| |  |  | | --- | --- | | Soil type | Range of vS, m/s | | Alluvium, silt, silty sand | 150 - 350 | | Clay | 300 - 600 | | Calcareous sandstone | 600 - 750 | | Marl | 650 - 800 | | Chalk | 750 - 950 | | Basalt | 1800 - 2300 | |

### Test procedure

The test was designed to be an addition to the other soil tests. With four geophones placed on and in the soil, the wave velocity in the soil was determined. The positions of the geophones can be seen in Figure 2.

The three geophones on the surface are triaxial geophones and measure the Rayleigh wave velocity, whereas the geophone inside the soil is only uniaxial (vertical direction) and measures the vibration caused by body waves. During the whole ramming process of four of the six piles the geophones were measuring the vibration velocity. However, as the excitation by the hammer is not very distinct, reflections at the test pit walls and the test pit ground occur and the excitation frequency of the hammer is fairly high with more than 2 Hz, additional tests were conducted. The additional tests consist of excitations of already installed piles with an impact hammer. The signals of these tests are slightly more distinct and especially the interaction effects of subsequent excitations are not present.

### Test results

The three geophones on the surface record time series of the vibration velocities in all spatial directions and the geophone in the soil only in vertical direction. The wave velocity can be calculated with aid of the distances to the excitation source as follows:

(18)

where vwave is the velocity of the wave, d’s are the distances of two different geophones to the pile and Δt is the time delay in the signal between the two geophones. So far it is not specified which type of wave is considered. However, as three geophones are placed on the surface, these geophones measure the vibration caused by a Rayleigh wave. The geophone in the soil, firstly, measures a combination of Rayleigh and body waves. Secondly, the soil is water saturated below 0.5m and dry at the surface. As the wave velocity in sand changes significantly for wet sand, the velocities are not comparable and geophone 4 (in the soil) is not further considered. A calculation of the wave velocity with only one geophone is not possible, as the total run time from the source to the geophone is not known because there is no information about the excitation available and the excitation cannot be regarded as a point source. Theoretically, it is possible to use the data of geophones 1 to 3 to gain information about the excitation itself. With this information, it would be possible to calculate the wave velocity in the soil only based on the measurements of geophone 4. However, this procedure leads to really uncertain results. Therefore, it was decided to abstain from endeavoring this approach here. Figure 26 clarifies the problem of massively higher wave velocities in the wet sand below the surface. Having in mind the higher distance from the pile to geophone 4 compared to all other geophones, the signal should reach geophone 4 at last which is not the case.

That is why, the following analyses concentrate on the Rayleigh wave velocity measured by geophones 1 to 3. Firstly, it has to be mentioned that reflections in the test pit are quite remarkable and lead to problems. Figure 27 shows the time series of geophones 1 to 3 with three major peaks at about t = 136.35 s, t = 136.75 s and t = 137.15 s for geophone 1. The time delay of about 0.005 s to geophone 2 can be detected (See also Figure 28). However, there is another minor peak for geophone 2 just in between at t = 136.6 s and t = 137 s. These minor peaks are reflections. For geophone 3 the situation is even worse. There are nearly no peaks detectable anymore because of a damping of the major peak and interactions with reflections.

Nevertheless, in most cases it is possible to distinguish between the different peaks and therefore, to calculate time delays. However, in the data there are thousands of excitations and peaks that have to be detected. So, some kind of automatized procedure is necessary. A straightforward peak picking algorithm has been developed that

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| *Figure 26: Time delays between the four geophones (Measurement 2.2.16, 5 m to 5.7 m depth, z-direction, pile hammer excitation during installation)* |

|  |
| --- |
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| *Figure 27: Reflections at pit walls (Measurement 2.2.16, 4 m to 5 m depth, y-direction, pile hammer excitation during installation)* |

matches peaks in the different signals considering some boundary conditions. Only peaks higher than a specific value are considered and only if the peak height of both peaks is similar. This restriction reduces the amount of falsely detected matches because only major peaks are considered. Furthermore, only peaks with a specific time delay are taken into account so that unrealistic high or low wave velocities that would only result from reflection effects are ignored. One example of the outcome of the peak picking algorithm is shown in Figure 28. The combination of blue crosses with blue circles symbolizes a match of a peak of geophone 1 with geophone 2. Red plus signs with red squares are a match of geophone 1 with 3 and green crosses with circles are geophone 2 with 3.

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| *Figure 28: Outcome of the peak picking algorithm (Measurement 28.1.16, 0m to 2m depth, z-direction, pile hammer excitation during installation)* |

The main peaks are matched quite well. However, there are some problems with peaks that are not sharp for example double peaks. In these cases, the time delay cannot be determined precisely, although, effects like this should not be significant if the number of matched peaks is high enough, as they average out. Nevertheless, the effect of unsharp peaks in combination with sampling rate of 2000 Hz, which is quite low for this purpose, leads to high standard deviations in the wave velocities.

For all four measurement days, data at different installations depths is available. The Rayleigh wave velocities have been calculated with the aid of the peak picking algorithm. The results of the wave velocities are summarized in Figure 29. The results in blue are based on geophone 1 and 2, in red are based on 1 and 3 and in green are based on 2 and 3. Black are the summed up results. The standard deviation of the results is given as well.

It becomes apparent that the standard deviations for most measurements are quite high and that the results differ a lot. This has several reasons. Firstly, as stated before, the peaks are partly unsharp. This leads to high standard deviations in each measurement in the first place. However, it is the reason for some differences in the mean values as well. The shape of the peak can lead to significant differences in the calculated wave velocity. To illustrate this problem, it is referred to Figure 30. Here, the left peak combination leads to too small velocities of about 100 m/s and the right one to too high wave velocities of about 200 m/s. For symmetric peaks (not shown here) the “correct” velocity will show up and lies in between. However, for some measurements, the majority of the peaks has such unsymmetrical peaks.

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| *Figure 29: Rayleigh wave velocities and their standard deviations. The results in blue are based on geophone 1 and 2, in red are based on 1 and 3 and in green are based on 2 and 3. Black are the summed up results.* |

|  |
| --- |
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| *Figure 30: Peak shapes and their influence on the wave velocity* |

The problem of very small time delays is more apparent for smaller distances. Hence, for the combination of geophone 1 and 3, it is less important which can been seen by comparing the standard deviations in Figure 29.

Additional problems occur for the first few hammer excitations at each pile. Here, the pile moves into the soil several centimeters every hit. For the measurements for that the pile is already driven into the soil several meters, this problem becomes less severe. However, another problem occurs. The excitation by the hammer is transferred to the pile into the soil. This happens over the whole pile. Therefore, the excitation is not at one point, but at a line down into the soil. As the wave velocity in the soil is higher compared to the surface because of the water in the sand, the signal becomes less clear. There are interaction effects in the signal of one excitation itself.

Nevertheless, it can be concluded that the wave velocity is probably somewhere in the range of 125 m/s to 250 m/s. The mean value is 175 m/s. These values are realistic if they are compared with the 150 m/s to 350 m/s in Table 5 for the shear wave velocity, since the Rayleigh wave velocity is then 135 m/s to 315 m/s and the wave velocity in sand is slightly smaller than in silty sand. However, the range is still quite large and only approximate statements can be done.

So far only excitations of the pile hammer during the installation have been examined. In order to underpin the results obtained so far, additional investigation have been made. As stated before, the additional tests are excitations of already installed piles with an impact hammer. The signals of these tests are slightly more distinct and interaction effects of subsequent excitations are not present. 16 excitations have been investigated at pile 2. The time between subsequent excitations was several seconds so that no interactions effects occurred. Hence, it was possible to analyze the first signals measured by all the geophones. These first signals are not the main peaks, as it can be seen in Figure 31*.* However, the main peak already shows some interaction effects and the peaks cannot always be detected properly. Nevertheless, the first main peaks were analyzed as well. The mean value of the wave speed for the first peaks is 168 m/s with a standard deviation of 9 m/s, whereas for the main peaks it is 177 m/s with 92 m/s standard deviation. It becomes clear that the use of the first peaks and single excitations is most beneficial.

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| *Figure 31: Peaks of single impact hammer excitation* |

Taking a mean wave velocity of 168 m/s and a three sigma interval, there is a range of 141 m/s to 195 m/s for the Rayleigh wave velocity and 157 m/s to 217 m/s for the S-wave velocity. If this range of the velocity is used together with the CPT-wave speed correlation of Tonni and Simonini [1]

, (19)

then there is a range of the cone resistance of 3.2 MPa to 6 MPa. This result corresponds quite well with the CPT results. The measured cone resistance in the first 0.5 m where the wave propagation occurs is exactly in this range (Figure 32).

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| *Figure 32: Cone resistances at the surface measured with CPT tests* |

Measurements of the wave velocities have an additional advantage. Wave velocities are not only correlated to CPT-results, but to other soil parameters as well. Frequently, the shear modulus () of the soil is of interest. For example, for foundations, the shear modulus is needed to determine the equivalent stiffnesses in the elastic half-space shall. To calculate the shear modulus, the shear wave velocity can be used. A solution of the three-dimensional wave equation is (for example according to Studer et al. [10]):

. (20)

The density of the sand has been investigated with the soil samples in section 2.1. A mean dry density of and a relatively small standard deviation of have been determined. Using these results are together with the shear wave velocities of the previous paragraph (mean: 187 m/s, standard deviation: 10 m/s), the mean shear modulus is with one and three sigma ranges of to and of to respectively. Although, the wave velocity values of the “first-signal-investigations” were used and these wave velocities have the smallest scattering, still, the deviations in the wave velocities are much more decisive than the variations in the density. Therefore, a precise measurement of the wave velocities is crucial for an adequate determination of the shear modulus or other soil parameters.

To summarize the results of the wave velocity test. The measurements in a test pit are not so straight forward because of reflections, short run times and interaction effects. However, it was possible to show that the measurement, especially of single excitations, allows to calculate the wave velocity. With the wave velocity, it is possible to approximate other soil parameters. Therefore, the use of wave velocity measurements can be a good alternative to classical methods of soil characterization. In particular, for field test, this method can be beneficial, as it becomes more appropriate in the field and methods like taking soil samples are more appropriate for test pits as in this project. Of cause, further investigations concerning this method of characterizing the soil have to be done.

# Dynamic tests

The dynamic tests concentrate on the dynamic behavior of the piles at different embedded lengths and examine the damping of the soil. Therefore, time series of accelerometers and strain gauges are analyzed with methods of the system identification.

## Basics in system identification

Modal properties of structures, especially of highly dynamic systems, are very important for the design. As offshore wind turbines are such highly dynamic structures and the piles cannot always be treated uncoupled to the rest of the turbine, the modal properties of the piles are important as well. In this context, particularly, the damping of the soil influences the overall turbine behavior.

There are many different approaches to calculate modal properties of structures with the aid of measurement data. Rudimentary approaches, like the fast Fourier transform (FFT) or power spectral densities (PSDs) in combination with visual peak picking for the eigenfrequencies or taking the natural log of the ratio of any two successive amplitudes for the logarithmic decrement (damping), can neither be automatized nor are they very accurate. Even quite common methods of the system identification, like the frequency domain decomposition (FDD) that is based on PSDs and peak picking, cannot be automatized. Therefore, system identification through data driven stochastic subspace identification (SSI), firstly introduced by Van Overschee and de Moor [11], is conducted here. This method is based on output-only measurement data and fits the data without using a physical model like FE or multi-body simulations. The automation of the system identification is carried out with the aid of triangulation-based extraction of model parameters (TEMP) based on the work of Häckell [12].

The SSI with the TEMP-method identifies automatically eigenfrequencies, mode shapes and damping ratios of the piles at different embedded lengths for all piles.

## Test procedure

All piles have been equipped with accelerometers. Piles 1, 3 and 4 had sensors only above the soil, whereas piles 2, 5 and 6 had sensors below the soil as well. For pile 2 and 5, there were only strain gauges below the surface. Pile 6 had accelerometers in addition.

Above the ground all six piles have been equipped with four high sensitive triaxial 20 g-accelerometers. These accelerometers have been attached to the piles with pile clamps, as they cannot be attached during the ramming. Therefore, they have been detached during the ramming. The measurements with these accelerometers have taken place during installation breaks and after the pile installation. To measure in between, the ramming process has been stopped every meter and dynamic tests have been performed. The first test for every pile has taken place after 3 meters of ramming for reasons of stability. The tests consisted of four excitations in all three directions and at different heights with an impact hammer. After the installation has been completed, further dynamic tests with excitations with an impact hammer have been conducted. Overall, there are more than 300 decay processes of the piles with different embedded lengths and varying excitations.

For piles 2, 5 and 6, measurements during the ramming and below the surface were possible with the strain gauges and the low sensitive 2000 g-accelerometers. Therefore, measurements at all embedded lengths were possible. The sensors distributed over the whole piles allowed a better identification of mode shapes below the surface. High damping ratios lead to nearly free decay processes even during the ramming, although it has to be stated that the pile guide in some cases and the hammer on top of the pile in all cases change the dynamic behavior for these measurements. Hence, a direct comparability is not given.

## Test data

The dynamic tests deliver a high amount of time series for free decay processes. These time series are analyzed with methods of system identification (e.g. stochastic subspace identification, SSI) and a comparison with a structural model is carried out in work package 7.4. Here, only a concise overview of the data evaluation is given and the data is adapted for WP 7.4. Therefore, only some results are presented proofing the plausibility of the gained data.

Figure 33 shows a time series of an impact hammer excitation. The free decay process can be seen and at least two eigenfrequencies are already noticeable.

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| D:\Users\clemens\Documents\4 IRPWind\Deliverable D72.2\TimePlot.jpg |
| *Figure 33: Free decay of pile 2 at 3 m embedded length, sensor at 0.4 m from the top* |

Figure 34and Figure 35 show two PSDs for different embedded lengths. The frequency range is limited to 100 Hz. This is done to concentrate on the first few eigenfrequencies that are most important here. A shift of the first bending mode (peaks at about 8 Hz and 15 Hz in Figure 34and Figure 35respectively) to higher frequencies for larger embedded lengths is apparent. Figure 36 shows a PSD with a broader frequency range. A very dominant peak at about 140 Hz can be seen. This eigenfrequency is nearly independent of the embedded lengths and is not a classical bending, torsion or axial mode of the pile. It is a radial mode where the cross section of the pile is deformed. As this mode is not of primary interest, it is neglected and the frequency range is limited to 100 Hz, as stated before.

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| D:\Users\clemens\Documents\4 IRPWind\Deliverable D72.2\FFTP6_3m_xy5_Sx1.jpg | D:\Users\clemens\Documents\4 IRPWind\Deliverable D72.2\FFTP6_6m_xy5_Sx2.jpg |
| *Figure 34: PSD of the free decay of pile 2 at 3 m embedded length, sensor 0.4 m below the top* | *Figure 35: PSD of the free decay of pile 2 at 6 m embedded length, sensor 0.4 m below the top* |

Further results concerning the change in eigenfrequency because of varying embedded length, the mode shapes and especially the damping behavior of the soil are not part of this report. These investigations are done in WP7.4 and are included in that deliverable.

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| D:\Users\clemens\Documents\4 IRPWind\Deliverable D72.2\FFTP5_25m_xy5_Sx1_long.jpg |
| *Figure 36: PSD of free decay of pile 5 at 2.5 m embedded length, sensor 0.4 m below the top* |

# Geotechnical tests

All the experimental phases are described in Section 1.5. In this chapter the experimental data are presented. The plausibility of the data is then discussed.

Figure 37 depicts the driving resistance of all the installed piles. As it was expected, Pile 1 and Pile 2, having the smallest diameter, present the lowest driving resistance. Since all the other piles do not present any difference in diameter, they should not show any discrepancy in driving resistance either. Nevertheless, Pile 4 and Pile 6 seem to have experienced more friction during the installation in comparison to Pile 3 and Pile 5. While this is explainable for Pile 6, as the pile was heavily instrumented from the outside (see Figure 10), it is not yet clear what caused this driving resistance increase showed by Pile 4.

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| *Figure 37: Driving resistance in blows per 10 cm* |

In Figure 38*,* the experimental raw data of the tensile tests for all the piles are plotted. The failure criterion for this geotechnical system was chosen as the force reached in correspondence to the 10% of the diameter length. The criterion for both pile diameters is shown as blue line in Figure 38.

All curves consist of a very stiff initial branch which starts to flat out after around 2 mm of displacement. It can be clearly noticed that the non-instrumented piles (i.e. Pile 1, Pile 3 and Pile 4) display a rather peculiar behaviour as failure approaches. The pile-soil interaction seems to be featured by cyclic reload branches followed by sudden unloads. At the time being the authors tend to attribute this phenomenon to the pile instrumentation. It is indeed deemed that this experimental observation might be related to a cyclic contact lost and regain between shaft and adjacent soil. It is however evident that these phenomena are in some way inhibited by the presence of instrumentation on the outer pile skin (i.e. Pile 2, Pile 5 and Pile 6). For similar reasons related to the presence of instrumentation on the pile external shaft, Pile 1, even though being shorter than Pile 2 and with equal diameter, presents a marked larger bearing capacity. The latter two observations corroborate the well-established hypothesis that pile instrumentation can dramatically influence bearing capacity and load carrying behavior of a geotechnical system.

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| *Figure 38: Raw data of the load-displacement curves for all the tensile loading tests* |

Overall, it can be concluded that the experimental data match the expectation and are therefore plausible. Care must be taken though when comparing the tests with each other as the type of instrumentation fitted to the pile significantly affects the bearing capacity as well as the bearing behavior.

# Conclusive remarks

The main objective of work package 7.2 task 7.2.2 within the IRPWIND project are geotechnical tests with model structures to determine soil-structure interaction effects in order to support probabilistic calculations of the reliability of offshore wind turbine support structures.

In regards to this aim, the conducted experiments are described in details in this report. The large-scale piles that were tested cover a broad range of different jacket piles (L/D ratios) and close a gap in investigated piles in literature (c.f. Figure 1).

In order to conduct accurate geotechnical measurements, well prepared soil (i.e. uniformly compacted sand) is required, and the soil has to be characterized as good as possible. Therefore, here, three different methods for soil characterization (soil samples, CPTs and wave propagation tests) were conducted. Soil samples and CPTs reveal a homogeneously compacted soil in vertical and horizontal direction. Both methods lead to similar results of relative densities of about 0.75. Wave propagation test were only used to analyze the near-surface soil. Here, CPT and wave propagation results are similar. Hence, the use of wave propagation tests (being faster and less expensive) might be an alternative for future soil characterizations, but have to be further analyzed especially with regard to measurements far below the surface.

Subsequent to the soil preparation, dynamic tests (impact hammer tests) of the soil-pile combination were conducted for different embedded lengths in order to identify varying eigenfrequencies, mode shapes and damping ratios. Using methods of the stochastic subspace identification, it is possible to identify several eigenfrequencies and modes shapes for nearly all dynamic tests. The raw data of the dynamic tests is handed over to WP7.4 for further analyses and investigations of soil-structure interaction models.

After the installation of all piles (and a waiting period to take settling effects into account), axial static bearing capacity tests (geotechnical tests) were conducted. Again, the raw data is handed over to WP7.4. Before handing it over, the plausibility was checked. The data matches the overall expectations, but should be handled with care. The instrumentation of some piles influences the pile behavior and the axial static bearing capacity. This fact has to be considered when using the data.

Finally, a probabilistic model for axially loaded piles was developed. This model uses the CPT and the geotechnical test data. The main uncertainty for axial static bearing capacities results from model errors. The uncertainty of the soil measurements (CPT uncertainty) can be neglected. Model errors of different numerical models for axial static bearing capacities are analyzed. The assessment reveals that model errors are differing significantly, and for some approaches, they are fairly high (see for example API-model in Table 4*).*

# References

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| [1] | L. Tonni and P. Simonini, "Shear wave velocity as function of cone penetration test measurements," *Engineering Geology 163,* pp. 55-67, 2013. |
| [2] | API, "Errata and Supplement 3 – API Recommended Practice 2A-WSD, Recommended Practice for Planning, Designing, Constructing Fixed Offshore Platforms – Working Stress Design," 2007. |
| [3] | A. Puech and P. Foray, "Refined Model for Interpreting Shallow Penetration CPTs in Sands," in *Offshore Technology Conference*, 2002. |
| [4] | CEN, "EN1990 (2002). Eurocode 0 - Basis of structural design," European Comitte for Standardization, Brussels, 2002. |
| [5] | Joint Committee on Structural Safety (JCSS)., "Probabilistic Model Code," Joint Committee on Structural Safety (JCSS)., http://www.jcss.byg.dtu.dk/, 2002. |
| [6] | M. Achmus and M. Müller, "Evaluation of pile capacity approaches with respect to piles for wind energy foundations in the North Sea.," in *2nd International Symposium on Frontiers in Offshore Geotechnics (ISFOG).*, Perth , 2010. |
| [7] | K. H. Stokoe, S.-H. Joh and R. Woods, "Some contributions of in situ geophysical mesurements to solving geotechnical engineering problems," in *Proc. ISC-2 on Geotechnical and Geophysical Site Characterization*, Rotterdam, 2004. |
| [8] | Y. Zaslavsky, A. Shapira, M. Gorstein, N. Perelman, G. Ataev and T. Aksinenko, "Questioning the applicability of soil amplification factors as defined by NEHRP (USA) in the Israel building standards," *Natural Science,* vol. 4, pp. 631-639, 2012. |
| [9] | G. Cai, A. Puppala and S. Liu, "Characterization on the correlation between shear wave velocity and piezocone tip resistance of Jiangsu clays," *Engineering Geology,* vol. 171, pp. 96-103, 2014. |
| [10] | J. A. Studer, J. Laue and M. G. Koller, Bodendynamik - Grundlagen, Kennziffern, Probleme, Berlin Heidelberg: Springer-Verlag, 2007. |
| [11] | P. VanOverschee and B. DeMoor, "Subspace algorithms for the stochastic identification problem," *Decision Control,* vol. 2, p. 1321–1326, 1991. |
| [12] | M. Häckel and R. Rolfes, "Monitoring a 5MW offshore wind energy converter-Condition parameters and triangulation based extraction of modal parameters," *Mechanical Systems and Signal Processing,* vol. 40, no. 1, pp. 322-343, 2013. |

# Appendix

The appendix contains data-sheets of the utilized pile hammer as well as of the accelerometers.