Geotechnical design monopile foundation Offshore Windpark Egmond aan Zee

Conception géotechnique des pieux de fondation Offshore Windpark Egmond aan Zee

A. Kooistra, J. Oudhof & M. Kempers

Ballast Nedam Infra Consult + Engineering, The Netherlands

ABSTRACT

The foundations of the turbine towers of the Egmond aan Zee wind park consist of monopiles with a diameter of 4.60 m, driven approximately 30 m into the seabed. The governing loads on the foundation are mainly horizontal loads, caused by wind, wave and current action. Geotechnical design comprises the modeling of the resistance of the soil by p-y curves and driveability analyses.

During installation of the foundation piles a pile-testing program was carried out. It can be concluded that the assumptions for the driveability analyses were conservative with regard to fatigue of the pile as a result of driving. The contribution however of the pile installation to the overall fatigue damage cannot be neglected.

RÉSUMÉ

Les fondations des turbines de parc de vente Egmond aan Zee sont des pieux simples avec un diamètre de 4.60 m, battus environ 30 mètre au fond de la mer. Les charges prédominantes sur les fondations sont des charges horizontales, causée par les actions du vent, des ondes et du courant. La conception géotechnique contient la modélisation de la résistance du sol par courbes p-y et des analyses du battage.

Pendant l'installation des pieux de fondation, on a exécuté une programme de teste. La conclusion est que, avec les assomptions utilisées, les analyses du battage ont surestimés le fatigue a cause de battage. La contribution au fatigue totale de l'installation des pieux quand-même ne peut pas été négligée.

Keywords : Seabed Morphology, Scour protection, P-y curves, Driveability, Pile Driving Analysis, Offshore, Wind park, Monopile

1 INTRODUCTION

In partnership with Vestas, Ballast Nedam has constructed the first offshore wind farm off the Dutch coast. The foundations of the turbine towers consist of steel monopiles with a diameter of 4.60 m, driven approximately 30 m into the seabed. The governing loads on the foundation are mainly horizontal loads, caused by wind, wave and current action. These loads are highly variable; therefore a fatigue analysis, including driving fatigue, is an important part of the design. Ballast Nedam Infra Consult + Engineering, the in-house engineering department of Ballast Nedam, was responsible for the design of the monopile foundation. In this paper the process of the geotechnical design, including the modeling of the lateral resistance of the soil by p-y curves and a driveability analysis are presented.

2 OFFSHORE FOUNDATIONS

The offshore wind farm comprises 36 turbines, sited 10-18 kilometres offshore, off the coast of Egmond aan Zee. First, a special filter layer was installed to prevent sand on the seabed from being swept away by the currents at the foundation. Subsequently, the foundation piles were driven in place. Then, the transition pieces were installed, on which the turbines would eventually be mounted. The transition piece and foundation pile are connected by a grouted connection. The turbine and tower components were assembled onshore and prepared for offshore transport and installation. Subsequently, the cables were installed and a scour protection is placed around the piles. A typical wind turbine construction is provided in figure 1.

Site investigations comprised weather, wave and current data. The turbine towers are founded on steel monopiles with a diameter of 4.60 m, with wall thicknesses ranging from 40 to 60 mm and weights up to 270 tonnes. The piles are driven approximately 30 m into the seabed with an IHC S1200 hydraulic hammer.

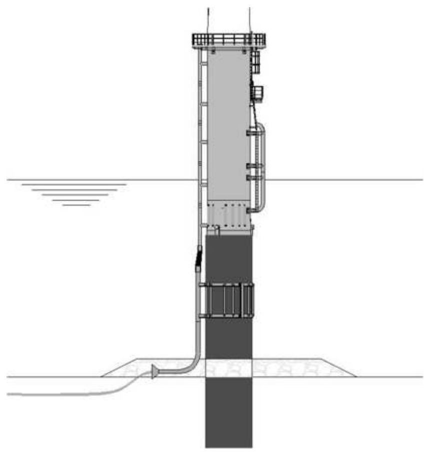

Figure 1. Wind turbine foundation

3 SOIL INVESTIGATION AND PILE GROUPS

An extensive soil investigation program was performed. This program included geophysical survey and geotechnical investigations. For the geotechnical investigations, 36 Cone Penetration Tests (CPT) were carried out, which is one at each turbine location. Furthermore nine borings were carried out, one at every four locations. On site the geotechnical classification tests were carried out, including torvane and pocket penetrometer tests.

In the laboratory, more classification tests were carried out, such as particle size distribution and Atterberg limits. To

Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering
M. Hamza et al. (Eds.)

doi:10.3233/978-1-60750-031-5-1111

determine the strength of the soil, unconsolidated and consolidated undrained triaxial tests were carried out. Also compressibility tests (oedometer) were carried out to determine the stiffness of the soil.

Figure 2 shows a typical CPT. The encountered soil layers are mainly medium dense to dense sand. Locally the top layers consist of loose sand, silt and clay layers. An old channel runs through the area, at that location more loose and silty material is found. This has resulted in the adjustment of some of the locations of the monopiles to a location outside the channel area.

Based on the performed geotechnical field investigation, a total number of 14 typical ground profiles were defined with relevant soil parameters. Each typical ground profile represents the location of one or more turbine locations. Based on the bathymetric survey at the location of the wind park, the water depth varies between 16 and 22 m.

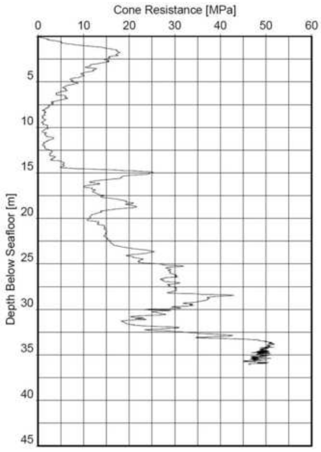

Figure 2. Typical CPT

4 SCOUR PROTECTION DESIGN

In the design process an evaluation was made to either let a scour hole develop around a monopile, and consequently design the foundation for this condition, or protect the surroundings of the monopile from a scour hole developing. Based on this evaluation the choice was made to provide all foundations with a scour protection.

The scour protection design is based on a set of physical modelling experiments, executed at WL Delft Hydraulics 'Schelde basin', based on a 100 year design storm event (figure 3). In the experiments the following investigations were done:

- Basic design: different experiments with varying scour protection diameter, armour stone size and armour layer thickness;
- Edge scour: the edge area is an important interface for the cable transition from scour protection to sea bed coverage;
- Sand waves: the wind park area is prone to migrating large scale sand waves.

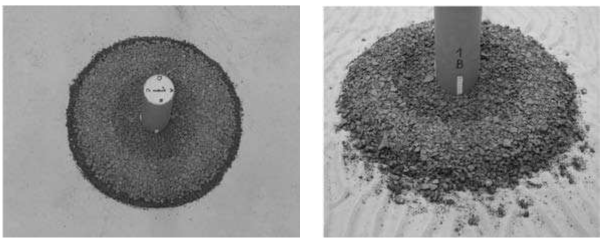

Figure 3. Experiments in the lab

The experiments have led to a dynamic stable scour protection, consisting of one filter layer and one armour layer. The design allows for some initial movement of individual armour stones, resulting in a stable situation after design events.

5 MONOPILE DESIGN

The Wind Turbine Generator (WTG) foundation must handle wind, wave and current effects for a projected lifetime of 20 years. A model of the full-integrated structure was built by using two computer programs: Flex5 and a pile-soil model ANSYS, respectively.

Flex5 is a time domain modal WTG model, which simulates wind turbine operations as well as the fully non-linear loads and response for the integrated wind turbine structure. The pile-soil model represents the support condition for the structure as a series of non-linear lateral springs (p-y curves).

The Flex5 model is loaded using nominal un-factored wave, wind and other loads and the output reactions at a series of pre-selected elevations down the structure are reported. The output reactions are then transferred as input loads to the pile-soil model and factored and combined using the safety factors and combinations for the ultimate limit state, fatigue limit state and serviceability limit state cases.

5.1 *Eigen frequency analysis*

From the pile-soil model the frequency of the first modal response is reported. Any modifications to the foundation needed to satisfy capacity criteria are identified. The modified structure and natural frequency is then passed back to the Flex5 model. After a few iterations this process will result in a eigenfrequency of the integrated system that satisfies the demands from the turbine supplier, allowing the eigenfrequency to stay within a small band-width.

Based on the water depths and typical ground profiles, in total four groups of essentially different foundations are defined. The four groups represent foundations with precisely the same eigenfrequency and therefore the same dynamic response. In this way the amount of necessary time domain simulations can be kept to a minimum.

5.2 *Fatigue analysis*

Based on the time domain simulations the fatigue loads and their frequencies are determined. By using the pile-soil model subsequently the fatigue stresses at every location in the structure can be obtained. In prescribed S-N curves (DNV (2004)) the cyclic stress level (S) is related to the amount of stress cycles (N) leading to a (fatigue) damage of 1.0. By using the S-N curves and applicable stress concentration factors (due to welds and/or wall thickness transitions) the resulting fatigue damage is determined for each transition piece and monopile section and consequently steel sections and details are checked. As part of the fatigue analysis the driving fatigue is determined based on driveability analyses.

5.3 *Stability and steel design*

From the integrated model the combined extreme wind-wave effect is determined to design for ultimate limit state and overall stability. Steel stresses are verified at governing sections of the foundation. The overall stability allows for limited plastic deformation of the upper soil layers.

5.4 *Deformations*

Deformations of the full structure must be limited to fulfil the requirements of the turbine designer. A maximum allowed rotation at the top of the mast must be adhered to. Furthermore, the deformations at the pile tip and at sea bed shall fulfil design demands, which are not prescribed by design rules.

At the sea bed, design loads may not lead to extensive plastic deformation of the top layers, at the pile tip a minimum elastic displacement is allowed.

6 P-Y CURVES

The interface between the foundation pile and the surrounding soil is modelled by means of multi-linear (elasto-plastic) springs. A common method for defining these springs is by means of load (P) - deflection (y) curves. This method is also stated in DNV (2004), API (2000) and Reese & Van Impe (2001).

The p-y curves give the relation between the integral value p of the mobilised resistance from the surrounding soil when the pile deflects a distance y laterally. The pile is modelled as a number of beam elements, supported by multi-linear springs, one per m length of the pile.

For the determination of p-y curves for the wind turbines, the type of soil, the type of loading and the pile diameter are considered. The p-y curves were established using low characteristic soil parameters for deformation and fatigue analysis and factored parameters for overall stability and ultimate limit state calculations. The soil springs are derived using the methods in the API [2] and these are calculated assuming cyclic degradation of the soil under repeated loading.

6.1 *P-y curves non cohesive soil*

The p-y curves for non-cohesive soil (sand) were established according to DNV (2004). The static ultimate lateral resistance per unit length (p_u [kN/m]) is calculated for the deflections y of the pile, typically taken at 0, 1/240D, 1/120D, 1/60D and 3/80D [m]. A typical curve is shown in Figure 4.

6.2 *P-y curves cohesive soil*

The p-y curves for cohesive soil (clay) were established according to DNV (2004) and Reese & Van Impe (2001). The cyclic P-Y relationship for clay is calculated according to API (2000) where p is calculated as a function of y. The deflection y is typically taken at 0, $0.22y_c$, y_c, $3y_c$ and $15y_c$, in which $y_c = 2.5D*\varepsilon_{50}$ and D is the diameter of the pile and ε_{50} is the strain that occurs half of the maximum stress in laboratory undrained compression tests. A typical curve is shown in Figure 4.

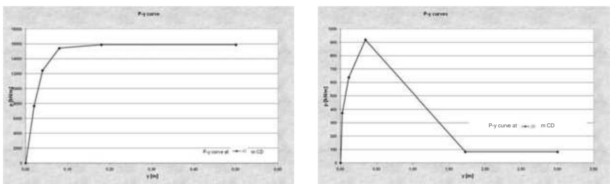

Figure 4. Example p-y curve non-cohesive and cohesive soil

7 DRIVEABILITY ANALYSIS

The driving fatigue has been predicted with the results of the driveability analysis. The driveability analysis included a dynamic analysis based on stress wave propagation through the pile. The stresses during driving and the number of blows were determined using the GRL WEAP (Wave Equation Analysis of Piles) program. The GRL WEAP program simulates the behaviour of a pile driven by (in this case) an impact hammer. The program uses the stress wave theory to describe motion and forces of hammer, driving system, pile and soil under the hammer action.

Although the 36 piles have different lengths and wall thicknesses, a typical pile is described here. The pile is divided into several sections with a wall thickness between 45 mm and 60 mm.

The IHC S1200 hammer details are modelled as per manufacturer's recommendations. The rated efficiency as per manufacturers recommendations is 95%. In the calculations, we have applied a hammer efficiency of 90% above water and 80% if the hammer is under water.

The location of the wind turbine location considered for the driveability analysis is expected to have one of the hardest soil conditions of the site and is therefore assumed to be governing for the fatigue analysis.

The soil resistance to driving (SRD) consists of the sum of the shaft friction and the pile tip resistance. The SRD values were calculated according to Stevens (1982) and Alm & Hamre (2001). It appears that in this case the method according to Alm & Hamre (2001) results in approximately 15% lower SRD values and a different distribution along the pile. Therefore the calculations were performed according to both methods. The results of the SRD calculations are presented in Figure 5.

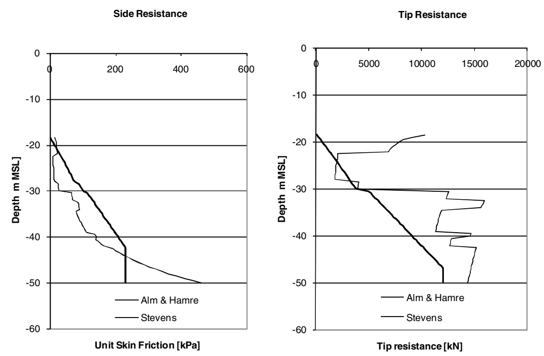

Figure 5. Soil resistance to driving

Assuming the maximum driving energy of 1200 kNm of the IHC S1200, the expected maximum compressive stress was 169 N/mm^2 and the maximum tensile stress 139 N/mm^2. The pile can be driven to its maximum depth with a total number of 1875 blows for the reference pile. Based on the amount of blows (number of cycles) and the stress range, being the sum of the compressive stress and the returning tensile stress, the fatigue damage to the pile can be obtained for every meter pile section. The maximum found fatigue damage was 0.13 for the governing pile section, meaning that 13% of the allowable fatigue damage has occurred solely due to driving of the monopile.

8 PILE DRIVING ANALYSIS

During installation of the foundation piles a pile testing program was carried out, comprising blow count recording on all piles and dynamic pile tests on two selected piles. The dynamic tests were used to measure the compressive and tensile stresses during driving.

Pile Driving Analysis (PDA) is a high-strain dynamic testing method of piles which is a relatively simple and competitive way to control the pile driving process. Whilst performing a PDA measurement, valuable data is recorded, monitored and analyzed:

- Force and velocity response of the pile to a driving impact force;
- Blow count, blow rate and set per blow;
- Hammer efficiency and transferred energy;
- Pile tension and compressive stresses during driving;
- Soil resistance to driving (side and tip resistance).

The PDA tests were carried out during driving of two foundation piles. It was expected these piles have the hardest driving conditions, due to either their large embedded length or hard soil conditions. The PDA FPDS-7 system of Profound b.v. has been used.

8.1 *Measured stresses*

From the PDA measurements the maximum compression and tension can be derived. The measured values are the stresses at the level of the sensors, i.e. 10.3 m from the pile top. The results of one of the test piles are presented in figure 6.

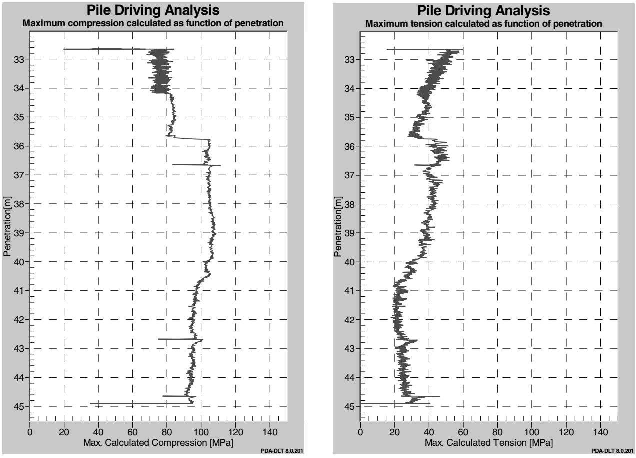

Figure 6. Results PDA test

For both piles the first meters with low soil resistance and low hammer energy the compression is 60-80 MPa, the tension is 50-60 MPa. From ca MSL – 35 m to –36 m the compression stresses increase to 95-110 MPa and the tension decreases to 20-40 MPa.

In the fatigue analysis maximum stresses of ca. 169 MPa compression and ca. 159 MPa tension were accounted for. The measured stresses of the test piles are lower than the calculated values, probably due to lower transferred energy.

8.2 *Blow count results*

During driving the blow count has been recorded for all piles. The total number of blows using the S1200 with hammer energy 1200 kNm was expected to be 1875 for hard soil conditions.

Most piles exceed the total number of blows of 1875 because of lower transferred energy. However the stresses during driving are lower, due to the lower transferred energy and probably lower soil resistance. Exceeding the number of blows did, in this case, not increase the driving fatigue.

8.3 *Back-Analysis*

The results of the PDA measurements were used to perform a back-analysis using GRL WEAP. This analysis was performed for the same reason as the driveability analysis before: the stresses for each half meter section could be determined in time. With these data, the fatigue damage could be calculated. As the actual driving was easier than expected, the driveability analysis, made as a prediction, overestimated the soil resistance. Also the fatigue as a result of driving was therefore overestimated. In the back analysis of the dynamically tested piles the driveability analysis was fitted to the blow counts encountered on site, the stresses measured during the PDA measurements and the actual hammer energy.

For the back-analysis, clearly the hammer energy was reduced, and so was the soil resistance. With these adjustments, the higher blow counts but lower stresses were found. It was possible to fit the analysis reasonably well to the results found on site. Only the somehow higher tension stresses measured on site in between NAP -35,5 m and NAP -36 m could not be simulated with the driveability analysis. Based on the back analysis a driving fatigue of 0.08 was found, a lower driving fatigue than predicted in advance.

9 CONCLUSIONS

- It can be concluded that site investigation at every location is necessary for this kind of project.
- Lateral pile capacity is modelled by means of multi-linear (elasto-plastic) springs, p-y curves according to (API 2000).
- Extensive physical model experiments have led to a dynamic stable scour protection design.
- Using the PDA results a back analysis was performed of the driving process. It was found that the actual driving damage was lower (0.08) than the estimated damage based on the driveability analysis as performed before driving (0.13).
- It can be concluded that the assumptions for the driveability analysis were conservative with regard to fatigue of the pile as a result of driving.
- Driving fatigue cannot be neglected in the determination of the overall fatigue damage of a monopile

REFERENCES

Offshore Standard DNV-OS-J101, 2004, Design of offshore wind turbine structures, Det Norske Veritas (DNV),.

API (2000), Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms, API Recommended Practice 2A-WSD, 20th Edition.

Reese, L.C. and Van Impe, W.F., 2001, Single Piles and Pile Groups Under Lateral Loading, Balkema Rotterdam

Stevens, R.S., 1982, Evaluating Pile Drivability for Hard Clay, Very Dense Sand, and Rock, OTC Publication 4205, Offshore Technology Conference.

Alm, T. and Hamre, L., 2001, Soil model for pile driveability based on CPT interpretations, International Conference On Soil Mechanics and Foundation Engineering.

GRL Weap Wave Equation Analysis of Pile Driving, Procedures and Models, version 2003, PDI inc., 1998-2003.