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Wave Loads on Monopile Structures

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Abstract
This article compares different approaches to determine the design forces and moments for the ultimate limit state for monopile structures for offshore wind turbines. The principles of the different approaches are explained. Four different methods, i.e. model test, volume of fluid method, boundary element method and the classical Morison approach are compared to each other.

Zusammenfassung
Wellenkräfte auf Monopile-Strukturen


Aus dem Vergleich der Berechnungen werden Schlüsse auf die zu verwendenden Berechnungsverfahren für die Konstruktion und Genehmigung zukünftiger Windparks gezogen.

Motivation
Monopiles have become the dominating foundation structure for offshore wind sites in Europe in recent years. Due the increasing size of the turbines and increased water depth for the wind farm sites
it is expected that the size of the monopiles will increase in the upcoming years. The pile are exposed
to varying loads from the turbine, wind, waves and currents. The large number of load combinations
and the dominance of the fatigue limit state in the structural design requires fast but reliable design
tools. The development of the size requires that the design assumptions and tools for the monopiles
are reviewed. The standard approach for determining wave and current loads both for fatigue and
ultimate limit state is the so-called Morison approach. This approach was proposed by Morison et al.
in 1950 for the determination of wave loads on slender piles (see Morison 1950). It was developed for
the design of the first offshore oil rigs, which have been installed in the Gulf of Mexico since 1947.
Since then it has been a standard tool for the offshore industry. However, if one compares the design
condition of the first offshore oil rig the “Kermag Rig No. 16” to the condition of a modern monopile
foundation used in the North Sea substantial differences can be observed. The Kermag rig was founded
on 16 wooden piles with a diameter of D = 0.6 m in d = 6 m water depth (see figure 1). A modern
monopile foundation can have a diameter of up to D = 8 m. However, they are still installed in water
depth of about d = 20 m with design wave heights of about H_{max} = 15 m. This decreases the diameter
to water depth fraction from about 10 for the Kermag rig to 2.5 (see also figure 2). In order to prove if
the design assumptions of Morison are still valid for this size of piles different approaches for the de-
termination of the ultimate limit state condition are compared to each other.

![Figure 1: First "offshore" oil rig "Kermag Rig No. 16" in 1947 (Source: New Orleans Times-Picayune)](image)

![Figure 2: Pile diameter and water depth for the Kermac Rig No. 16 (left) and modern monopile on the North Sea with indicated design wave (right)](image)
Determination of wave loads

This article focuses on the determination of the wave loads on fixed slender members of offshore structures in particular monopiles. The term fixed slender members implies that there is no wave radiation from a moving structure. Furthermore, due to the limit diameter of a slender structure compared to the wave length the disruption of the wave by the existence of the structure is marginal and can therefore be ignored. In principal, for the determination of the loads one can distinguish between three different methods whereas some of the methods are further subdivided. The methods are described more in detail in the following subsections before the methods used for the presented work are outlined.

Empirical Methods

The first group of methods are empirical methods. These can be further subdivided into full scale measurements and model tests. In both cases it is quite challenging to develop measurement strategies that allow the determination of the forces and moments acting on the structure without interference of the waves. In full scale campaigns, the forces cannot be measured directly. Instead structural responses of the structures are measured with stress and acceleration sensors (see e.g. Camp et al., 2003). Hence, the evaluation of the measurements requires an accurate structural model. Full scale measurements can only be used for the calibration of prediction tools or the collection of hind cast data for the determination of the actual structural life, e.g. for live time extension campaign. But, they cannot be used directly for the prediction of the forces acting on a structure in the design stage. In addition, full scale measurements are dependent on the weather and wave pattern. Hence, even in a measurement campaign which lasts for several years it cannot be ensured, that extreme events can be observed. In contrast, model tests allow reproducible measurements for determined wave conditions. The validity of model tests depends on a test setup which allows accurate measurements, scale effects which determine the possibility to scale the results to the full scale and the properties of the test basin, i.e. size of the model, wave height and steepness, as well as water depth. A proper design of the model set-up requires for instance that if forces on a rigid body shall be determined, that the body is as rigid as possible. As wave motions are fluctuating boundary layer effects are generally not as critical as for example in classical ship model or wind tunnel tests. The effects of gravity waves can be scaled without significant problems as long as effects which depend on surface tension are negligible.

Semi-empirical solutions

The second type of methods can be classified as semi-empirical. The most prominent example of these methods is the so-called Morison approach, which was developed by Morison et al. in 1950 (Morison et al.). From observations and evaluation of model tests Morison and his colleagues concluded that the wave forces acting on a slender member can be divided into two parts which are superimposed. The first part is inertia driven and depends on the acceleration of the fluid at the position of the member. The second part is drag driven and depends on the signed square of the velocity of the fluid. In particular the force acting on a section of a circular member in the direction of the fluid flow can be determined as follows:

\[
f_N = \rho (1 + C_A) \frac{\pi D^2}{4} \dot{\nu} + \frac{1}{2} \rho C_D D |\nu|
\]

with \( \rho \) being density of the fluid, \( D \) the diameter of the section, \( \dot{\nu} \) acceleration of the fluid, \( \nu \) velocity of the fluid, \( C_A \) added mass or inertia coefficient and \( C_D \) the drag coefficient. For a certain point in time
the force and resulting moment acting on a monopile can be determined by integrating the equation above along the wetted part of the pile. At first glance, this approach looks very simple. However, there are some important aspects to be considered. Amongst others, the added mass and drag coefficient need to be determined as well as the acceleration and velocity of the fluid. The first is determined from model tests or numerical simulations. The latter is typically done by using a wave model. Therefore, the validity of the results obtained by the Morison approach is highly dependent on the validity of the wave model.

Another aspect which needs to be considered for the implementation of the Morison approach is, that due to the orbital motion in a wave the acceleration and the velocity have a phase shift of 90°. In practice, this means that either the inertia or the drag component are dominant. If the diameter is relatively large compared to the wave length the forces are inertia driven, if it is small than they are more drag driven.

**Numerical Simulations**

The third type of methods are numerical simulations. Although implementations of the Morison approach are also dependent on the discretization of the members in each section it is normally not considered a numerical method. For the problem under investigation one can consider three types of numerical simulations: boundary element methods, field or mesh depending methods and field independent methods like smoothed-particle-hydrodynamics (SPH).

Numerical simulations have key advantages: Full scale investigations can be performed, there are less limitations for geometry than for instance for model tests and detailed evaluation of the flow field properties can be performed both in space and time domain. The major disadvantages are, that although computational resources can be bought or rented at low costs, the investigation of more complex problems still requires hours or even days of computational time and might even be more expensive than experimental tank tests (see Schmitt et al. 2012).

Boundary element methods are usually based on potential flow theory. As already stated above gravity wave interaction is mainly an inertia and gravity driven process, potential flow theory can thus capture the most important effects. The definition of surface grids is much easier in most cases as for volume mesh. Therefore, the preprocessing efforts are very limited. However, the boundary element methods have two major disadvantages: classical potential flow implementations result in solving dense equation systems. Hence, solving the equation system is time consuming. The second disadvantage is that effects like splashes or effects depending on friction cannot be captured with these methods.

Field methods in particular numerical implementations of the Navier-Stokes equations together with a volume of fluid approach (VoF) for multiphase flows are state of the art in the industry for various applications. The main advantage in the context of sea state computations is the intrinsic ability to treat transient problems, especially under the consideration of geometrical and topological aspects of the free surface. Another advantage is the availability of viscous source terms, which enables the dissipation of energy from the system. These two features allow the representation of breaking and spilling waves.

Smoothed-particle-hydrodynamics (SPH) methods are very suitable for capturing free surface effects like splashes or droplets. However, for engineering design purposes they are still in development and have therefore not been considered in this work.
Used Methods
In the presented work four different methods have been used to determine the shear force and over-turning moment caused by design waves on a monopile.

Morison Approach
Calculations employing the Morison approach were done with the software Wave Loads which was developed by the Institute of Fluid Mechanics, Leibniz Universität Hannover, Germany. The software allows to use different wave models starting from Airy wave theory over 2nd to 5th order Stokes implementations to various stream functions.

Model Tests
To validate the calculation results model tests have been conducted in the small towing tank of HSVA in Hamburg. The tank is 80 meters long, 3.8 meters wide and 2.45 meters deep. The wave generation is done by a modern wave fold generator. Waves up to a wavelength of six meters and a wave height of 0.3 meters can be created. Two different pile configurations have been tested. As test piles two PVC-tubes have been used. A plug at the still water level prevented vertical water flows. Forces were measured at two positions above the water surface to calculate the wave forces and moments. In order to generate a wide field of validation points regular waves with different wave heights and lengths were produced and tested. All configurations have been tested at least twice to minimize measurement errors and assess repeatability (see Puder 2017).

BEM Method
For the calculation of forces and moments with the BEM method the potential solver panMARE of the “Institute for Fluid Mechanics and Ship Theory” of Hamburg University of Technology is used. An incompressible, inviscid and rotation free flow model is used and the potential equation is solved. It consists of the flow potential and the cylinder induced potential, which is calculated after the method of Katz (2001). To solve the potential equation, the velocity potential must be known. Therefore, the Neumann and Dirichlet boundary condition is used.

Figure 1: Surface mesh for the BEM (left) and dynamic pressure on the pile for one condition (right) (Puder 2017)
The free surface discretization consists of an inner circle with a radius of one wave length and an outer circle for damping with a radius of half a wavelength. The monopile also consists of two areas. The upper part of the discretization moves with the wave elevation to handle bigger wave amplitudes. The lower part is fixed. Figure 3 shows a snap shot on the surface mesh discretization.

**VoF Method**

The Numerical Wave Tank implementation for this case study is based on the interDyFoam solver from the OpenFOAM toolbox, which uses a volume of fluid (VOF) approach, for modelling the two different fluid phases of air and water, to capture the free surface. More details on this solver can be found in Rusche (2003).

Wave generation is implemented by adding a source term to the impulse equation, similarly a numerical beach is implemented by a dissipation term. This implementation has been shown to recreate physical waves accurately over a wide range of conditions while minimizing reflections from boundaries (wee Windt et al. 2017, Schmitt 2017, Schmitt and Elsaesser 2015). The domain is modeled according to the dimensions of the wave tank dimensions of the model test, but is truncated in length to reduce the numerical burden.

**Test Case**

The test case which is considered for the presented work is a monopile of 8 m diameter which is installed on 20 m water depth. The scenario corresponds to actual developments in the eastern North Sea. The design wave for these sites is estimated with a height $H = 16.3$ m and a period of $T = 12.46$ s. This results in a relative wave height of 0.011 and relative water depth of 0.014. In figure 4 these parameters are plotted in diagram showing the applicability of different wave theories. The orange dot is the design condition. It can be seen that the design wave is already a breaking wave. The investigation of breaking waves requires great effort both for model tests and numerical assessment as a single wave needs to be generated by overlapping waves of different lengths or periods. In order to reduce the efforts for the model tests and due to the given water depth in the model basin it was decided to investigate the same wave steepness although on a larger water depth (blue dot). The corresponding values for the relative wave height and relative water depth read 0.011 and 0.01 respectively.
Detailed Results

Investigations have been performed for various wave heights, periods and model scales. Emphasis was given to the above described modified design wave (blue dot in figure 4), with a relative wave height of 0.011 and a relative water depth of 0.01. Figure 5 shows a closeup view of the wave elevation and local slope at the monopile for various time steps of the said case. Results from the model tests are shown in the left and from the VoF computations on the right. The agreement with regard to the local and global effects is very good.
Figure 6 shows the shear force and overturning moment over time for the same configuration as determined with Wave Loads, the BEM solver PanMare, OpenFOAM and model tests. All values are for the model scale with given relative water depth and wave height for a pile diameter of $D = 0.125$ m. The highest forces and moments can be observed when the flank passes the pile. When the crest or the trough are at the center of the pile the resulting force and moment is close to zero. For the shear force Wave Loads gives the same results as the BEM computation. But, the comparison to the model tests and the VoF computations shows that the shear force is underestimated. The same result can be seen for the overturning moment, although the BEM method shows better results. The model test results show a hump at the upwards zero crossing which is when the wave trough hits the pile. The volume of fluid method underestimates the overturning moment when the backside flank of the wave passes the pile. This requires further investigations. One potential reason for this it was observed in the model tests, that the model pile was not fully rigid.
Summary and Outlook

In this article, different methods for the determination of the wave design loads on monopiles have been presented. Although the Morison approach is the standard tool in the offshore industry it needs to be used with great care. The results from the Morison approach in particular for extreme wave conditions, like they can be observed in the eastern North Sea, are highly dependent on the reliability of the wave model and the correct choice of drag and inertia coefficients. Therefore, comparative investigations should be made with model tests or numerical methods. Results from the VoF-computations are very promising. Further investigations should focus on the reliability in different wave conditions as well as on wave conditions which are relevant for the fatigue assessment.

References


Figure 2: Shear force and overturning moment of the monopile


