HYDRODYNAMIC LOADING ON MONOTOWER SUPPORT STRUCTURES FOR PRELIMINARY DESIGN

J.H. Vugts, J. van der Tempel and E.A. Schrama
Interfaculty Offshore Technology, Faculty of Civil Engineering and Geosciences,
Delft University of Technology
Stevinweg 1, 2628 CN Delft, The Netherlands. Tel. +31 15 27 86828 Fax +31 15 27 85124
Vugts@offshore.tudelft.nl; J.vanderTempel@offshore.tudelft.nl; E.A.Schrama@offshore.tudelft.nl

ABSTRACT: To determine the global dimensions of the support structure of an offshore wind turbine the hydrodynamic forces and moments need to be calculated in a fast and straightforward manner. To this end Airy Linear Wave Theory and the Morison Equation can be used. The calculation in combination with the single diameter cylinder of the part of the monotower in the water can be solved analytically. The solution can be plotted in look-up graphs to significantly speed up the design process without losing touch with the physics.
Keywords: Offshore, monotower, hydrodynamic loading, design method

1 INTRODUCTION

Successful application of offshore wind energy requires the effective integration of two complex engineering fields, i.e. wind energy and offshore engineering. As we are on the threshold of an expected boom of large offshore parks, it is essential that parties on both sides create adequate understanding of each other's technical area. Situations in which a turbine manufacturer simply requests an offshore company to design and install a support structure for him on which he can put his mills or, conversely, in which an offshore company comes up with a support structure design and then requests a turbine manufacturer to provide him with a turbine to put on top, must definitely be avoided. The wind energy converter and the support structure form an integrated technical system that must be developed in mutual interdependency and close co-operation. This paper provides a contribution to this integration process by explaining the hydrodynamic loading on monotower support structures in simple terms and by presenting results that are easy to use in design practice.

The additional challenge that an offshore wind turbine presents over design and operation of a land-based installation is the offshore environment. For design purposes, this manifests itself in the several meters of water surrounding its foot. When moving from the shallow, sheltered waters of the first offshore parks in the Baltic and the IJsselmeer into the exposed and appreciably deeper waters of the North Sea, a monotower on a monopile foundation is expected to offer the most favourable support structure concept.

2 DESIGN PROCESS

The first step in the design process of a support structure of an offshore wind turbine is the determination of its global dimensions. The process can be divided into two interconnected parts. First the quasi-static response of the structure to extreme load cases with or without Dynamic Amplification Factor (DAF) is determined. The structure must be strong enough to withstand the biggest loads that are likely to occur. Second, a dynamic analysis must demonstrate that the tower's natural frequency does not coincide with the frequencies associated with rotor rotation and blade passing, nor with high energy wave excitation, to prevent resonant behaviour and accompanying fatigue damage. Figure 1 shows the flowchart of the preliminary design process.

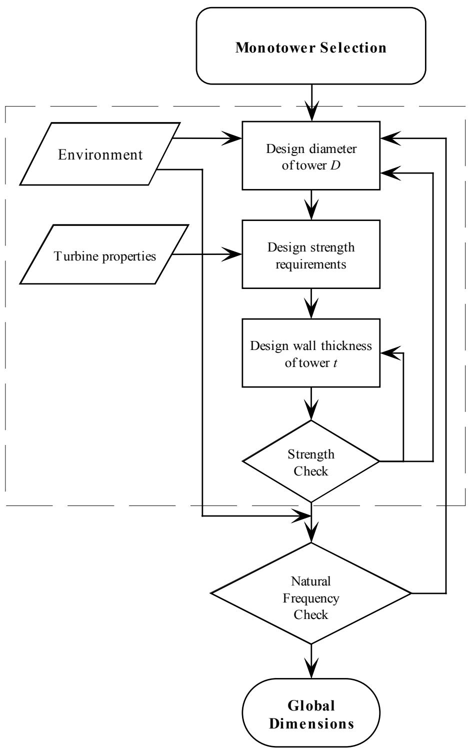

Figure 1. Flowchart of the preliminary tower design

The strength check of a monotower, in the box with a dotted line in figure 1, is shown in more detail in figure 2. For a number of extreme load cases the wind and wave loads on the structure are determined and combined with the gravity loads. For an offshore wind turbine these extremes may not necessarily occur in the severest storm, they can also be associated with events during maximum energy production: the aerodynamic loads are much larger when the rotor is rotating than when it stands still.

The hydrodynamic forces are subject to large changes with every new design step. The diameter D greatly influences the magnitude of these forces. An initial value has to be chosen to start the optimisation.

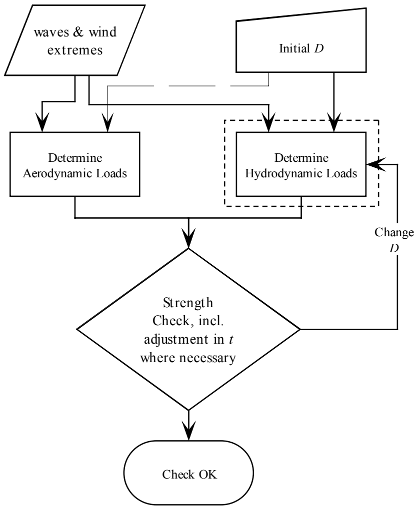

Figure 2. Detailed view on the tower strength check

The determination of the wave loads can be done using a variety of wave theories. For preliminary design it is best to use a simple and straightforward method: the Airy Linear Wave Theory. Based on wave height (*H*), period (*T*) and depth (*d*), this theory will return wave particle velocities (u) and accelerations ($\dot{u}$), which can be used in the Morison Equation to be translated into forces (*F*) and moments (*M*) on the structure. The details of this calculation step are shown in figure 3.

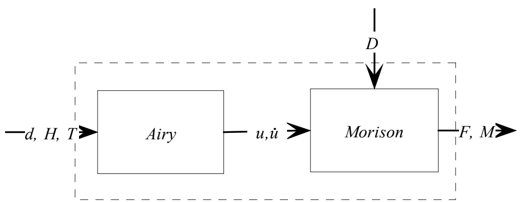

Figure 3. Steps to determine the hydrodynamic force and moment

3 AIRY WAVE THEORY AND THE MORISON EQUATION

The Airy Linear Wave Theory is the most widely used and straightforward way of calculating water particle velocities due to waves. The water particles are thought to describe closed orbits that are circular for deep water waves and elliptical for waves in intermediate water depths. The radius of the circle and the axes of the ellipse (long axis horizontal, short axis vertical) both decrease with depth below the free surface; see figure 4. As the linearised free surface boundary condition is applied at the mean still water level, the theory is not capable of predicting kinematics up to the wave crest.

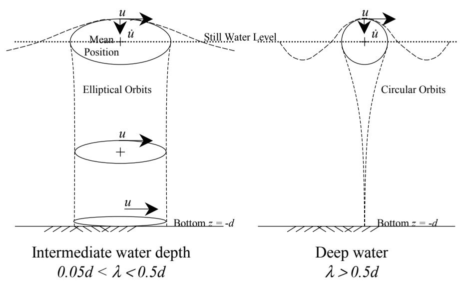

Figure 4. Particle orbits according to the Airy theory

The horizontal water particle kinematics are described as follows [1], with the *z*-axis pointing upwards from the free water surface and position *s* horizontal; *t* is time.

$$u(s,z;t) = \hat{\zeta}\,\omega \frac{\cosh k(z+d)}{\sinh kd}\cos(ks-\omega t)$$
$$\dot{u}(s,z;t) = \hat{\zeta}\,\omega^2 \frac{\cosh k(z+d)}{\sinh kd}\sin(ks-\omega t) \qquad (1)$$

Where $\hat{\zeta}$ = wave amplitude, k = wave number = $2\pi/\lambda$, ω= wave frequency and λ =wavelength. The Morison Equation is an empirical formula to calculate the hydrodynamic forces on slender members with a drag and inertia coefficient (C_d, C_m) and water density ρ. It contains a drag force (f_d) and an inertia force (f_i) using the velocity and acceleration (u and $\dot{u}$) from the Airy theory.

$$f(s,z,t) = f_d(s,z,t) + f_i(s,z,t)$$
$$f_d(s,z,t) = C_d \cdot \tfrac{1}{2}\rho D \cdot \left| u(s,z,t) \right| u(s,z,t) \qquad (2)$$
$$f_i(s,z,t) = C_m \cdot \frac{\rho\pi D^2}{4} \cdot \dot{u}(s,z,t)$$

Figure 5 shows the representation of a slender member under hydrodynamic loads.

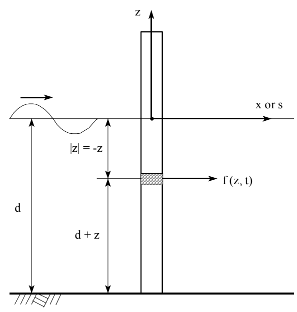

Figure 5. Slender member with hydrodynamic loads

The sum of drag and inertia load is the total hydrodynamic load on the cylinder. Note that velocity and acceleration have a 90^{o} phase difference, so inertia and drag loads will also be out of phase. This means that in general the maximum force is not equal to either maximum drag or maximum inertia force.

4 ANALYTICAL SOLUTION FOR WAVE LOADING

The analytical solutions for the total hydrodynamic force (*F*) and the overturning moment (*M*) are obtained by integrating f_i and f_d from the seabed z=-d to the instantaneous water surface elevation ζ:

$$F(s,t) = \int_{-d}^{\zeta} \{ f_i(s,z,t) + f_d(s,z,t) \} dz$$
$$M(s,t) = \int_{-d}^{\zeta} \{ f_i(s,z,t) + f_d(s,z,t) \} \cdot (d+z) \cdot dz \qquad (3)$$

The resulting equations [2] can be simplified when the integration extends from the sea floor to the still water level z = 0. This simplification does not affect the inertia force, which reaches a maximum when the wave surface has a zero crossing, but it discards the additional wave drag load during the passage of the wave crest. Though this effect can be significant if drag load dominates, it will be shown in the next section that for the particular application considered here, the simplification is valid.

The magnitudes of the inertia and drag force and moment ($\hat{F}_I$, $\hat{F}_D$, $\hat{M}_I$, $\hat{M}_D$) are then (using the dispersion relation: $\omega^2 = gk \cdot \tanh(kd)$ with g the gravitational acceleration):

$$\begin{aligned}
\hat{F}_I &= \rho g \frac{C_m \pi D^2}{4} \hat{\zeta} \cdot \tanh kd \\
\hat{F}_D &= \rho g \frac{C_d D}{2} \hat{\zeta}^2 \cdot \left[\frac{1}{2} + \frac{kd}{\sinh 2kd} \right] \\
\hat{M}_I &= \rho g \frac{C_m \pi D^2}{4} \hat{\zeta} \cdot d \left[\tanh kd + \frac{1}{kd} \left(\frac{1}{\cosh kd} - 1 \right) \right] \\
\hat{M}_D &= \rho g \frac{C_d D}{2} \hat{\zeta}^2 \cdot \left[\frac{d}{2} + \frac{2(kd)^2 + 1 - \cosh 2kd}{4k \sinh 2kd} \right]
\end{aligned} \quad (4)$$

5 WAVE LOAD ON A SINGLE CYLINDER

5.1 The art of reduction

An overview of the critical factors for preliminary design will aid the designer in making a good first estimate of the dimensions of the structure. The monotowers, as they have been and will be constructed in present day projects, are of substantial diameters and will be placed in relatively shallow coastal regions, which make a reduction of the full scope of the wave force calculation problem possible.

5.2 Intermediate water depth and large diameter towers

The offshore wind turbines under investigation will be installed in intermediate water depths not very far from the coast. Water depth is defined in relation to the wave length λ and translated into the product kd:

deep:	$d > \frac{1}{2}\lambda$	and	$kd > \pi$
intermediate:	$\frac{1}{20}\lambda < d < \frac{1}{2}\lambda$	and	$0.1\pi < kd < \pi$
shallow:	$d < \frac{1}{20}\lambda$	and	$kd < 0.1\pi$

Examples of North Sea support structures with maximum wave height, period and water depth are:

	H_{max}	T	λ	d	D
	(m)	(s)	(m)	(m)	(m)
Opti Owecs	12.8	9.5	113.3	20	3.5
Blyth	8	7	50.7	8.5	3.5
Horns Rev	8.1	12	129.4	13.5	4

Compared to deep offshore waves, the waves in these coastal areas are relatively short and lower by comparison. This fact combined with a relatively large diameter will often point to dominance of the inertia loads.

5.3 The load components

The ratio of $\hat{F}_D$ to $\hat{F}_I$ is, introducing $H = 2\hat{\zeta}$:

$$\frac{\hat{F}_D}{\hat{F}_I} = \frac{1}{\pi} \cdot \frac{C_D}{C_M} \cdot \frac{H}{D} [A + B]$$

with: $A = \frac{1}{2 \tanh kd}$ and $B = \frac{kd}{2(\sinh kd)^2}$

For intermediate water depth:

- deep water limit: $kd = \pi$.

$\tanh kd \approx 1.0 \qquad \sinh kd \approx 10$

$A \cong \frac{1}{2} \qquad B \cong \frac{\pi}{(2)(100)} = 0.0157 \qquad A + B \approx 0.52$

- shallow water limit: $kd = 0.1\pi$.

$\tanh kd \approx kd = 0.1\pi \qquad \sinh kd \approx kd = 0.1\pi$

$A \approx \frac{1}{(0.2)(\pi)} = 1.59 \qquad B \cong \frac{(0.1)(\pi)}{(2)(0.1)^2(\pi)^2} = 1.59$

$A + B \approx 3.18$

Including drag and inertia coefficients C_D=1.0 and C_M = 2.0 the ratio $\hat{F}_D$ to $\hat{F}_I$ becomes:

$$0.083 \frac{H}{D} < \frac{\hat{F}_D}{\hat{F}_I} < 0.51 \frac{H}{D} \quad (5)$$

In a similar fashion the ratio of drag to inertia moments can be shown to be:

$$0.10 \frac{H}{D} < \frac{\hat{M}_D}{\hat{M}_I} < 0.51 \frac{H}{D} \quad (6)$$

The total wave force on the tower is $\hat{F}_{tot} \approx \sqrt{\hat{F}_I^{\,2} + \hat{F}_D^{\,2}} = \hat{F}_I \sqrt{1 + (\hat{F}_D / \hat{F}_I)^2}$ and similarly the total wave induced overturning moment is $\hat{M}_{tot} \approx \sqrt{\hat{M}_I^{\,2} + \hat{M}_D^{\,2}} = \hat{M}_I \sqrt{1 + (\hat{M}_D / \hat{M}_I)^2}$. Thus, the ratios of total force and moment to the inertia component are:

$$\begin{aligned}
\frac{\hat{F}_{tot}}{\hat{F}_I} &= \sqrt{1 + (\hat{F}_D / \hat{F}_I)^2} \\
\frac{\hat{M}_{tot}}{\hat{M}_I} &= \sqrt{1 + (\hat{M}_D / \hat{M}_I)^2}
\end{aligned} \quad (7)$$

This results in the following boundaries on total wave force and moment relative to the wave inertia force and moment:

H/D	$\hat{F}_{tot} / \hat{F}_I$	$\hat{M}_{tot} / \hat{M}_I$
1	$1.00 < \hat{F}_{tot} / \hat{F}_I < 1.12$	$1.00 < \hat{M}_{tot} / \hat{M}_I < 1.12$
2	$1.01 < \hat{F}_{tot} / \hat{F}_I < 1.43$	$1.02 < \hat{M}_{tot} / \hat{M}_I < 1.43$
3	$1.03 < \hat{F}_{tot} / \hat{F}_I < 1.83$	$1.04 < \hat{M}_{tot} / \hat{M}_I < 1.83$

This demonstrates that for monotower support structures of wind turbines the predominant extreme wave loading is due to inertia loading. In several cases the drag load may be ignored altogether for initial design purposes, while if this is considered too rough an approximation, the total load and moment can still be related to the inertia load by applying a multiplication factor. For lesser than extreme wave conditions the predominance of the inertia loading is only enhanced. This means that wave induced fatigue will be governed by linear inertial wave loading.

6 GRAPHIC REPRESENTATION OF WAVE LOADS

After this analytical contemplation the more ready-to-use version of the wave forces can be compiled. Graphs representing the inertia and the drag loads and moments, respectively, based on the wave period for different water depths from 5 to 30 m in 5 m intervals have been determined using equations (4).

Both load and moment amplitudes are plotted in a normalised form by eliminating the influence of the cylinder diameter and the wave amplitude, thus explicitly showing the influence of wave period (wave length) and water depth. The normalised loads and moments are of the following form:

$$\frac{\hat{F}_I}{D^2 H} \quad \text{and} \quad \frac{\hat{F}_D}{D H^2} \qquad (\text{kN/m}^3)$$

$$\frac{\hat{M}_I}{\hat{F}_I \cdot d} \quad \text{and} \quad \frac{\hat{M}_D}{\hat{F}_D \cdot d} \qquad (\text{non-dimensional})$$

The plots in figure 6 are made for hydrodynamic coefficients with values of C_m = 2.0 and C_d = 1.0, while the density of seawater is set to ρ = 1025 kg/m^3. Due to the simple proportionality of loads and moments with the hydrodynamic coefficients, the loads for other values than 2.0 and 1.0 are obtained by simple multiplication.

7 APPLICATION OF GRAPHS

The design of the support structure of an offshore wind turbine will start with the selection of a location. Wind and wave data have to be gathered from all available sources. Careful study of the data will return a number of design environmental conditions. For waves these conditions are usually represented by a wave height H and wave period T. By making an estimate of the tower diameter D the hydrodynamic force and accompanying moment can be determined from the graphs in figure 6 (or the underlying equations (4)) and combining the two components using equation (7).

Because offshore wind parks usually cover a significant area, water depth may vary together with wave height and period (should the environmental data have delivered these details). Finding the maximum forces can be done very fast by means of a spreadsheet calculation.

The advantage of the graphs is that the designer has a lot of direct feedback on the influence of changes in wave period and water depth. For example, it is directly seen that for a constant wave period the inertia force increases with water depth while the drag force decreases, thus reducing the influence of drag loading. For a constant water depth the inertia force decreases with wave period, but the drag force increases somewhat with a smaller gradient. This reflects the elongation of the elliptical orbits for longer waves, in which the horizontal velocity increases more rapidly than the horizontal acceleration ($u/\dot{u} = 1/\omega > 1.0$ as long as $T >$ 6.28s). The arms of both inertia and drag force naturally increase with water depth and decrease with wave period (wavelength), as shown in the moment graphs.

REFERENCES

[1] Vugts, J.H., *Handbook of Bottom Founded Structures*; Delft University of Technology

[2] Ippen, A.Z. (editor), *Estuary and Coastline Hydrodynamics;* Engineering Society Monographs, McGraw Hill Book Company, 1966; chapter 8.1.

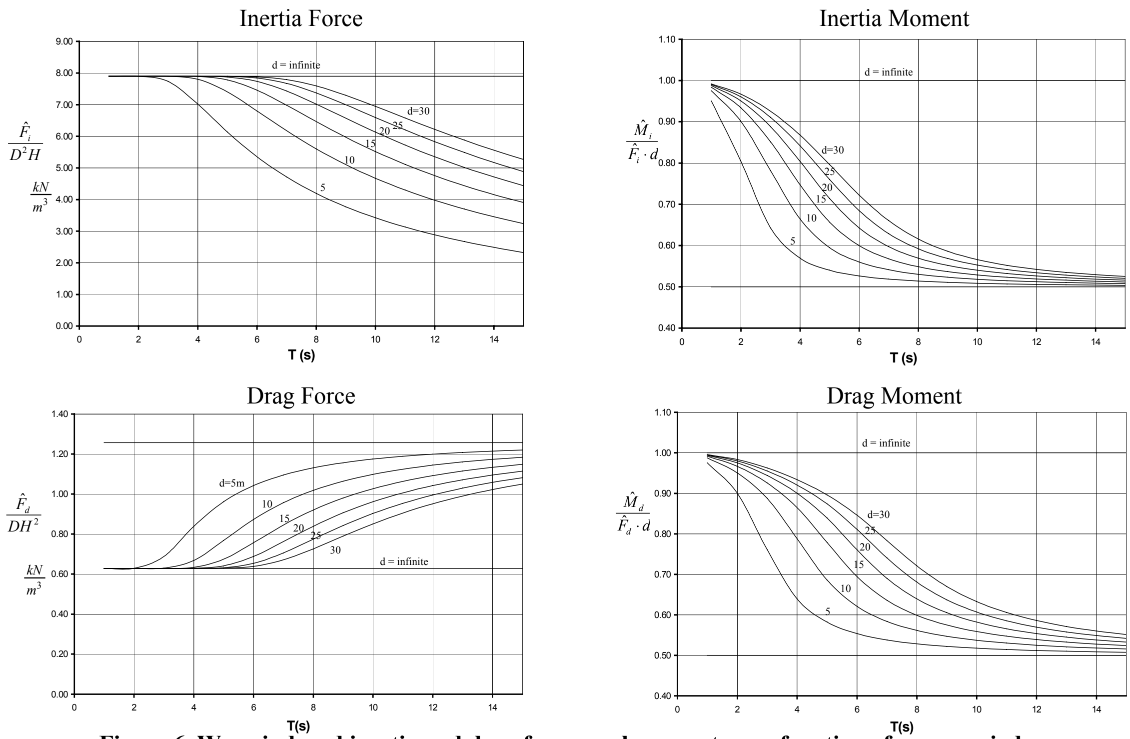

Figure 6. Wave induced inertia and drag forces and moments as a function of wave period