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Ultimate load analysis of a 10 MW offshore monopile wind turbine incorporating fully nonlinear irregular wave kinematics

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Abstract

Loads from storm waves can in some cases be dimensioning for offshore wind turbine substructures. Accurate determination of nonlinear wave loads is therefore important for a safe, yet economic design. In this paper, the fully nonlinear waves, realized by a fully nonlinear potential wave solver OceanWave3D, are incorporated into coupled aero-servo-hydro-elastic simulations for a reduced set of wave-sensitive design load cases, in comparison with the widely used linear and constrained waves. The coupled aero-elastic simulations are performed for the DTU 10MW reference wind turbine on a large monopile at 33 m water depth using the aero-elastic code HAWC2. Effect of the wave nonlinearity is investigated in terms of the ultimate sectional moments at tower bottom and monopile mudline. Higher ultimate moments, 5% at tower bottom and 13% at monopile mudline as maximum, are predicated when the nonlinear waves are used. It could be explained by the fact that the extreme nonlinear waves, that are close to the breaking limit, can induce resonant ringing-type responses, and hereby dominate the ultimate load responses. However, the constrained wave approach shows marginal difference compared to the standard linear wave approach. It can be concluded at least for the present configuration that the industry standard approaches (linear and constrained wave approach) underestimate the ultimate load responses on offshore wind turbines in severe sea states.

Keywords: ultimate load, fully nonlinear wave kinematics, offshore monopile wind turbine

1. Introduction

Offshore wind energy is growing quickly all around the world, especially in a number of European countries such as Denmark, Germany, the United Kingdom and the Netherlands. In order to make offshore wind power a cost effective solution, the design of the substructures must be optimized to be as cost effective as possible. One way to achieve this is to reduce
the model uncertainties in wave loads calculation, which are accounted for by using a design
safety factor in most engineering designs. For the offshore wind turbine (OWT) design, the
maximum load responses, namely ultimate design loads, must be assessed carefully to make
sure that the OWT is safe enough over its lifetime. The wave model uncertainties in the
ultimate load assessment can be reduced by applying an advanced nonlinear wave model,
which is believed to be more physically realistic for shallow and intermediate depth. Hereby
the implication for design and structural safety can be assessed.

In a realistic sea, the wave fields vary continuously over space and time in an irregular
manner. Hereby, a realistic sea can not be represented by a deterministic regular wave, and
it should be treated as a stochastic process. In the offshore wind industry, the engineering
practice is to generate an irregular wave realization by applying a Gaussian random process
within linear wave theory, based on the deep water practice from the offshore oil and gas
industry. This model is fairly accurate when the waves are not too high and steep in
deep water. Ultimate limit state (ULS) waves, extreme by nature, however, cannot be
reliably described by the linear wave theory. Moreover, wave kinematics and its associated
hydrodynamic loads on an OWT are likely to be underestimated in shallow water due to the
nonlinear wave effects. The extreme steep waves in a severe sea state can induce resonant
ringing-type responses on OWTs, which may be critical for ULS design loads [1]. In order
to overcome the shortcoming of running a long simulation to capture the extreme wave, a
constrained wave method is widely used for engineering practice, by embedding a "design
wave", for instance a NewWave [2, 3] or a large nonlinear stream function wave [4], into a
linear stochastic wave background.

The wave nonlinearity effect on OWT load responses has been extensively investigated
in recent years. Agarwal and Manuel [5] investigated the effect of second-order nonlinear
irregular waves on the 20-year long-term loads for an offshore monopile structure, and
observed higher loads when the nonlinear wave was used. The study was limited to two
governing environmental states. It was further studied by Natarajan [6] and around 25%
higher extreme overturning moments were obtained using the second-order wave. Besides,
more advanced fully nonlinear wave models have been studied by a number of researchers.
Marino et al. [7, 8, 9] presented a novel numerical procedure for simulating fully nonlinear
irregular waves and coupled it with aero-elastic simulations. The results showed that the
structural responses are greatly influenced by using the nonlinear waves. Bredmose et al.
[10] studied an experiment with a flexible pile subjected to steep and breaking irregular
waves, where the classical ringing response was observed and well reproduced by applying
a fully nonlinear potential flow solver. Schloer et al. [11] further incorporated the fully
nonlinear waves, realized by the same solver used in this study, into coupled aero-elastic cal-
culations on a 5 MW offshore monopile wind turbine, and found that the linear wave theory
is generally sufficient for the fatigue load assessments, while wave nonlinearity is important
in determining the ultimate design loads. However, only six wind speeds and associated sea
states were used in this study. Similar to the study performed by Schloer et al. [11], the
fully nonlinear potential flow solver OceanWave3D is used in this paper to calculate the fully
nonlinear wave kinematics, and the coupled simulations are performed for the DTU 10MW
reference wind turbine on a large monopile at 33 m water depth using the aero-elastic code
In summary, wave nonlinearity effects on the hydrodynamic loads and its associated structural response of OWTs have been found to be significant, especially in severe sea states and shallow waters. However, the previous studies were limited to either only few governing environmental states or absence of aerodynamic loads. Therefore it may be of concern whether these findings apply to a realistic 10 MW OWT design subject to different load cases with a combination of turbulent wind and irregular waves. Therefore, instead of focusing on only few governing conditions or absence of aerodynamic loads, this paper aims at investigating the influence of fully nonlinear waves on the ultimate design loads of a realistic 10 MW OWT benchmark case for a reduced set of design load cases (DLCs) required by the standard IEC 61400-3-1:2019 [12].

The selection of these load cases based on the existing literature. Morató et al. [13] pointed out DLC1.6 is the most onerous load case among the power production and parked load cases for the NREL 5 MW prototype turbine model, mounted on a monopile with a rigid foundation. Wang and Larsen [14] showed that the hydrodynamic loading in severe sea state is the design driver with respect to the ultimate bending moment at foundation mudline for a jacket foundation supporting the DTU 10 MW reference turbine. This paper considers both power production and parked situation with the presence of the normal sea states and severe sea states.

The paper starts with a description of the benchmark case including the OWT model, the selected design load cases and the site-specific metocean data. Afterwards, the irregular wave realizations and its coupling with aero-elastic simulations are presented. Following the description of the models, all results are presented with discussions on the effect of fully nonlinear irregular wave loading on the ultimate design loads. In the end, conclusions are given.

2. Benchmark case

2.1. Offshore wind turbine model

The DTU 10MW reference wind turbine [15], which is a conventional horizontal axis, three bladed and upwind type turbine on a tubular tower, supported by a large monopile at 33 m water depth is used in this paper, depicted in Figure 1. The original tower of the DTU 10MW onshore turbine is truncated for the offshore environment with an air gap of 18 m. It is considered to be representative for the present 10-MW wind turbine on the market. The operational rotor speed covers the range from 0.10 Hz to 0.16 Hz, which defines the first rotor harmonics 1P. In order to avoid structural resonance, the global natural frequency should be designed between 1P and 3P. The first global natural frequency is designed to be 0.21 Hz with a 7.5 m wide, 0.085 m thick monopile.

The foundation model is represented by a spring model including lateral $K_{uu}$, rotational $K_{\theta\theta}$ and cross coupling $K_{u\theta}$, illustrated in Figure 1. The shear force $F$ and bending moment
\( M \) can hereby be represented through a stiffness matrix as the following [17]:

\[
\begin{bmatrix}
F \\
M
\end{bmatrix} = \begin{bmatrix}
K_{uu} & K_{u\theta} \\
K_{u\theta} & K_{\theta\theta}
\end{bmatrix} \begin{bmatrix}
u \\
\theta
\end{bmatrix}
\] (1)

where \( u \) is the lateral displacement in the fore-aft or side-side direction and \( \theta \) is the tilt angle at the mudline.

For an embedded pile with large diameter, it is considered to behave in a rigid manner [18]. The stiffness formulae for a rigid pile in linear inhomogeneous soil, derived by Darvish-Alamouti et al. [17], is written as:

\[
\begin{bmatrix}
K_{uu} & K_{u\theta} \\
K_{u\theta} & K_{\theta\theta}
\end{bmatrix} = \begin{bmatrix}
\frac{1}{2}L_p^2n_h & -\frac{1}{3}L_p^3n_h \\
-\frac{1}{3}L_p^3n_h & \frac{1}{4}L_p^4n_h
\end{bmatrix}
\] (2)

where \( L_P \) is the embedded pile length and \( n_h \) is the coefficient of subgrade reaction, which is constant with depth. This formulae is consistent with the linear part for a \( p - y \) curve description of the soil reaction force. In this study, a 40 m long pile is assumed and \( n_h \) is considered as 5000 kN/m\(^3\) with an assumption of medium dense sand. As a result, the stiffness matrices are calculated as

\[
\begin{aligned}
K_{uu} &= 4 \text{ [GN/m]} \\
K_{u\theta} &= -107 \text{ [GN]} \\
K_{\theta\theta} &= 3200 \text{ [GNm]}
\end{aligned}
\] (3)

In addition, a damping factor is implemented proportional to the soil stiffness matrices to have a reasonable damping value for the whole structure around 6% logarithm decrement.
damping \cite{19}. The controller is based on a variable-speed pitch control strategy. The overall rotor performance in terms of pitch, rotational speed, thrust and power, simulated using the aero-elastic tool HAWC2 with step-wise steady wind, are shown in Figure 2. It is noticeable that the thrust peak force acting on the rotor is achieved at around the rated wind speed 11.4 m/s. Table 1 summarizes the key design properties of the DTU 10MW reference wind turbine mounted on the large monopile.

Table 1: Key design properties of the DTU 10MW reference wind turbine mounted on a monopile at \( h = 33 \) m (Nat.Freq means natural frequency and damping is given in the format of logarithmic decrement).

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rated power</td>
<td>10MW</td>
</tr>
<tr>
<td>Rated wind speed</td>
<td>11.4 m/s</td>
</tr>
<tr>
<td>Cut-in, cut-out speed</td>
<td>4 m/s, 26 m/s</td>
</tr>
<tr>
<td>Controller</td>
<td>Variable-speed pitch control</td>
</tr>
<tr>
<td>Rotor speed</td>
<td>6 rpm - 9.6 rpm</td>
</tr>
<tr>
<td>Rotor diameter</td>
<td>178.3 m</td>
</tr>
<tr>
<td>Hub height</td>
<td>119 m above mean sea level</td>
</tr>
<tr>
<td>Water depth</td>
<td>33 m</td>
</tr>
<tr>
<td>Embedded pile length</td>
<td>40 m</td>
</tr>
<tr>
<td>Monopile diameter</td>
<td>7.5 m</td>
</tr>
<tr>
<td>Monopile thickness</td>
<td>0.085 m</td>
</tr>
<tr>
<td>1(^{st}) Nat.Freq, Damping</td>
<td>0.21 Hz, 5.5%</td>
</tr>
</tbody>
</table>

Figure 2: Pitch and rotational speed curve within the operational wind speed (left) and the corresponding thrust and power curve (right).

2.2. Selected design load cases

As the focus of this study is on wave nonlinearity, only a reduced set of wave-sensitive DLCs, including both operating and parked conditions, are investigated based on the IEC
61400-3 offshore wind turbine design standard [12]. More specifically, DLC1.1, DLC1.6 and DLC6.1 are selected. Table 2 gives a brief summary of the investigated DLCs, where the design situation and its associated environmental conditions are specified.

Table 2: Summary of the investigated design load cases (NTM: normal turbulence model; $V_{in}$: cut-in wind speed; $V_{out}$: cut-out wind speed; NSS: normal sea state; $E[H_S | V_{hub}]$: expected significant wave height based on the mean wind speed; SSS: severe sea state; $H_{S,SSS}$: conditional severe significant wave height with a recurrence period of 50 years based on the mean wind speed; $V_{50}$, $H_{S,50}$: wind speed and significant wave height with 50-years recurrence period respectively).

<table>
<thead>
<tr>
<th>DLC</th>
<th>Design situation</th>
<th>Wind Model</th>
<th>Speed</th>
<th>Yaw</th>
<th>Wave Model</th>
<th>Height</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>Power production</td>
<td>NTM</td>
<td>$V_{in} : 2 : V_{out}$</td>
<td>$0^\circ$, $\pm 10^\circ$</td>
<td>NSS</td>
<td>$E[H_S</td>
<td>V_{hub}]$</td>
</tr>
<tr>
<td>1.6</td>
<td>Power production</td>
<td>NTM</td>
<td>$V_{in} : 2 : V_{out}$</td>
<td>$0^\circ$, $\pm 10^\circ$</td>
<td>SSS</td>
<td>$1.09 \cdot H_{S,SSS}$</td>
<td>$0^\circ$</td>
</tr>
<tr>
<td>6.1</td>
<td>Idling</td>
<td>NTM</td>
<td>$0.95 \cdot V_{50}$</td>
<td>$0^\circ$, $\pm 8^\circ$</td>
<td>SSS</td>
<td>$1.09 \cdot H_{S,50}$</td>
<td>$0^\circ$, $\pm 30^\circ$</td>
</tr>
</tbody>
</table>

Power production under normal operation within the cut-in and cut-out range of wind speed is considered in the DLC1.1 and DLC1.6, where normal sea state and severe sea state are used respectively. In addition, DLC6.1 considers idling wind turbine under extreme wind and wave conditions with a 50 years return period. To account for the stochastic nature of wind and waves, six seeds are used for each combination of wind, wave and operation condition, resulting in 486 simulations for the selected DLCs. In terms of the simulation length involving severe sea states under DLC1.6 and DLC6.1, 1 hour simulation length is used. All simulations for DLC1.1 are performed for only 10 minutes.

2.3. Reference site metocean data

The reference site is located at German Bight in the North Sea with a water depth around 33 m, which is considered to be a realistic and reasonable OWT location with available metocean data provided by DHI. The operational wind speed goes from 4 m/s to 26 m/s with a 2 m/s wind speed bin, and the associated turbulence intensity, significant wave height and peak wave period, in accordance with the normal sea states and severe sea states, are listed in Table 3. The extreme wind speed, the significant wave height and peak wave period with a 50 years recurrence period are listed in the last row of Table 3 as well.
Table 3: Site-specific metocean data used for the investigated DLCs (\textit{wsp}: mean wind speed; \textit{TI}: turbulence intensity; \textit{H}_S: significant wave height; \textit{T}_P: peak wave period).

<table>
<thead>
<tr>
<th>(wsp) [m/s]</th>
<th>(TI) [-]</th>
<th>Normal Sea State</th>
<th>Severe Sea State</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(H_S) [m]</td>
<td>(T_P) [s]</td>
<td>(H_S) [m]</td>
</tr>
<tr>
<td>4</td>
<td>0.041</td>
<td>0.7</td>
<td>0.7</td>
</tr>
<tr>
<td>6</td>
<td>0.044</td>
<td>0.8</td>
<td>0.9</td>
</tr>
<tr>
<td>8</td>
<td>0.047</td>
<td>0.9</td>
<td>1.2</td>
</tr>
<tr>
<td>10</td>
<td>0.050</td>
<td>1.6</td>
<td>1.2</td>
</tr>
<tr>
<td>12</td>
<td>0.053</td>
<td>2.0</td>
<td>1.6</td>
</tr>
<tr>
<td>14</td>
<td>0.056</td>
<td>2.4</td>
<td>2.0</td>
</tr>
<tr>
<td>16</td>
<td>0.059</td>
<td>2.9</td>
<td>2.4</td>
</tr>
<tr>
<td>18</td>
<td>0.062</td>
<td>3.5</td>
<td>2.9</td>
</tr>
<tr>
<td>20</td>
<td>0.065</td>
<td>3.8</td>
<td>3.5</td>
</tr>
<tr>
<td>22</td>
<td>0.068</td>
<td>4.2</td>
<td>3.8</td>
</tr>
<tr>
<td>24</td>
<td>0.071</td>
<td>4.2</td>
<td>4.2</td>
</tr>
<tr>
<td>26</td>
<td>0.074</td>
<td>5.1</td>
<td>5.1</td>
</tr>
<tr>
<td>45.8</td>
<td>0.100</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Le Méhauté [20] established a useful diagram for understanding the applicability of a variety of classical wave theories, based on the relation between water depth parameter $\frac{h}{g\tau^2}$ and wave steepness parameter $\frac{H}{g\tau^2}$, shown in Figure 3. Although the diagram is originally made for regular waves, we mark the present sea states in terms of $H_S$ and $T_P$ to get an indication of the wave region we see that the waves are in the intermediate depth region and with some steepness, depicted as dots. In linear theory, the wave height of the largest in a 3 hours sea state would be approximately $1.86H_S$ and thus place the largest waves in the left-most states close to the breaking limit.

![Figure 3: The site-specific sea states presented on the wave theory diagram established by Le Méhauté [20]. The significant wave height $H_S$ and peak wave period $T_P$ are used instead of $H$ and $\tau$. $h$ is the water depth.](image)

3. Irregular wave realizations and aero-elastic simulations

3.1. Linear irregular wave realization

Stochastic ocean waves represented by a linear irregular wave model are widely used in aero-elastic simulations for the dynamic analysis of OWTs. A linear irregular wave is commonly represented by a wave spectra, for instance the widely used JONSWAP spectrum [21], as defined by the following equations:

$$S(\omega) = (1 - 0.287 \ln(\gamma)) \cdot \frac{5}{16} H_S^2 \omega_p^4 \omega^{-5} \exp\left(-\frac{\omega^4}{\omega_p^4}\right) \gamma^a$$  \hspace{1cm} (4)

$$a = \exp\left(-\frac{(\omega - \omega_p)^2}{2\omega_p^2\sigma^2}\right)$$ \hspace{1cm} (5)
\[ \sigma = \begin{cases} 0.07 & \omega \leq \omega_p \\ 0.09 & \omega > \omega_p \end{cases} \]  

(6)

where \( \beta = \frac{5}{4} \), \( \gamma = 3.3 \), \( \omega \) is the wave frequency and \( \omega_p \) is the peak wave frequency.

The stochastic wave series \( \eta(t) \) for the wave spectrum can be generated using the following linear superposition:

\[ \eta(t) = \sum_i A_i \cos(\omega_i t + \phi_i) \]  

(7)

\[ A_i = \sqrt{2S(\omega_i)} \Delta \omega \]  

(8)

\[ \phi_i = \text{rand}(0, 2\pi) \]  

(9)

where \( A_i \) is the \( i \)th wave amplitude, \( \omega_i \) is the \( i \)th wave frequency, \( \Delta \omega \) is the wave frequency bandwidth and \( \phi_i \) is the \( i \)th random wave phase. In this study we thus use the random phase approach, but keep the amplitude deterministic.

### 3.2. Constrained irregular wave realization

In order to capture the ULS wave, namely the design wave, in a stochastic irregular wave series without performing many hours of random time domain simulation, constrained wave methods can be used according to IEC standard [12]. Cassidy et al. [3] used a method called constrained NewWave methodology to embed a NewWave [2] with a pre-determined large crest into an arbitrary random wave series. Rainey et al. [4] used a suitably large stream function wave instead of a NewWave to represent the extreme wave in a linear, stochastic wave background. It is considered to be a combination of wave nonlinearity with the design wave approach, hereby the method developed by Rainey et al. [4] is used in this study. To prevent any discontinuities in the constrained waves, short blending regions are used where all the water properties are calculated as a weighted average of the embedded stream function wave solution and the linear irregular wave. The extreme wave height \( H_{\text{stream}} \) in a 3 hours stationary sea state \( H_S \) is calculated as \( H_{\text{stream}} = 1.86 \cdot H_S \) by assuming a Rayleigh distribution of the wave elevation [12]. In addition, the wave period for the embedded stream function wave \( T_{\text{stream}} \) can be taken within the range between \( 11.1 \sqrt{\frac{H_S}{g}} \) and \( 14.3 \sqrt{\frac{H_S}{g}} \), according to [12]. For each sea state, different values of \( T_{\text{stream}} \) were evaluated in the full dynamic model and the shortest \( T_{\text{stream}} \), corresponding to \( 11.1 \sqrt{\frac{H_S}{g}} \), always led to the most conservative results in terms of the dynamic response. This paper presents the results from the shortest embedded wave period. Figure 4 shows an example of the constrained wave by embedding an extreme stream function wave on a linear irregular wave background.
3.3. Nonlinear irregular wave realization

The nonlinear waves are calculated using a validated fully nonlinear potential flow solver OceanWave3D, developed by Engsig-Karup et al. [22]. It solves the 3D Laplace equation for the velocity potential with nonlinear boundary conditions at the free surface and the sea bed. Hereby, this model is believed to be more physically realistic, in comparison with the linear and constrained wave. The accuracy has been validated extensively using the experimental data [10, 23, 24, 25, 26, 27, 28, 29, 30]. To overcome an issue that potential flow solution does not model actual breaking and therefore waves may become unreasonably steep, a breaking filter is applied based on the rate of vertical water particle velocity $\frac{dw}{dt}$. A threshold of $\frac{dw}{dt} < -\beta g$ ($\beta = 0.5$) is applied in this study. If the value is exceeded, local dissipation is introduced to represent the effect of wave breaking.

A representative 11500 m long seabed profile is used in this paper with a slope around 1:100 between water depth $h = 100$ m and $h = 30$ m, depicted in Figure 5. The linear irregular waves from a Jonswap spectrum are generated at 100 m water depth within a 1000 m wave generation zone and propagate uni-directionally to simplify the problem to two dimensional. At the end of the fluid domain, a wave relaxation zone is defined where the waves are damped out numerically. The length of the wave relaxation zone is identical to the wave generation zone. In Figure 5, the OWT location is shown as a black dot at 33 m water depth.

Based on the convergence study performed by Schlöer et al. [11], 12 to 15 points in the vertical direction and grid spacing of 0.78 m in the horizontal direction are considered acceptable to get a converged solution from OceanWave3D. The energy spectrum shown in the sea states is approximately bounded between 0.05-0.40 Hz. A shortest wave component with the frequency of 0.40 Hz has the wave length of $L = 9.8$ m at 33 m water depth based...
on the linear dispersion relation, and therefore the minimum grid spacing is set as 0.75 m in the $x$-direction. Besides, 12 points are resolved underneath the wave surface elevation. This is consistent with the choice of Schløer et al. [11] for waves of $T_P \geq 7.66$ s, while 15 points were used for smaller values of $T_P$. We chose to apply 12 points in all computations, since our focus is the ULS loads caused by the large period waves.

The transition time for wave traveling from the wave generation zone to the investigated location is calculated using the travel distance and wave group velocity. A slowest wave with the frequency of 0.40 Hz travels at a group velocity of $V_g = 1.95$ m/s. Therefore it takes approximate 5300 s to reach the location at 33 m water depth. In order to make sure all interested wave components are captured at the investigated location, a transition time is set as 6400 s, shown in Figure 6. The total simulation time is 10000 s for each sea state to obtain a 1-hour useful wave time series for each sea state and 6 seeds are used to generate 6 random fully nonlinear irregular waves for each case.

3.4. Aero-elastic simulations

The DTU Wind Energy developed aero-elastic code HAWC2 [31, 32] is used to perform all aero-hydro-elastic simulations. The stochastic wind field is modeled applying the Mann turbulence box. The aerodynamic loads on the wind turbine are calculated by the unsteady blade element momentum (BEM) theory with further consideration of dynamic inflow, skew inflow, shear effect on induction, effect from large blade deflections and tip loss [33]. A detailed description of the aerodynamic model is not given in this paper.

With respect to the focused hydrodynamic force, it is calculated based on the extensively used Morison equation given the undisturbed wave kinematics. The Morison force is calculated as a summation of two force components: an inertia force in phase with the local flow acceleration and a drag force proportional to the square of the instantaneous flow.
velocity. The formulation of the inline force, considering the structural vibration, is written as:

\[ f = \frac{1}{4} \rho \pi D^2 \ddot{u} + \frac{1}{4} C_a \rho \pi D^2 \dot{u}_{rel} + \frac{1}{2} C_d \rho D u_{rel}|u_{rel}| \]  

(10)

where \( u_{rel} \) represents the relative water particle velocity, while \( \dot{u} \) and \( \dot{u}_{rel} \) represent the associated undisturbed and relative acceleration, respectively. For the relative acceleration, \( \dot{u}_{rel} = \dot{u} - \ddot{x} \), the structural acceleration \( \ddot{x} \) is included into the equations of motion as added mass. Furthermore, \( \rho \) is the water density and \( D \) is the member diameter. A single set of empirical drag and added mass coefficient are denoted as \( C_d \) and \( C_a \). Their values are, in general, functions of the Reynolds number, the Keulegan-Carpenter number and the relative roughness. For the monopile, the inertial force is normally dominant over the drag force. The value of added mass coefficient 1.0 is used in the simulations. Additionally, the value of \( C_d \) is also chosen as 1.0 according to the DNV GL standard [34] considering a rough structure due to corrosion and marine growth. Except the widely used Morison equation, the Rainey equation can be used as an alternative load model [11]. It is not included in HAWC2 at the time of the present study, and is therefore not applied here. The nonlinearities in wave kinematics are identified more important than the nonlinearities in the hydrodynamic loading model, especially at the intermediate to shallow water depth, hence the conclusion from this study would be still valid even if the Rainey equation were used [35, 30].

4. Results of wave surface elevation and associated wave forcing

Before discussing the load response of the OWT, the key aspects related to the wave surface elevations and their associated wave forcing on the rigid monopile are presented and discussed, focusing on the linear and nonlinear waves.
4.1. Wave spectra

The distribution of wave energy is important to determine the structural response. Figure 7 provides the amplitude spectral density for three representative sea states. Compared to the spectrum of linear waves, nonlinear waves show a secondary energy content close to zero frequency. Such phenomenon is more pronounced for the most severe sea state, in connection with a stronger interaction of frequencies in the nonlinear wave process. Although the wave nonlinearity will lead to second- and higher-harmonic content for the individual steep waves, this does not show up in the spectra of the two smaller sea states. For the largest sea state, a second harmonic spectral peak at $f = 0.15$ Hz is visible.

![Figure 7: Amplitude spectral density of the linear and nonlinear waves, corresponding to three representative sea states.](image)

4.2. Statistics of wave surface elevations

For the description of a non-Gaussian process, skewness has a key role representing to which degree the process is non-Gaussian. Hence, wave skewness were calculated for both linear and nonlinear waves, summarized in Table 4. The skewness is around 0 for linear waves, indicating a Gaussian nature of these linear waves. Whereas, the nonlinear waves have a positive skewness, hence these nonlinear waves are non-Gaussian. The positive skewness is a result of the sharper crests and flatter troughs associated with these nonlinear waves. In addition, higher skewness is presented for more severe sea states, relating to stronger nonlinearity.

The description of extreme waves and their associated exceedance probabilities represents an alternative key parameter for the design of OWTs [36]. Given a linear irregular wave realization, the normalized crest heights, $\eta_c/H_S$ will be Rayleigh distributed, written as:

$$P(\eta_c > \eta) = \exp\left(-8\left(\frac{\eta}{H_S}\right)^2\right)$$

Figure 8 presents the probability of exceedance of the normalized crest height, $\eta/H_S$, for all used sea states. The crest heights were identified using zero-crossing method and
Table 4: Wave skewness for all investigated sea states.

<table>
<thead>
<tr>
<th>$H_S$ [m]</th>
<th>$T_P$ [s]</th>
<th>Wave Skewness</th>
<th>$H_S$ [m]</th>
<th>$T_P$ [s]</th>
<th>Wave Skewness</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Linear</td>
<td>Nonlinear</td>
<td></td>
<td>Linear</td>
</tr>
<tr>
<td>0.7</td>
<td>7.6</td>
<td>-0.03</td>
<td>0.05</td>
<td>2.5</td>
<td>8.7</td>
</tr>
<tr>
<td>0.8</td>
<td>6.5</td>
<td>0.00</td>
<td>0.07</td>
<td>2.8</td>
<td>8.6</td>
</tr>
<tr>
<td>0.9</td>
<td>6.3</td>
<td>0.00</td>
<td>0.07</td>
<td>3.3</td>
<td>8.8</td>
</tr>
<tr>
<td>1.2</td>
<td>6.2</td>
<td>0.00</td>
<td>0.08</td>
<td>3.9</td>
<td>9.2</td>
</tr>
<tr>
<td>1.6</td>
<td>6.5</td>
<td>-0.01</td>
<td>0.10</td>
<td>4.5</td>
<td>9.7</td>
</tr>
<tr>
<td>2.0</td>
<td>6.7</td>
<td>-0.03</td>
<td>0.12</td>
<td>5.3</td>
<td>10.3</td>
</tr>
<tr>
<td>2.4</td>
<td>7.1</td>
<td>-0.04</td>
<td>0.15</td>
<td>6.0</td>
<td>10.9</td>
</tr>
<tr>
<td>2.9</td>
<td>8.0</td>
<td>0.03</td>
<td>0.13</td>
<td>6.8</td>
<td>11.6</td>
</tr>
<tr>
<td>3.5</td>
<td>8.5</td>
<td>-0.03</td>
<td>0.15</td>
<td>7.5</td>
<td>12.2</td>
</tr>
<tr>
<td>3.8</td>
<td>8.7</td>
<td>-0.04</td>
<td>0.13</td>
<td>8.1</td>
<td>12.6</td>
</tr>
<tr>
<td>4.2</td>
<td>8.8</td>
<td>0.03</td>
<td>0.16</td>
<td>8.6</td>
<td>12.9</td>
</tr>
<tr>
<td>5.1</td>
<td>9.6</td>
<td>0.06</td>
<td>0.18</td>
<td>9.0</td>
<td>13.2</td>
</tr>
<tr>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>9.9</td>
<td>13.8</td>
</tr>
</tbody>
</table>

Six individual realizations for each sea state were merged and re-ordered to a very small exceedance probability. As expected, significant variability is shown in the distributions, especially in the extreme tail part corresponding to a low exceedance probability. More importantly, systematic deviations form the Rayleigh distribution present in the nonlinear waves. Comparisons between the linear and nonlinear waves highlight that the nonlinear waves exhibit higher wave crests, which is more pronounced for severe sea states. This is consistent with the higher skewness associated with these severe sea states.

Figure 8: Probability of exceedance of the normalized crest heights, $\eta/H_S$, for all investigated sea states, in comparison with Rayleigh distribution.
4.3. Wave forcing on the rigid monopile

While the extreme wave is normally considered as a design driver for the design, the largest wave crest height is not the only determining parameter leading to the largest wave force. Instead, the largest force could be introduced by a wave with a small wave height and amplitude but with a very steep wave front, especially if the wave force is inertia-dominated [37]. For the dynamic response of an OWT, accurate prediction of the extreme wave force would be more important instead of the extreme wave crest height. The wave forcing on the rigid monopile is calculated using the Morison equation, neglecting the influence from structural dynamic properties. Consider a monopile with a radius of $R$ at water depth $h$, the peak force $F$ and peak moment $M$ could be normalized by the significant wave height $H_S$ as $F/\rho g R^2 H_S$ and $M/\rho g R^2 H_S h$ to make it aligned with $\eta/H_S$ [38]. Figure 9 shows the exceedance probability of the normalized peak forces and moments for all sea states in Table 3. The peak forces induced by nonlinear waves are larger than using linear waves, and the difference is more significant in terms of the peak moments.

![Figure 9: Probability of exceedance of the normalized peak forces and moments, with respect to $F/\rho g R^2 H_S$ and $M/\rho g R^2 H_S h$.](image)

5. Results of OWT load response

In this section, the dynamic response of the reference OWT subject to simultaneous turbulent wind and irregular waves simulated by the aero-elastic code HAWC2 are presented.
The results on two key global design parameters, the bending moments at the tower bottom and monopile mudline, will be discussed for the investigated DLCs. The resultant bending moment can be calculated from the fore-aft moment $M_x$ and the side-side moment $M_y$ using the relation $M = \sqrt{M_x^2 + M_y^2}$.

5.1. DLC1.1: power production in normal sea state

The power production situation is the most important operating condition of an OWT, where the turbine is producing electricity to the grid within the cut-in and cut-out wind speed range. Considering the wind turbine dynamic response, significant aerodynamic loads and damping need to be taken into account. In terms of DLC1.1, the normal turbulence model (NTM) is used together with the normal sea state (NSS) to represent the environmental condition for the turbine. In addition, three yaw misalignment angles, namely $0^\circ$ and $\pm 10^\circ$, are simulated to account for the possible delay of the yaw controller. For each combination of wind speed bin and yaw misalignment angle, six 10-minutes simulations were performed, which resulted in 216 simulations in total. Although the constrained wave approach is not usually applied within this load case, we include it here to allow comparison with the linear and fully nonlinear wave results.

The maximum values of the bending moment at the tower bottom and monopile mudline are plotted in Figure 10 over the wind speed bins. A similar load pattern between the bending moments and the thrust force is obtained and the largest moments at the tower bottom and monopile mudline are reached at the rated wind speed and is associated with the nonlinear waves. It can be explained by the fact that the aerodynamic forces on the rotor have a dominating influence on the bending moments with a larger lever arm, compared to a relatively small lever arm for hydrodynamic forces. In general, though, the results from nonlinear waves do not generate significantly larger moments than the other two wave approaches.

Figure 10: Maximum values of the bending moment at the tower bottom (left), monopile mudline (middle) and the aerodynamic thrust force on the rotor (right) grouped into wind speed bins under DLC1.1. For each wind speed bin, 18 simulations were performed using 6 seeds together with 3 yaw misalignment angles.

Figure 11 shows a time series segment of the overall dynamic response of the wind turbine, corresponding to an extreme operation case at the wind speed of 12 m/s. Due to the turbulent wind, the thrust force varies greatly with the variable-speed pitch controller.
The variance of thrust force is also presented in the time series of the bending moment at the tower bottom and monopile mudline. The hydrodynamic force caused by the extreme wave can be seen at the monopile mudline moment, at the time around 210 s to 260 s. However, this load contribution is overshadowed by the trough of the thrust force. In fact, the contributions from wind and wave forces, especially on the substructure, are not separable in a given response time series, and a phase shift normally exists between the wind peak force and the wave peak force resulting in a rare possibility that the largest wind and wave loads occur simultaneously [39]. In the power production situation with normal sea states, the synchronized behavior between the thrust force and the response bending moments provides a strong evidence that the aerodynamic force is the governing force for the OWT dynamic response. Limited difference is observed for the tower bottom moment when different wave model is used, indicating that the structural parts above the wave zone do not feel the existence of wave fields in a mild sea state. In addition, no pronounced structural vibration is observed due to the large aerodynamic damping introduced by the operating wind turbine. The results indicate that for wind turbine operating in normal sea states, wave nonlinearity effect is insignificant and it would be sufficient to use linear waves.

![Figure 11: Time series of the wind speed, thrust force, wave surface elevation, tower bottom moment and monopile mudline moment (from top to bottom), corresponding to the linear, constrained and nonlinear waves for the operation condition of 12 m/s wind speed, 1.6 m significant wave height and 6.5 s peak wave period.](image)
5.2. DLC1.6: Power Production in Severe Sea State

Similar to DLC1.1, DLC1.6 also simulates the operating wind turbine within the cut-in and cut-out wind speed range, whereas the severe sea states are used to represent the stochastic wave field instead of normal sea states. Hence, 216 simulations were performed as well, corresponding to 12 wind speed bins, 3 yaw misalignment angles and 6 random seeds. Following IEC standard [12], a 1-hour duration was applied for each seed. The maximum values of the bending moment at the tower bottom and monopile mudline are shown in Figure 12. Although the largest moments are still greatly dominated by aerodynamic force when the linear and constrained wave are used, application of the nonlinear wave significantly changes the load response above rated wind speed where significantly higher loads are obtained. In terms of the tower bottom moment, although the extreme value is still obtained at the rated wind speed with the extreme aerodynamic force, considerably large moments are achieved at the cut-out wind speed of 26 m/s with the use of nonlinear waves. The dominating load contribution shifts from aerodynamics to hydrodynamics when the nonlinear wave is used, and it is more distinct for monopile mudline moment. The extreme monopile mudline moment is obtained at the cut-out wind speed.

![Figure 12: Maximum values of the bending moment at the tower bottom (left) and monopile mudline (right) grouped into wind speed bins under DLC1.6. For each wind speed bin, 18 simulations were performed using 6 seeds together with 3 yaw misalignment angles.](image)

A further insight into an extreme load response time series at tower bottom introduced by the occurrence of an extreme wave is shown in Figure 13. The maximum crest heights are aligned to appear at the same time for a straightforward comparison. While almost identical crest height is achieved between the constrained wave and nonlinear wave, a much steeper wave front is observed in the nonlinear wave time series. This steep wave front in the nonlinear wave eventually results in much larger response on the wind turbine, shown in the time series of bending moment at the tower bottom. Strong structural vibration is presented at the tower bottom with the use of the nonlinear wave, triggered by the passage of an extremely steep wave. Continuous wavelet transformations are performed to localize the response in time and frequency domain, also shown in Figure 13. The color scale is
normalized by the nonlinear wave case, with high energy content indicated by warm colors. The pronounced resonance triggered by the passage of the extreme wave in the nonlinear wave is found to be excited at its first mode. The first mode resonance phenomenon is widely known as ringing-type response [11 40]. It can be concluded that the hydrodynamic forces, compared to the aerodynamic forces, become more important and dominating for an operating wind turbine in a severe sea state when nonlinear waves are used. Hereby, load response of an operating wind turbine may be significantly underestimated by application of the linear or constrained waves, caused by the fact that the structural resonance is not accurately captured.

Figure 13: Time series and its associated wavelet spectrum of the fore-aft tower bottom bending moment for a local extreme wave event, in connection with the operation conditions of 26 m/s wind speed, 9.8 m significant wave height and 13.2 s peak wave period. The wave surface elevations are plotted as colorful lines.

5.3. DLC6.1: idling in severe sea state

In the extreme weather condition when the wind speed exceeds the operational wind speed, the rotor is either in a standstill or idling condition. In order to simulate this situation, the blades are pitched to feather without activation of the pitch controller and generator. The extreme wind and wave condition with a 50 year return period is used to represent the extreme environmental condition. In order to account for the yaw misalignment and wind-wave misalignment, 3 yaw angles, corresponding to 0° and ± 10°, and three wave misalignment angles as 0° and ± 30° are considered in the 54 1-hour simulations with 6 random seeds.

When the rotor is idling, the aerodynamic forces and damping are negligible so that the dynamic response of the wind turbine is strongly influenced by the wave forcing. Figure 14 shows a typical amplitude spectrum of the wave surface elevation and its associated fore-aft bending moment at the monopile mudline with a highlight of the dynamic amplification at the structural natural frequency around 0.21 Hz. The dynamic amplification is more
pronounced with the use of nonlinear waves. On top of the dynamic amplification, the moment at the monopile mudline also contains the quasi-static response to the wave forcing.

Figure 14: A typical amplitude spectral density of the wave surface elevation (left) and the fore-aft bending moment at monopile mudline (right), corresponding to the idling wind turbine in the severe sea state under DLC6.1.

The governing resonances are related to the so-called springing- and ringing-type response. The difference between springing and ringing response is difficult to identify because both responses occur close to the first natural frequency of the structure [11]. Here, we characterize ringing response as a transit event generally triggered by a high, steep wave, while springing response is characterized as a steady-state response [1]. Figure 15 provides a selected extreme wave event in the constrained and nonlinear wave, associated with the load response at the tower bottom. The underlying steady-state responses across both wave series are characterized as springing-type response, while a more evident ringing-type response is only triggered by the extreme wave in the nonlinear wave at the time around 1530 s. Less impulsive responses are observed for the constrained wave, although it can also be considered as a ringing-type response. The results clearly demonstrate the importance of applying nonlinear waves into ultimate load response analysis for an idling turbine.

To further understand how waves influence the OWT response in idling condition, the correlation of the maximum monopile bending moment and the wave height is investigated by a zero up-crossing analysis, where all the individual waves are detected. The maximum bending moment at monopile mudline is plotted against the wave steepness $H/L$ and the depth parameter $h/L$, shown in Figure 16. Wave length is calculated based on the linear wave dispersion relation. In Figure 16, the black line denotes a breaking criteria. The results show that no matter which wave model is used, the largest moments at the monopile mudline occur for the steep waves that are close to the breaking limit. This research finding is consistent with Bredmose et al. [10]. No distinct difference can be identified between the linear wave and constrained wave approach, as expected, while the use of nonlinear wave results in larger bending moment at the monopile mudline for the individual waves.
that are close to the breaking limit. It can be concluded that a "dimensioning wave" which determines the ultimate design loads on OWTs can not be established only based on a certain wave height. The wave shape including the wave steepness is also crucial for its definition.

Figure 15: Time series of the fore-aft bending moment at the tower bottom, in connection with the occurrence of the extreme wave in the constrained wave (top) and nonlinear wave (bottom).

Figure 16: Correlation of the maximum bending moment at the monopile mudline with each individual wave steepness and the depth parameter. The black line shows a breaking limit, and the color scale shows the magnitude of bending moment in MNm.
5.4. Ultimate design load analysis

Following the detailed load and response analysis, we now present the characteristic loads comparison for ULS design for each design load case. The ultimate characteristic loads were calculated as the average value over the worst case (with 6 seeds) identified from the simulated response time-series. Figure 17 shows the obtained characteristic values for the resultant moment at the tower bottom and monopile mudline. It is observed that DLC6.1 results in the highest characteristic values, and can therefore be considered as the design driver for both tower bottom and monopile mudline in this case. In addition, the difference of these three DLCs is less distinct for the tower bottom characteristic moment, compared to the difference for the monopile mudline. This is due to the fact that the wave field can only impact the tower indirectly and the aerodynamic force is normally more important. Furthermore, larger characteristic loads were predicted when nonlinear waves were used, with the highest difference being about 13% for DLC6.1. The commonly used constrained wave in the industry does not lead to strong differences relative to the linear wave theory loads for the present case of a monopile supported 10 MW wind turbine at 33 m water depth. This might be explained by the fact that the global hydrodynamic forces are inertia-dominated and are thus likely sensitive to the local wave steepness, while the constrained wave is based on a certain extreme wave height but has no asymmetry. The nonlinear wave induced moment, however, is visibly stronger than the linear wave results. We explain this by a more pronounced steepness for the design-driving waves in this model.

Figure 17: Ultimate bending moment at the tower bottom and monopile mudline for the investigated design load cases with the use of linear, constrained and nonlinear wave model.

Table 5 lists all the characteristic values shown in Figure 5. With respect to the partial safety factor suggested by the standards [41, 12], using a more accurate and realistic wave model, the fully nonlinear wave, indicates some conservatism in current design. In terms of the tower design, the partial safety factor should be mainly determined by the reliability in the aerodynamics modeling. One can see here that a 5% higher value, as maximum, was predicted using the nonlinear wave model. In terms of the monopile design, the hydrody-
namic load effect contribution is a design driver. 13% higher value was predicated when the nonlinear wave was used in this study.

Table 5: Ultimate characteristic bending moment at the tower bottom and monopile mudline for the investigated DLCs (PSF means partial safety factor, NL means nonlinear and L means linear).

<table>
<thead>
<tr>
<th>DLC</th>
<th>PSF</th>
<th>Moment at tower bottom [MNm]</th>
<th>Moment at monopile mudline [MNm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Linear</td>
<td>Constrained</td>
</tr>
<tr>
<td>1.1</td>
<td>1.35</td>
<td>171.4</td>
<td>171.7</td>
</tr>
<tr>
<td>1.6</td>
<td>1.35</td>
<td>185.0</td>
<td>186.3</td>
</tr>
<tr>
<td>6.1</td>
<td>1.35</td>
<td>188.4</td>
<td>191.4</td>
</tr>
</tbody>
</table>

6. Conclusions

Accurate predication of the ultimate design loads on an OWT is a challenging problem which requires proper modeling of the various load sources. With this in mind, the objective of this study was to improve the accuracy of calculating hydrodynamics load on an offshore wind monopile structure, by incorporating fully nonlinear irregular waves that are more suitable for relatively shallow waters. The fully nonlinear irregular waves were realized by a potential flow solver OceanWave3D, developed by Engsig-Karup et al. [22]. Aeroelastic simulations for the DTU 10MW wind turbine mounted on a monopile at the water depth of 33 m were performed for a reduced set of wave-sensitive DLCs, including both operation and parked conditions, with the use of linear, constrained and nonlinear waves.

The comparisons between the linear and nonlinear waves showed that the nonlinear waves exhibited higher wave crests, more pronounced for the severe sea states due to the fact that nonlinear waves tend to be more non-Gaussian. As a result, the extreme hydrodynamic force introduced by the nonlinear waves were larger than the results with the use of linear waves.

The influence of the nonlinear waves on the wind turbine response was further investigated focusing on the bending moment at the tower bottom and monopile mudline. In terms of the power production situation with normal sea states, the results showed that the aerodynamic force dominated the wind turbine dynamic response, and the wind turbine can not "feel" the existence of the wave field. The observation was also reported in the existing literature [12]. Hereby, the influence of wave modeling was very limited, and the rated wind speed with its associated wave condition was the most critical environmental condition in DLC1.1. A different scenario was observed for the design situation of the operating turbine with severe sea states. Application of the nonlinear wave significantly changed the load response from the aerodynamic-dominated regime to the hydrodynamic-dominated regime. It was found that ringing type response at the first eigenfrequency of the structure was triggered by the extreme waves in the nonlinear wave series when the aerodynamic force and damping were relatively insignificant at the cut-out wind speed.
Furthermore, the parked situation with severe sea states, namely DLC6.1, was shown to be the most critical design load case. The load response at the tower bottom was mainly classified into springing- and ringing-type response. A significant underlying springing-type was presented across all of the different wave model, while a more evident ringing-type response was only triggered by the extreme, steep waves in the nonlinear wave realizations. The ringing-type response had an important contribution to the higher ultimate design loads at the tower bottom and monopile mudline when the nonlinear wave was used. The results clearly demonstrated the importance of using the nonlinear waves for a parked turbine in order to determine the accurate ultimate design loads.

No distinct difference was observed for the bending moment at the tower bottom and monopile mudline using the linear and constrained waves. Hereby, the embedded stream function wave with a certain large wave height is not considered as a suitable "design wave" determining the ultimate design loads. It was found that higher ultimate moments, 5% at the tower bottom and 13% at the monopile mudline as maximum, were obtained when the nonlinear wave was used. In most engineering design, the uncertainties in a deterministic model prediction is accounted for by using safety factors. A discussion on the possible modification of the design safety factor needs further study, and this work is left for the future.

Due to lack of the field measurements data from a real offshore wind turbine, this study is limited to numerical evaluation although the nonlinear wave solver has been extensively validated in the wave tank experiments. While the applicability of nonlinear wave kinematics is usually limited by computational demands, a database of precomputed kinematics is under production in the Derisk project [43]. Hereby the present approach of long irregular fully nonlinear wave kinematics will become available for use in practical engineering.

Acknowledgements

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