Statistical Estimation of Extreme Loads for the Design of Offshore Wind Turbines During Non-Operational Conditions

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ABSTRACT
The International Electrotechnical Commission (IEC) design standard (IEC 61400-3) for offshore wind turbines includes a design load case which considers loads on the turbine during extreme conditions, when the wind turbine is not operating and the blades are feathered. The recommendation of this design standard is to simulate 6 one-hour periods, each with wind and wave fields modeled as random processes, and to calculate design loads as the mean of the maximum from the six simulations. Previous studies have investigated this recommendation for fixed bottom offshore wind turbines and raised concerns about the stability of the estimate of the design load calculated using these guidelines. The research presented in this paper calculates statistics of extreme design loads as a function of the number of one-hour simula¬tions considered for three offshore wind turbine support structures: a fixed-bottom turbine supported by a monopile and two floating turbines, a spar buoy and a semi-submersible. The study considers one extreme design load, the moment at the tower base, one set of 50-year metocean conditions representive of the Northeast U.S. Atlantic coast, and five combinations of wind and wave models, including linear and nonlinear representations of irregular waves. The data generated from this study are used to assess the stability of the estimate of the extreme load for various numbers of one-hour simulations for each turbine type. The study shows that, for the considered metocean conditions and models, the monopile exhibits the least stability in the estimate of the extreme load and therefore requires more one-hour simulations to have comparable stability as the two floating support structures. Explanations for this result are provided along with a probabilistic formulation for estimating variability of design loads more efficiently, using fewer one-hour simulations.

1. INTRODUCTION
The worldwide installed capacity of offshore wind energy has grown quickly over the past 10 years, with 1.5 GW of capacity installed in 2014 and 4.0 GW projected to be installed in 2015 [16]. The current cost of harvesting energy generated by offshore wind is currently 2–3 times that of energy generated from fossil fuel, nuclear and onshore wind [3]. A portion of the high cost of generating energy from offshore wind is related to risk of structural damage to offshore wind turbines (OWTs) during extreme metocean conditions, such as those caused by a severe storm [3]. The overall risk or probability of structural damage is related to a combination of so-called short-term uncertainty (i.e., uncertainty in the maximum or extreme design loads caused by stationary metocean conditions for a particular period of time, e.g. one hour) and long-term uncertainty (i.e., uncertainty

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in the maximum metocean conditions for a particular period of time, e.g. the design life of the OWT). A quantitative understanding of both sources of uncertainty is essential to understanding the overall risk to OWTs caused by extreme metocean conditions, and this paper is focused on characterizing the short-term uncertainty.

Analytical predictions of this uncertainty are difficult because of the nonlinear forcing mechanisms of wind and waves, the significant geometric nonlinearity of OWT structures, the importance of dynamic effects in calculating loads [17]. For these reasons, it is common practice to estimate OWT loads with nonlinear dynamic time history analyses with wind and wave fields modeled as random processes (often, but not always, modeled as Gaussian processes) for a finite period of time [17]. The overall duration of the time history simulation as well as the number of independent simulations of random wind and wave processes are important factors in the short-term uncertainty of the prediction of the extreme design load, and the multiple frequencies present in the response [17]. Additional factors can influence the stability of the estimate of the extreme load, including: site specific conditions such as water depth and soil conditions, the severity of metocean conditions, and varying support structure configurations.

The preeminent design standard for offshore wind turbines, IEC 61400-3, recommends that offshore wind turbines be designed to withstand loads equal to a load factor (either 1.35 or 1.10, depending on whether the turbine is in a normal or abnormal state) multiplied by the mean of the maxima from six independent one-hour simulations of metocean conditions calculated to have a 50-year mean return period [1, 2]. Six independent simulations are recommended in consideration of the short-term uncertainty of loads due to the random wind and wave fields. The recommendation is intended to provide a stable estimate of the mean of the maximum load over a one-hour period. The recommendation is held fixed regardless of structural, site or metocean characteristics even though it is expected that these characteristics will influence the stability of the estimate of the mean. Indeed, recent research has shown that, depending on the random seeds chosen for the six one-hour simulations, the mean of the extreme load can vary significantly [17].

This research aims to understand the variability of the mean of the extreme loads as it relates to the number of one-hour simulations considered and structural characteristics. To consider variability due to structural characteristics, three types of support structure, each with different geometries and fundamental periods, are considered: a fixed-bottom monopile foundation as well as two floating platforms, a spar buoy and a semi-submersible. Additional, five combinations of wind and wave are considered: no wind with linear irregular waves, no wind with nonlinear irregular waves, wind with linear irregular waves, wind with nonlinear irregular waves, and wind with no waves. The focus of the results in this paper is on two combinations: wind with linear irregular waves and wind with nonlinear irregular waves. The simulations are based on a single metocean condition, characterized by three statistics, the significant wave height, the peak spectral period of the seastate and the mean wind speed, and assumed to be stationary for a one-hour period. The numerical values of the three statistics considered in this paper are calculated to have a 50-year mean return period based on 32 years of buoy measurements from a location near Nantucket, off the U.S. Northeast Atlantic coast. The water depth at the buoy site is 66m. One thousand one-hour simulations are conducted for each combination of structural type and wind/wave model, totaling 15,000 simulations. For all simulations, the maximum, resultant tower base moment is selected as the extreme load for comparison.

This paper begins with background on research into the short-term uncertainty of OWT loads and on commonly used models for calculating aerodynamic and hydrodynamic loads in the design of OWTs. In the next section, details of the numerical study considered in this paper are provided including descriptions of the three OWT structural configurations and the considered wind and wave models. This is followed by a presentation of the results of the numerical study, with particular emphasis on the stability or convergence of the mean of the maximum tower base moment as a function of the number of one-hour simulations to evaluate implications of the number of simulations in the design of OWTs for extreme conditions. Next, the results are discussed, and comparisons are made between the three structural configurations. Finally, after a section that describes the statistical theory underlying the results, conclusions and recommendations for additional research are provided.

2. BACKGROUND

Offshore wind turbines are subjected to both wind and wave loads, which cause drag and inertial forces on OWTs and their support structures. These drag and inertial forces are usually considered using nonlinear models, such as Blade Element Theory Momentum (BEM) for the calculation of wind forces on the blades of OWTs, and the Morison Equation or diffraction theory for the calculation of wave forces on the support structure of OWTs [1, 2]. This combined with the relative flexibility of OWTs, leads to a significantly nonlinear relationship between the wind and wave conditions and the response of the structure.

The response of OWTs subjected to wind and wave loads has been shown to have significant dynamic effects [17], and it is common practice to estimate OWT design loads with nonlinear dynamic time history analyses of structural models of OWTs subjected to random wind and wave fields for a finite period of time [1, 2, 17]. Because of the randomness in the wind and wave, there is short-term uncertainty in the magnitude of the maximum loads over a particular simulation period and due to this uncertainty, estimation of design loads requires two important decisions. The first is selecting the appropriate duration of simulations. Since the OWTs loads are influenced by interaction of time varying structural oscillations with randomly time varying winds and waves, the expected value of the magnitude of the loads will increase with an increasing simulation duration, as longer simulations provide more opportunities for unfavorable combinations of structural dynamics, wind and waves. The number of opportunities to encounter an unfavorable combination in a particular time period is related to the dynamic characteristics of the structure (e.g., modal periods), the seastate (e.g., peak spectral period and spectral shape) and the wind field (turbulence intensity and spectral shape). The second decision, which is directly tied to the first, is how many simulations of the selected duration are required to have a stable estimate of the statistics of the maximum load for each simulation. The typical statistic employed in OWT design is the mean of the maxima from each of the simulations [1].

Various design standards and authors use different simulation durations and number of simulations in the design and analysis of OWTs. The IEC 61400-3 standard prescribes that the design load for OWTs should be a fixed load factor (either 1.35 or 1.10, depending on whether the turbine is in a normal or abnormal state) multiplied by the mean of the maxima from six one-hour simulations. The American Bureau of Shipping suggests that the design load should be a load factor multiplied by the mean of the maxima from at least 12 one-hour simulations for jackets and monopiles [7]. Agarwal and Manuel used 100 ten-minute simulations in order to predict the statistics of the base shear and overturning moment of a monopile structure [6, 4]. Fogle et al. found that with 30 one-hour simulations, the 90% confidence interval on the 84th percentile load never exceeded 15% [8]. Hallowell et al. found that 24 one-hour simulations were required to have an appropriately stable estimate of the mean of the maximum overturning moment acting on a monopile subjected to one-hour of hurricane conditions including breaking waves [9].

An alternative method for calculating statistics of extreme loads is the Peak Over Threshold (POT) method. In this method, extreme loads are recorded every time loads exceed a threshold value during the duration of the simulation and design loads are calculated based on statistics of the distribution of the record of loads exceeding the threshold. This method also requires making two decisions: the duration of the simulation and the magnitude of the threshold. While this method has some computational advantages by using simulation results more efficiently, this study is focused on the method described in the IEC standard, since it is more common in practice. More details on the POT method as it relates to the estimation of OWT loads are provided by Agarwal [4].

3. SIMULATION TOOLS AND METHODS

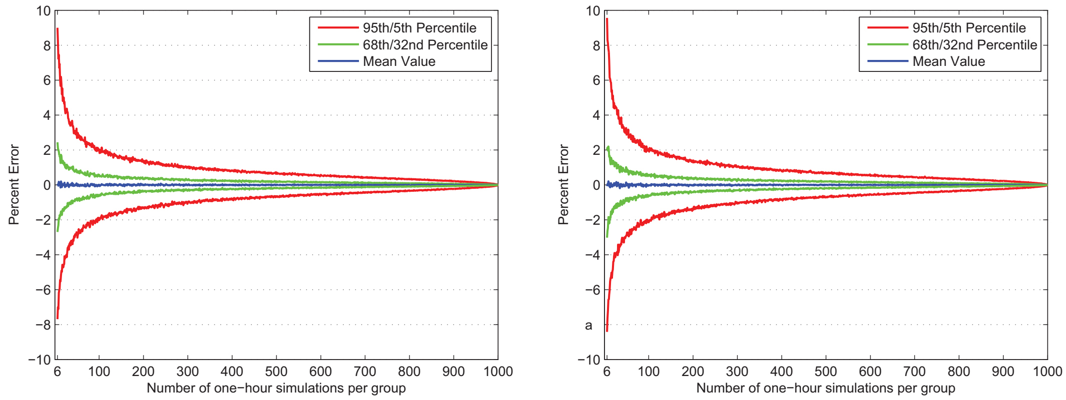
The analyses presented in this paper are run using the National Renewable Energy Laboratory's (NREL) open-source program FAST, which is a time-domain analysis program with coupled aero-hydro-servo-elastic behavior for simulating onshore and offshore wind turbines. Aerodynamic forces are calculated with submodule AeroDyn, which uses blade-element momentum (BEM) theory with corrections for tip and hub losses and dynamic stall. Hydrodynamic forces are calculated with the submodule HydroDyn, which uses linear or nonlinear wave theory to solve the wave kinematics equations and the Morison equation to convert wave kinematics to structural loads. The structural analysis is based on a modal representation involving two modes each in the fore-aft and side-side directions to define the flexibility of the blades and tower/support structure. The program also includes a controls module, which incorporates the blade pitch and torque controllers into the model [13, 12].

Models for the three structural types considered in this paper are from the Offshore Code Comparison and Collaboration pro ject. The monopile and spar buoy models are taken from the OC3 project with water depths of 30m and 320m respectively, while the semi-submersible model is taken from the OC4 project with a water depth of 200m [11, 15]. All three structures have been modified to support a representative 5 MW wind turbine model developed by NREL and based on the REpower 5 m machine, a three bladed upwind machine with a 90 m hub height and a 126 m rotor diameter [10]. While the tower loads on the two floating platforms are influenced by both the rigid body motions of the platform and the tower vibration, the monopile loads are driven by the cantilever-like deformation of the tower. The baseline control includes variable speed operation and collective blade pitch control, however, for this research only wind speeds greater than operational conditions are considered, so the details of the controllers are not relevant here.

All simulations consider a mean wind speed of 35 m/s, a significant wave height of 15 m, and a peak spectral period of 14 s. These conditions are based on a calculation of 50-year conditions, using measurements from the U.S. National Data Buoy Center buoy #44008, which is located southeast of Nantucket. The procedure for calculating the 50-year conditions of each variable follows that described by Myers et al. [14]. The wind random process is modeled as a Gaussian process with Kaimal spectral model and the Normal Turbulence Model A, as defined in IEC 61400-3, which, for 35 m/s wind, corresponds to a turbulence intensity of approximately 13.7%. The linear irregular wave random process is modeled as a Gaussian process with a JONSWAP spectral model, while the nonlinear irregular wave random process is modeled as a non-Gaussian process, with Rayleigh distribution assumed for the wave amplitudes, with a JONSWAP spectrum and the kinematics solved in the spectral domain including a second-order, nonlinear component of the wave series using a method outlined by Agarwal and Manuel [5]. Note that wave stretching is not considered in the wave modeling, as this feature is not included in the current edition of FAST. All simulations are run with the OWT in an idling condition, with the rotor allowed to spin and with the blades feathered to minimize aerodynamic loading. The study includes 15,000 one-hour simulations. Each of the three support structures are simulated for 1000 one-hour periods for each of the following combinations of wind/wave models: wind and nonlinear irregular waves, wind and linear irregular waves, no wind with nonlinear irregular waves, no wind with linear irregular waves, and wind with no waves. In all cases, waves are modeled as unidirectional. The results use the maximum resultant bending moment at the tower base, calculated using a root-sum-square value of the fore-aft and side-to-side bending moments.

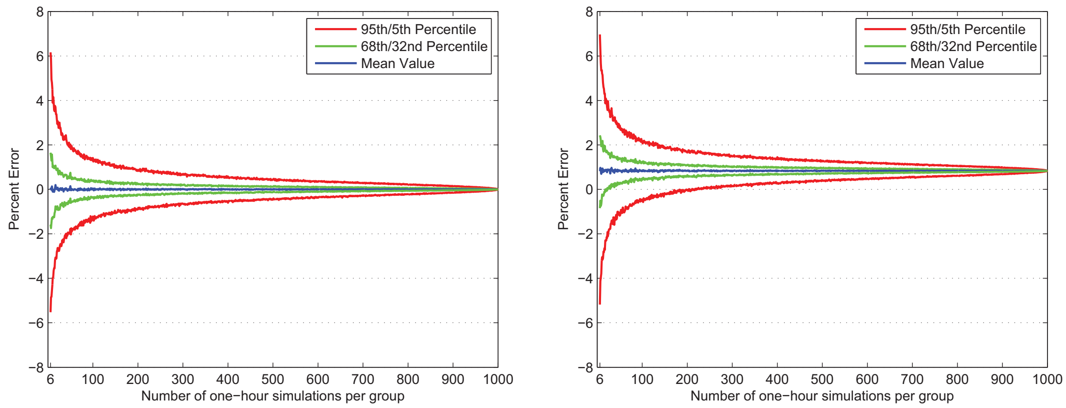
4. RESULTS

Results from the simulations are provided in Figures 1–7. Figures 1–5 show statistical curves from distributions calculated from 1000 one-hour simulations for a particular support structure and wind/wave model. Figure 7 compares mean results among the three support structures and the five combinatioms of wind/wave models. The first five figures each include several curves which indicate how the variability in the mean of the maximum moment at the tower base changes with the number of one-hour simulations considered. The variability is expressed through statistics of the distribution of the percentage error of the mean maximum moment for a given number of one-hour simulations relative to the mean from all 1000 maximum moments from each one-hour simulation. The distribution is calculated by sampling, without replacement, from the 1000 maximum moments to create 1000 random groups, each containing n maximum moments. Group sizes are considered from n equals 6 to 1000, spaced by 1. A mean is calculated for each of these 1000 groups, and statistics of the distribution of these 1000 means are calculated and then converted to percentage error, where error is measured relative to the mean of the 1000 maximum moments. In the Figures, the mean value (plotted in blue), the $32^{nd}/68^{th}$ percentile values (plotted in green) and the $5^{th}/95^{th}$ percentile values (plotted in red) are provided for the distribution of the percentage error for each considered group size (from six to 1000, spaced by 1). For example, Figure 1a indicates that there



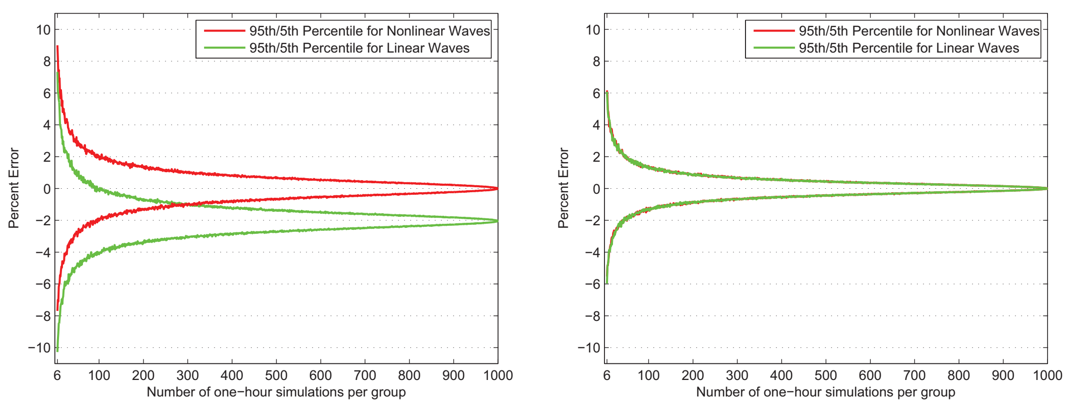
(a) (b)

Figure 1. Statistics of the distribution of the percentage error of the mean maximum moment for 1000 random groups for various numbers of one-hour simulations per group, from 6 to 1000. Results are for a semi-submersible support structure under wind and (a) linear waves and (b) nonlinear waves. Error is measured relative to the mean of the maximum moment from all 1000 simulations



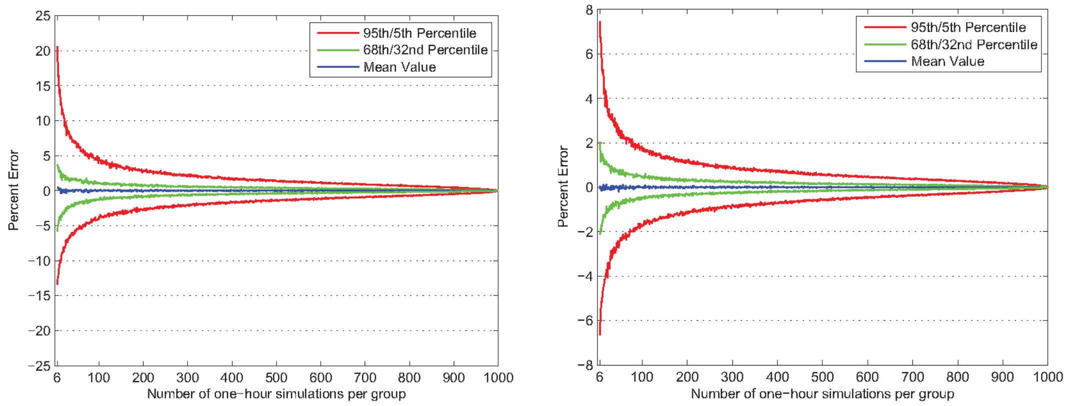
(a) (b)

Figure 2. Statistics of the distribution of the percentage error of the mean maximum moment for 1000 random groups for various numbers of one-hour simulations per group, from 6 to 1000. Results are for a spar buoy support structure under wind and (a) linear waves and (b) nonlinear waves. Error is measured relative to the mean of the maximum moment from all 1000 simulations



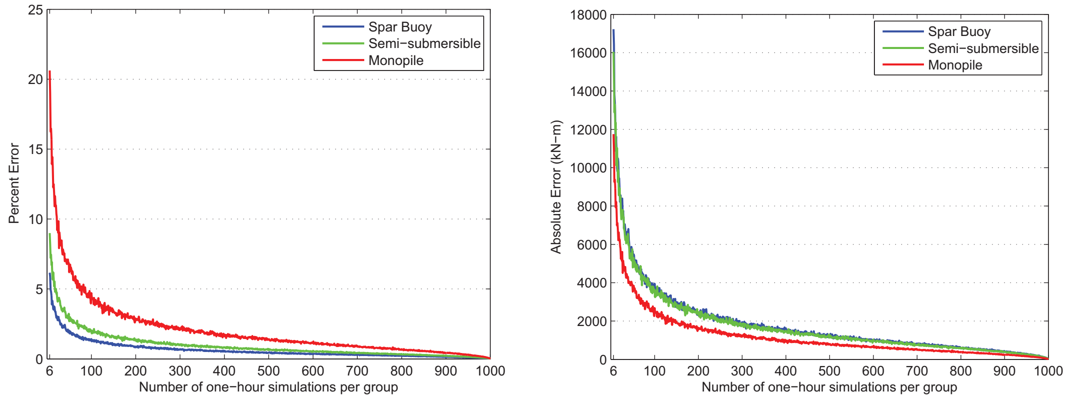
(a) (b)

Figure 3. Statistics of the distribution of the percentage error of the mean maximum moment for 1000 random groups for various numbers of one-hour simulations per group, from 6 to 1000. Results are for conditions with wind and linear (green) and nonlinear (red) waves for (a) semi-submersible and (b) spar buoy support structure. In both Figures, error is measured relative to the mean of the maximum moment from the corresponding 1000 simulations under linear waves



(a) (b)

Figure 4. Statistics of the distribution of the percentage error of the mean maximum moment for 1000 random groups for various numbers of one-hour simulations per group, from 6 to 1000. Results are for a monopile support structure under wind and (a) linear waves and (b) nonlinear waves. Error is measured relative to the mean of the maximum moment from all 1000 simulations



(a) (b)

Figure 5. 95th percentile values of the distribution of the error of the mean maximum moment for 1000 random groups for various numbers of one-hour simulations per group, from 6 to 1000. Results are for all three support structures and for wind with nonlinear waves. Error is measured relative to the mean of the maximum moment from all 1000 simulations and expressed as (a) a percentage and (b) an absolute value

is 90% confidence that the mean of the maxima from a group of 6 one-hour simulations will be within approximately ±9% of the mean of maxima from 1000 one-hour simulations.

Figure 1 provides results for the semi-submersible support structure for wind with nonlinear waves (Figure 1a) and wind with linear waves (Figure 1b). Figures 2 provides results for the spar buoy for wind with nonlinear waves (Figure 2a) and wind with linear waves (Figure 2b). Comparing Figure 1a with Figure 1b and Figure 2a with Figure 2b shows that is very little difference in variability of the prediction of the extreme moment between the linear and nonlinear waves for both the spar buoy and semi-submersible support structures. There is, however, a difference in the mean of the extreme moments between the linear and nonlinear wave cases. This differences is illustrated in Figures 3a and 3b, which show results for the semi-submersible and the spar buoy, respectively. In these two figures, 5th/95th percentile values results are plotted for both linear (green) and nonlinear waves (red), and the percentage error is calculated relative to the mean of the 1000 maximum moments for nonlinear waves. The figures show that, while the spar buoy and semi-submersible support structures both have nearly identical variability for both linear and nonlinear waves, the semi-submersible support structure converges to a mean value that is 2% higher for nonlinear waves than for linear waves.

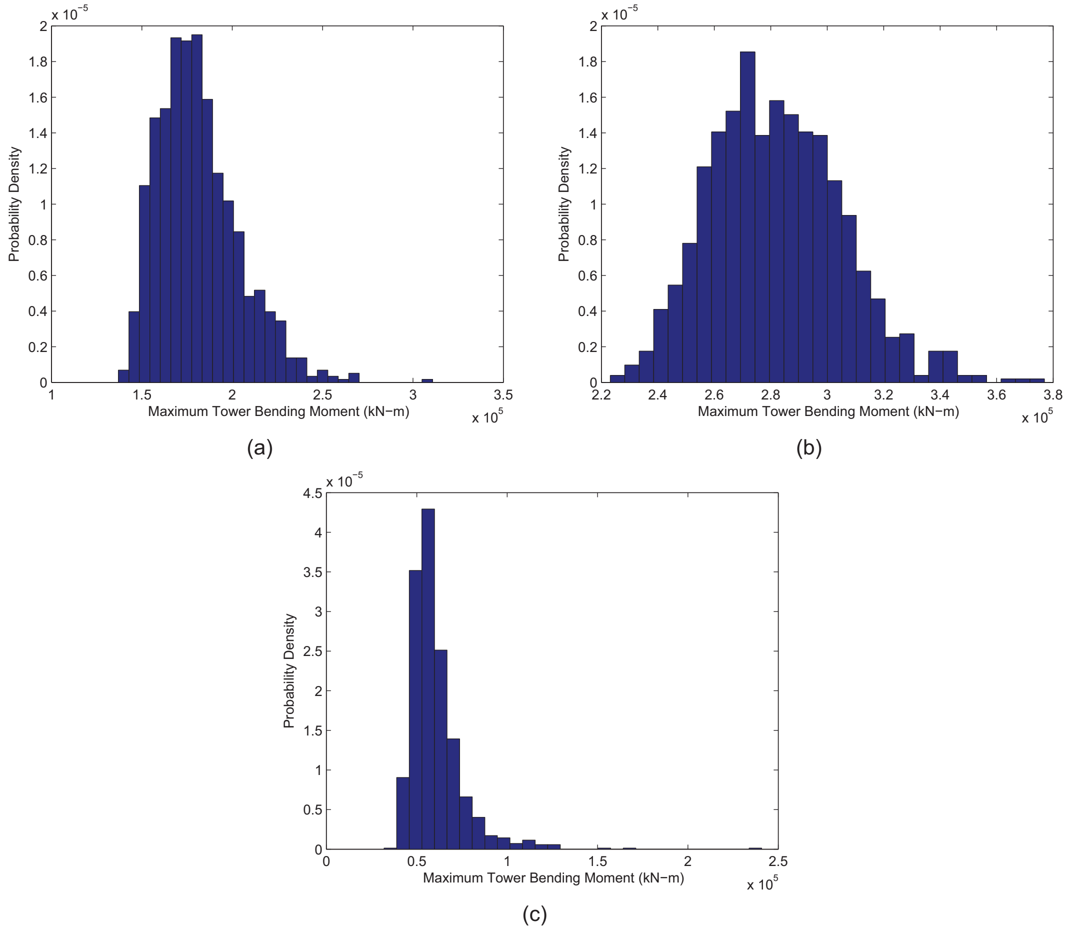


Figure 6. Normalized histograms of maxima from 1000 simulations with wind and nonlinear waves for (a) the semi-submersible, (b) the spar buoy, and (c) the monopile platforms

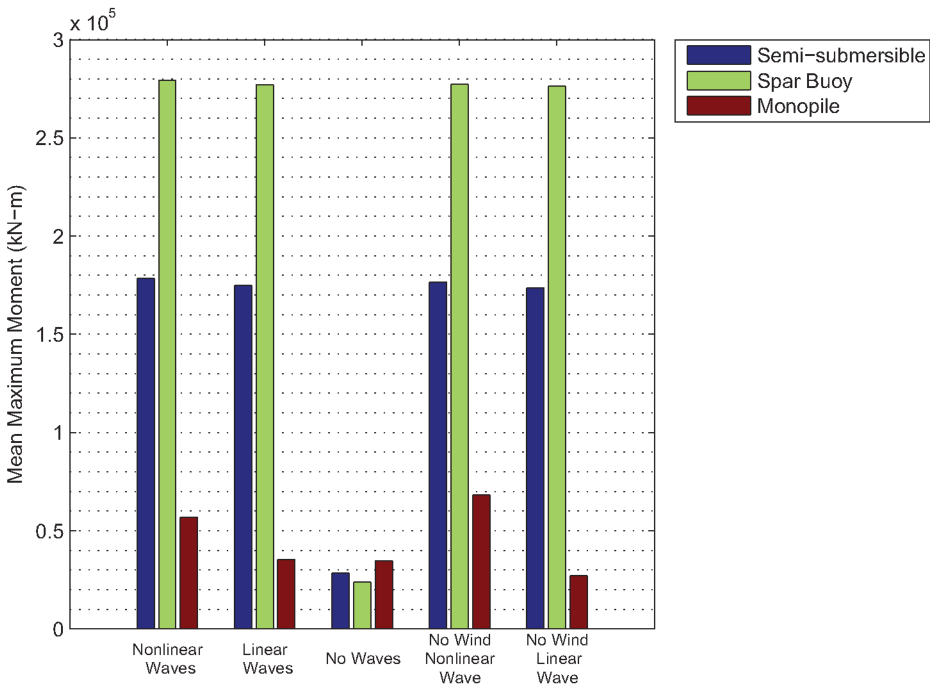


Figure 7. Comparison of the mean of the maximum moment from all 1000 simulations for all three support structures and all five combinations of wind/wave models

Figure 4 provides results for the monopile support structure for wind with nonlinear waves (Figure 4a) and wind with linear waves (Figure 4b). Interestingly, the variability in the extreme moment is higher for the monopile than for the two floating support structures. For example, the 95th percentile value for the percent error from a group of 6 one-hour simulations is 20% for nonlinear waves. The explanation for this effect is that the response of the two floating support structures tends to be more periodic and is dominated by inertial loads driven by the peak spectral period of the seastate (approximately 0.8 Hz) and the natural rigid body period of the floating structures. (Note that the monopile has a first tower bending frequency of 0.28 Hz while the spar and semisubmersible have surge frequencies of 0.008 and 0.009 Hz and pitch frequencies of 0.034 and 0.037 Hz respectively. Both floating structures have first tower bending frequencies of 0.5 Hz.) Because of this, for the floating structures, random variation of the response from one one-hour simulation to another is less than that for a monopile, which has loads that are more sensitive to the size of the largest wave during the one-hour simulation. The size of the largest wave can vary significantly from one one-hour simulation to another. The variability of wave height and the sensitivity of loads to wave height are greater for nonlinear waves than for linear waves, and this is reflected by the simulations for linear waves (Figure 4b) having less variability than the simulations for nonlinear waves (Figure 4a).

In addition, Figures 1–4 exhibit asymmetry in the positive and negative percentile bounds. For all platforms, it is more likely that the maximum load is overpredicted than underpredicted, as shown by a higher percent error above the mean for the positive percentiles. This is due to the shape of the distributions of the maxima, which exhibit long upper tails. Figures 6a–6c show normalized histograms of the maxima from 1000 simulations for the semi-submersible (Figure 6a), spar buoy (Figure 6b) and monopile (Figure 6c). The monopile histogram has the largest skew toward the right tail, and also shows the highest degree of asymmetry of the percentile plots in Figure 4. The result of this finding is that the estimate of the maximum load for a given group of simulations is more likely to be a conservative estimate.

Figure 5 compares the 95th percentile value for the percent error (Figure 5a) and for the absolute error (Figure 5b) with wind and linear/nonlinear waves for all three support structures. Figure 5a shows that the monopile has the largest variability in percent error, followed by the semi-submersible and then the spar buoy. This result implies that a designer should run more one-hour simulations of a monopile than for a semi-submersible or a spar buoy if comparable confidence is desired in the estimation of the extreme load. Although the monopile has the largest variation in the percentage error, the variation in the absolute value of the error is smallest because the mean maximum moment from all 1000 simulations is smallest for the monopile (5.70×10^4 kN-m for the monopile, 1.78×10^5 kN-m for the semi-submersible, and 2.79×10^5 kN-m for the spar buoy). For example, the results in Figure 5b indicate that, for the mean of the maxima from a group of 6 one-hour simulations, a designer can expect with 95% confidence to be within approximately 17,000 kN-m of the maximum moment from 1000 one-hour simulations for the spar buoy and semi-submersible and within 12,000 kN-m for the monopile.

Figure 7 compares the mean value of the maximum moment from all 1000 simulations for the three support structures and five combinations of wind/wave models. The figure shows that, for the considered situations, the mean of the maximum tower base moment is dominated by wave action, because not only is the moment corresponding to the "no waves" condition small for all three support structures, but there is also almost no difference between the mean moments for conditions considering waves with or without wind.

5. THEORY

In this section, the estimation of the variability or uncertainty of the extreme load for this design case is examined analytically. This analysis suggests that there is a more computationally efficient way to estimate variability in the extreme design load than the method used in this paper. The problem of the quality of the estimate of the peak demand obtained from samples of a wind turbine simulation can be cast in the theory of stochastic processes. In that context, the problem becomes that of the rate of convergence, towards zero, of the variance of the estimator of the extreme value of a stochastic process observed in a given time interval. Let the bending moment at the tower base be denoted by $M(t)$, a stochastic process and let $M(t)$ be observed during a time interval $(0, t_f)$. The maximum value of the process in that time interval is a random variable

$$M_{max,\, t_f} = \max\,(M(t) : t \in (0, t_f)) \tag{1}$$

that has distribution $F_{m_{max},\, t_f}(m)$ and second moment properties $_{m_{max},\, t_f}$, $\sigma_{m_{max},\, t_f}$. If n observations of $M(t)$, $t \in (0, t_f)$ are available the mean of the extreme value can be estimated by

$$\bar{M}_{max,t_f} = \frac{1}{n}\sum_{i=1}^{n} m_{max,t_f,i} \tag{2}$$

which is itself a random variable with distribution $F_{\bar{m}_{max},\, t_f}(m)$ and second moment properties $_{\bar{m}_{max},\, t_f}$, $\sigma_{\bar{m}_{max},\, t_f}$.

If the observations of $M(t)$ are independent then the estimator $\bar{M}_{max,\, t_f}$ is unbiased with variance and coefficient of variation

$$\begin{aligned} \sigma^2_{\bar{m}_{max},t_f} &= \frac{\sigma^2_{m_{max},t_f}}{n} \\ \mathrm{COV}_{\bar{m}_{max},t_f} &= \frac{\sigma_{m_{max},t_f}}{\mu_{m_{max},t_f}\sqrt{n}} \end{aligned} \tag{3}$$

Since the estimator $\bar{M}_{max,\, t_f}$ is typically used to define design loads it is the variance, or coefficient of variation, as given in Eq. 3 that must be understood to provide confidence in the design load. To directly estimate the stability of $\bar{M}_{max,\, t_f}$, however, requires the generation of large number of samples as shown in the examples presented in this paper. Equation 3 shows that, rather than having to directly estimate $\mathrm{COV}_{\bar{m}_{max},\, t_f}$, all that is needed is a high quality estimate of $_{\bar{m}_{max},\, t_f}$ and $\sigma_{\bar{m}_{max},\, t_f}$, estimates that can typically be obtained with many fewer samples.

The numerical examples presented in this paper with wind and nonlinear waves are now examined in the context of Eq. 3, but first it is important to note that the process $M(t)$ is both non-Gaussian (see Figure 8 for sample time histories and Figure 9 for a comparison to the Gaussian distribution; note that in order to facilitate comparison to the Gaussian distribution the samples of $M(t)$ have been standardized to have unit variance and zero mean) and non-narrow-banded (see

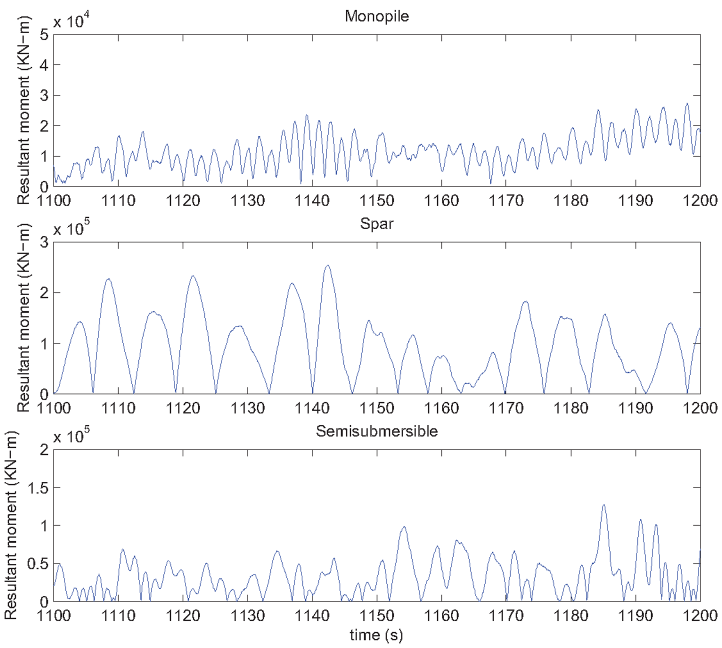


Figure 8. One hundred second samples of the time history of the tower base bending moment $M(t)$ for the (a) monopile, (b) spar buoy, and (c) semi-submersible support structures subjected to wind and nonlinear waves

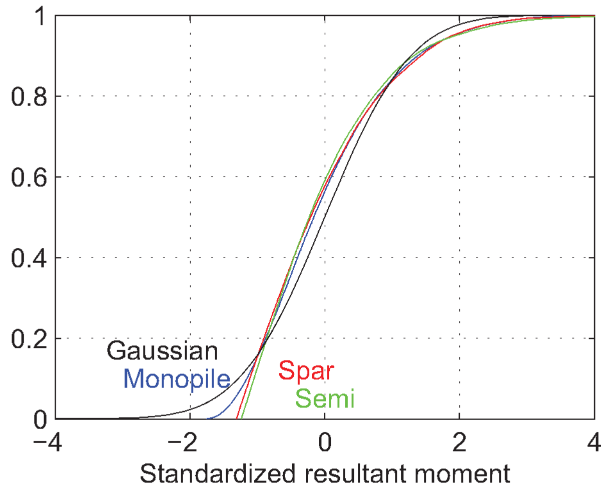


Figure 9. Marginal cumulative distribution functions of the time history of the tower base bending moment $M(t)$ standardized to zero mean and unit variance for the monopile, spar buoy, and semi-submersible support structures subjected to wind and nonlinear

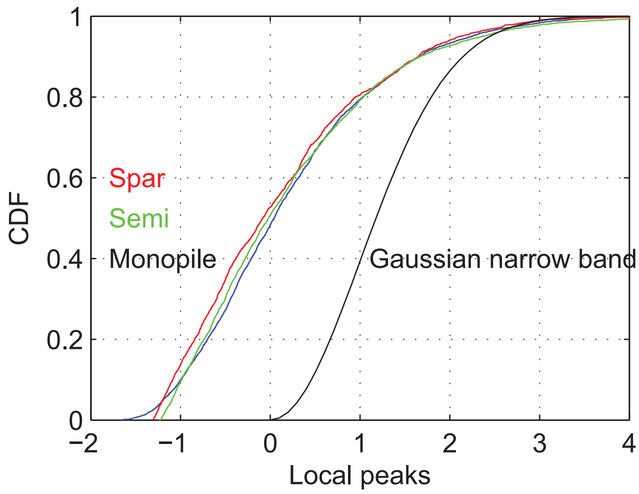


Figure 10. Comparison of a Gaussian narrow-banded process and the distribution functions of the local peaks of the time history of the tower base bending moment $M(t)$ standardized to zero mean and unit variance for the monopile, spar buoy, and semi-submersible support structures subjected to wind and nonlinear waves

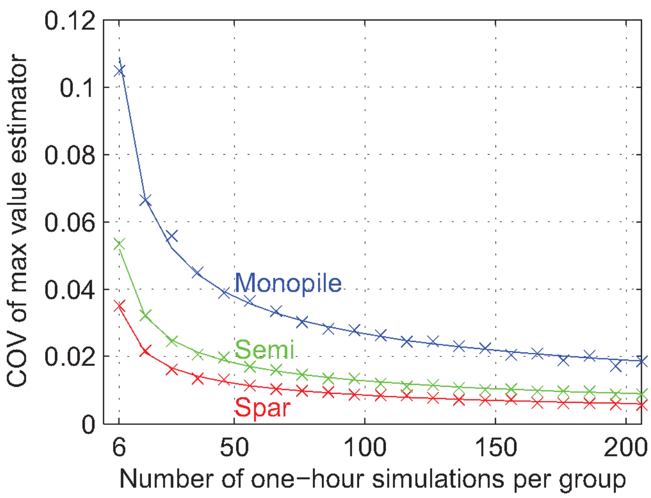


Figure 11. The COV of the estimate of the mean of the maximum moment based on Eq. 3 (solid lines) and results from the simulations with wind and nonlinear waves (x markers) for 500 random groups with various numbers of one-hour simulations per group, from 6 to 200

Figure 10 for a comparison of the local peak distribution of $M(t)$ to that for a Gaussian narrow-banded process; again, samples have been standardized to zero mean and unit variance for ease of comparison). These characteristics significantly increase the challenge in directly estimating the properties of $\bar{M}_{max,\, t_f}$. Specifically, if $M(t)$ were Gaussian and narrow-banded direct expressions would be available for the mean and variance of the extreme value of the process.

Figure 11 shows the computational benefit of using Eq. 3 to estimate the extreme value statistics of the random process. The markers indicate $COV_{\bar{m}_{max},\, t_f}$ as estimated from 500 random groups for various numbers of one-hour simulations and the solid lines are the prediction of Eq. 3. The agreement is essentially exact, and since good estimates of $\bar{m}_{max,\, t_f}$ and $\sigma_{\bar{m}_{max},\, t_f}$ can be obtained with relatively few simulation runs, this approach promises significant reduction in computational expense for designers seeking to quantify uncertainty in their estimates of extreme loads.

6 CONCLUSIONS

The IEC design standard 61400-3 recommends calculatingdesign loads during extreme non-operational conditions, when the rotor is idling and the blades are feathered, as the mean of the maximum loads from six one-hour simulations. To investigate the accuracy of this recommendation, a numerical study was completed considering 1,000 one-hour simulations for three support structures (monopile, semi-submersible and spar buoy), five combinations of wind/wave models and one metocean condition, calculated to have a 50-year mean return based on measurements from a buoy near the U.S. Northeast Atlantic coast. By sampling from the distribution of 1,000 one-hour simulations, the stability of the mean of the maximum tower base moment was quantified for particular numbers of one-hour simulations. The results are generally informative to designers considering running more one-hour simulations to get a more stable estimate of the mean of the maximum moment. For the considered combinations of wind/wave models and metocean conditions, the results show that considering six one-hour simulations results in a coefficient of variation of the mean of maximum moment equal to 11%, 5% and 4% for the monopile, semi-submersible and spar buoy, respectively. For 50 one-hour simulations, these numbers reduce to 4%, 2% and 1% respectively. The results suggest that considering six one-hour simulations may be sufficient for some applications (e.g., preliminary design estimations), particularly for the floating structures, but the level of uncertainty must be weighed against the increased computation cost for running more simulations.

The monopile showed more variability in extreme loads than the two floating structures, and would therefore require more simulations to estimate design with comparable stability as the two floating structures. The variability in the extreme loads for the monopile was even greater for nonlinear waves. Both of these observations can be explained by noting that the instances of extreme loading on the monopile are driven by extreme wave heights, while the extreme loading on the floating platforms is driven more by inertial loads caused by rigid body motions. Additionally, for the considered numerical study, the design loads were dominated by wave loading, with the presence or absence of wind having little effect on the loads. An analytical study showed that coefficient of variation of the mean of the maximum moment can be reliably estimated using estimates for the mean and variance. The authors recommend that further study of extreme load stability of offshore structures should include investigation of changes due to water depth, metocean conditions, breaking waves (particularly for the monopile), as well as a comparison with a jacket structure or other floating structures.

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