Probabilistic Modeling of Joint Hurricane-induced Wind and Wave Hazards to Offshore Wind Farms on the Atlantic Coast

V. Valamanesh, A. T. Myers & J.F. Hajjar

Department of Civil and Environmental Engineering, Northeastern University, Boston, MA 02115, USA

S.R. Arwade

Department of Civil and Environmental Engineering, University of Massachusetts, Amherst, MA 01003, USA

ABSTRACT: Offshore wind turbines are designed per the IEC 61400-3 standard to withstand dozens of load cases involving combinations of wind and wave-induced load effects. Three of these load cases consider extreme loading wherein a turbine with a parked rotor and feathered blades is expected to sustain combined wind and wave loads corresponding to a 50-year mean return period. For most locations along the Atlantic coast, this 50-year combination of wind and wave will be influenced by hurricanes. During offshore hurricanes and other storms, a multi-hour lag between the maximum winds and the maximum waves is typical and this has important implications for the development of a rational method to estimate the joint wind and wave hazard at a particular return period. The lag causes significant differences in the magnitudes of the wind and wave depending on whether the hazards are assessed independently, as is currently recommended by existing guidelines, or jointly. In this paper, we introduce a procedure based on multivariate annual extreme value distributions, the Nataf model and a joint exceedance condition to estimate contours of wind and wave with a constant mean return period. Using 24 years of wind and wave measurements from a NOAA buoy off the Atlantic coast of Florida, we present numerical results to assess the impact of two methods for selecting the joint annual maxima of wind and wave. In the first method, the annual maximum for the hourly wind speed is paired with the simultaneous value of the significant wave height and, in the second method, the annual maximum for the significant wave height is paired with the simultaneous value of the hourly wind speed. The contours of constant mean return period resulting from each method are significantly different. The 24 year period at this particular station includes measurements from five Atlantic hurricanes.

1 INTRODUCTION

The development of renewable energy sources is a critical global need. In the United States, the Department of Energy (DOE) has set a target for the U.S. to generate 20% of its electricity demand from wind energy by 2030 and the U.S. National Renewable Energy Laboratory (NREL) has calculated that an optimal strategy to achieve the DOE target should include 54 GW of offshore capacity. The middle and northern Atlantic coast, with large wind resources and proximity to major population centers, is a natural place for such development, however, this region is also at considerable risk from severe hurricanes.

Despite approximately 20 proposed offshore projects in the middle and northern Atlantic region, no offshore wind turbine has yet to be installed in the U.S. This situation is in contrast to that of Europe where 1000s of offshore turbines already generate a capacity of 3 GW of electricity. The experience in Europe provides much technical insight for the U.S., however, the presence of hurricane risk in the U.S. is a technical challenge that Europe has not needed to address. Hurricane risk adds to the cost of energy by increasing uncertainty associated with wind farm performance and therefore the financing and insurance costs associated with offshore wind energy. These costs can compose as much as 40% of the total cost of offshore wind energy, which now exceeds the cost of traditional sources by a factor of two. A better understanding of both the hurricane hazard and the fragility of offshore wind turbines can reduce this uncertainty, thereby reducing the cost of offshore wind energy.

A 2010 report by NREL identified a lack of models and methodologies to assess the joint hurricane-induced wind and wave hazards to offshore wind farms along the Atlantic coast as a major barrier to the development of offshore wind energy in the United States. In the current study, we make progress towards the development of rational methods and methodologies to consider hurricane-induced joint wind and wave hazard in the design of offshore wind turbines. The paper first provides background on methods within two existing design standards, IEC 61400-3 (2009) and API 2INT-MET (2007), for assessing the joint wind and wave hazard associated with the design of offshore structures. Both of the methods described in these standards are based on univariate extreme value theory wherein annual maxima of the wind and wave hazard are first assessed independently and then combined. Following the review of these methods, an alternative procedure, based on multivariate extreme value theory and on the Nataf model, is proposed and the impact of the time lag between maximum wind and maximum wave during storms is discussed. Following this section, a numerical example of the proposed multivariate method is presented based on 24 years of data obtained from a NOAA buoy station (Station 41010) located 210 km off the Atlantic coast of Florida. The paper ends with some discussion and conclusions on the assessment of hurricane-induced wind and wave hazard.

2 BACKGROUND

2.1 Review of existing design methods for assessing wind and wave hazard

Two widely used standards to determine the magnitudes of wind and wave for the design of offshore structures are IEC 61400-3 (2009) and API 2INT-MET (2007). In IEC 61400-3, the intended return period for the design wind and wave is 50 years while in API 2INT-MET, the return period is 100 years. It is important to note that API 2INT-MET explicitly considers the influence of hurricane-induced hazard, while the current edition of IEC 61400-3 does not. The next edition of IEC 61400-3 is expected to include explicit guidance on the assessment of design wind and wave in locations vulnerable to hurricanes.

The current edition of the IEC standard requires that wind turbines be designed for three combinations of wind and wave loading corresponding to the extreme 50-year conditions during which the turbine is in survival mode (i.e. the rotor is parked and blades are feathered). The first combination considers a turbulent wind history defined in terms of the 50-year hourly wind speed and an irregular wave history defined in terms of the 50-year hourly significant wave height. The standard prescribes no fewer than 6 1-hour simulations of a dynamic structural model of the turbine simultaneously subjected to the irregular wave history and the turbulent wind history and that the maximum structural demands from these simulations be included as one of the design load cases. The second and third combinations consider steady loading wherein structural demands are assessed on a turbine model subjected to simultaneous wind and wave. The second combination considers loading due to the 50-year hourly wind speed, converted to the maximum expected three second gust, and the 50-year hourly significant wave height, converted to the maximum expected wave and then reduced to reflect the expectation that the 3-second gust and maximum wave will not occur simultaneously. The third combination considers loading due to the 50-year extreme wave height, converted to the maximum expected wave and the 50-year hourly wind speed, converted to the three second gust speed and then reduced to reflect the improbability of simultaneous maxima. The standard does not provide a reference for the basis of the reduction factors. All three load combinations are based on the 50-year hourly wind speed and the 50-year significant wave height, each of which is required to be assessed independently. The maximum demands from each of these three combinations of loading controls the design for survival during extreme 50-year conditions.

API 2INT-MET, a guideline targeting the oil & gas industry, emphasizes the assessment of hurricane-induced design loads for offshore structures located in the Gulf of Mexico. Hindcasts of historical hurricanes between 1950 and 2005 form the basis for the provided values for wind and wave intensities at different return periods. The association of intensities with return periods is based on independent univariate analysis of annual maxima of the wind and wave. Joint (i.e. simultaneous) wind and wave intensities are determined by a reduction factor which accounts for the expectation that annual maxima of wind and wave may not occur simultaneously. The guideline recommends that the 10-year wind and wave intensities be combined with no reduction, but, for return periods greater than 10 years, a reduction factor of 0.95 is recommended. In practice, this means that the 50-year intensity of one independent hazard is combined with 0.95 times the 50-year intensity of the other independent hazard. The guideline does not provide justification for these factors and the authors are not aware how they are determined.

2.2 Extreme value approach

Extreme value distributions are a well-established family of distributions that have been applied to many practical situations, such as modeling of riverine flooding, simulation of weakest link phenomena, such as brittle fracture of materials, or, as is relevant for this paper, prediction of annual maxima of wind speeds and sea state for an offshore location. Extreme value theory forms the basis for the association of a wind or wave intensity with a mean return period. Extreme value theory describes how, for sufficiently long sequences of independent and identically distributed random variables, the maxima of samples of size n can be fitted to one of three basic families when n is sufficiently large. These three families, known as the Gumbel or Type I, Frechet-Tippett or Type II and Weibull or Type III distributions are often combined into a single distribution known as the so-called generalized extreme value (GEV) distribution which has the following cumulative distribution function:

$$F_x = \exp\left\{-\left[1+\xi\left(\frac{x-\mu}{\sigma}\right)\right]^{-1/\xi}\right\} \quad (1)$$

The GEV distribution has three parameters, the location parameter, μ, the scale parameter, σ and the shape parameter, ξ. The three types of distributions described previously are special cases of the GEV and can be obtained by varying the magnitude of the shape parameter ξ. The Type II and III distributions correspond respectively to $\xi > 0$ and $\xi < 0$ and the Type I distribution is approached in the limit as $\xi \rightarrow 0$. The unification of the three extreme value distributions into a single family simplifies modeling because there is no necessity to subjectively select one of the three special cases (Coles, 2001).

3 MULTIVARIATE EXTREME VALUE MODEL

The previous section described the generalized extreme value distribution for univariate random variables, however, in the context of offshore hazard, a multivariate extreme value distribution, which provides the joint distribution of two random variables with marginal GEV distributions, is more relevant. One model for creating a joint distribution of two random variables is the so-called Nataf model (Liu & Kiureghian, 1986, Bucher, 2009), which approximates the joint probability density function of random variables X_i and X_j based on their marginal distributions and covariance. The model is based on a transformation between the original correlated variables and variables V_i and V_j whose joint density is assumed to be multi-dimensional Gaussian with standard normal marginal distributions. The random variables are transformed between the Gaussian space and the original space per the following equations:

$$X_i = F_{X_i}^{-1}\left(\Phi_{V_i}(v_i)\right) = g(v_i) \quad (2)$$

$$X_j = F_{X_j}^{-1}\left(\Phi_{V_j}(v_j)\right) = g(v_j) \quad (3)$$

where Φ_{V_i} is the cumulative distribution function of the standard normal distribution and $F_{X_i}^{-1}$ is the inverse of the cumulative distribution function of X_i. The correlation coefficient between random variables X_i and X_j is:

$$\rho_{X_iX_j} = \frac{\mathrm{E}[(x_i-\mu_{X_i})(x_j-\mu_{X_j})]}{\sigma_{X_i}\sigma_{X_j}} \quad (4)$$

where μ_{X_i} and σ_{X_i} are the mean and standard deviation of random variable X_i. After expanding, simplifying and rearranging Eq. 4, the expectation of the product of X_i and X_j can be expressed as:

$$E[X_iX_j] = \rho_{X_iX_j}\sigma_{X_i}\sigma_{X_j} + \mu_{X_i}\mu_{X_j} \quad (5)$$

The expectation operation in Eq. 5 can be transformed into standard normal space by substitution of $g(V_i)$ for X_i and $g(V_j)$ for X_j per Eqs. 2 and 3. Following this substitution and expansion of the expectation operator, Eq. 5 is recast into the following form:

$$E[X_iX_j] = E[g(v_i)g(v_j)] = \int_{-\infty}^{\infty}\int_{\infty}^{\infty} g(v_i)g(v_j)\phi_{V_iV_j}(v_i, v_j, \rho_{V_iV_j})dv_i dv_j \quad (6)$$

where $\rho_{V_iV_j}$ is the correlation coefficient between random variables V_i and V_j and $\phi_{V_iV_j}$ is the joint probability density function for standard normal random variables V_i and V_j. Combining Eqs. 5 and 6 yields Eq. 7 which can be solved iteratively for $\rho_{V_iV_j}$.

$$\rho_{X_iX_j}\sigma_{X_i}\sigma_{X_j} + \mu_{X_i}\mu_{X_j} = \int_{-\infty}^{\infty}\int_{\infty}^{\infty} g(v_i)g(v_j)\phi_{V_iV_j}(v_i, v_j, \rho_{V_iV_j})dv_i dv_j \quad (7)$$

Once $\rho_{V_iV_j}$ is determined, the joint probability density function $\phi_{V_iV_j}$ can be transformed to original space to provide a distribution that has the same marginal distributions and covariance as $f_{X_iX_j}$, the joint probability density function of X_i and X_j, and thus serves as second order approximation of $f_{X_iX_j}$.

Unlike scalar hazard intensity meausres, vector intensity measures do not have a unique rank order nor a unique association between intensities and mean return period. One method for associating mean return periods with joint random variables in the inverse first order reliability method or IFORM, proposed by Winterstein et al. (1993) and based on the concept of "environmental contours" which determine combinations of joint random variables that have identical mean return periods. These contours are calculated by transforming contours with a constant radius in uncorrelated standard normal space to the original joint random variable space using methods such as the Rosenblatt transformation (Rosenblatt, 1952). The mean return period, N, associated with each contour is calculated as:

$$N = \frac{1}{1-\Phi(\beta)} \quad (8)$$

where β is the radius of the circle in standard uncorrelated normal space from which the considered contour was transformed. IFORM is the recommended method in IEC 61400-3 to determine the 50-year joint wind wave design load case corresponding to operational conditions (i.e. power production conditions when the rotor is spinning), but not for the determination of extreme loads, because, as mentioned before, those are determined by considering the 50-year wind and 50-year wave as independent random variables. In this paper, we take a different approach than IFORM to determine wind and wave contours with constant mean return periods. In this approach, contours with constant joint probability of exceedance (i.e. contours with constant values of the joint cumulative distribution function) are calculated and the inverse of this joint probability of exceedance is the constant mean return period associated with the contour.

The shape and central values of the joint distribution of the wind and wave random variables, and, consequently, the shape and central values of the contours with constant mean return period change significantly based on the choice of the joint annual extreme values during a year. Unlike with scalar hazard intensities, it is not obvious which instant of yearly data corresponds to the annual extreme values. This is a consequence of the maximum wind speed not typically coinciding in time with the maximum wave height because in most hurricanes and other storms the maximum wave tends to lag behind the maximum wind speed. In the next section, the impact of two methods for determining the instant of the joint annual extreme values is assessed.

4 NUMERICAL EXAMPLE

In this section, the procedure, described in the previous section, is applied to 24 years of environmental data from a specific offshore NOAA recording station. Wind and wave contours with fixed mean return period are calculated using this data based on two methods for determining the instant of the joint annual extreme values. In the first method, referred to as Method 1, the joint annual extreme values are determined based on the maximum annual hourly wind speed (MAWS) and the joint annual significant wave height (JASWH), which is the significant wave height recorded during the same hour as the MAWS. In the second method, referred to as Method 2, the joint annual extreme values are determined based on the maximum annual significant wave height (MASWH) and the joint annual wind speed (JAWS), which is the wind speed recorded during the same hours as the MASWH. The data for this example are compiled from NOAA measurement station 41010, located 210 km off the Atlantic coast of Florida and specified in more detail in Table 1.

Table 1. Specification of selected station.

Latitude	28°54'22" N
Longitude	78°28'16" W
Distance to coast	210 km
Water Depth	872 m
Anemometer Height	5 m
Data time range	1988-2011

The hourly wind speed is measured at an elevation of 5 m above sea level. This measurement is converted to the wind speed at a standard elevation of 10 m using the logarithmic law equation (Simiu, 2011):

$$\frac{V(z)}{V(z_{ref})} = \frac{Ln\frac{z}{z_0}}{Ln\frac{z_{ref}}{z_0}} \quad (9)$$

where V(z) and $V(z_{ref})$ are wind velocity at elevations z and z_{ref}, respectively, and z_0 is the roughness length (z_0=0.002 for open water; Franklin, et al. 2001). Based on this model, the ratio of the wind speed at 10 m to that at 5 m is 1.09.

4.1 Marginal distribution of extreme values for wind and wave

The annual maximum wind and maximum significant wave height are not simultaneous for 23 out of the 24 years of data considered in this example. Of these 23 years, 13 years have annual maximum winds and waves from totally different storms. For the other 10 cases, the wave height lags behind the maximum wind by a duration ranging from 1 to 22 hours, an observation that is well-known (Morton, 1996). As a specific example, Figure 1 shows wind and wave measurements from 86 hours of data during Hurricane Wilma in October 2005. Red circles indicate simultaneous measurements corresponding to the maximum wind speed, while blue circles indicate simultaneous measurements corresponding to the maximum significant wave height. For this year, the annual maxima of both the wind and wave occurred during hurricane Wilma, and, as is common, the maximum wave height lagged behind the maximum wind speed. In this case, the lag was one hour.

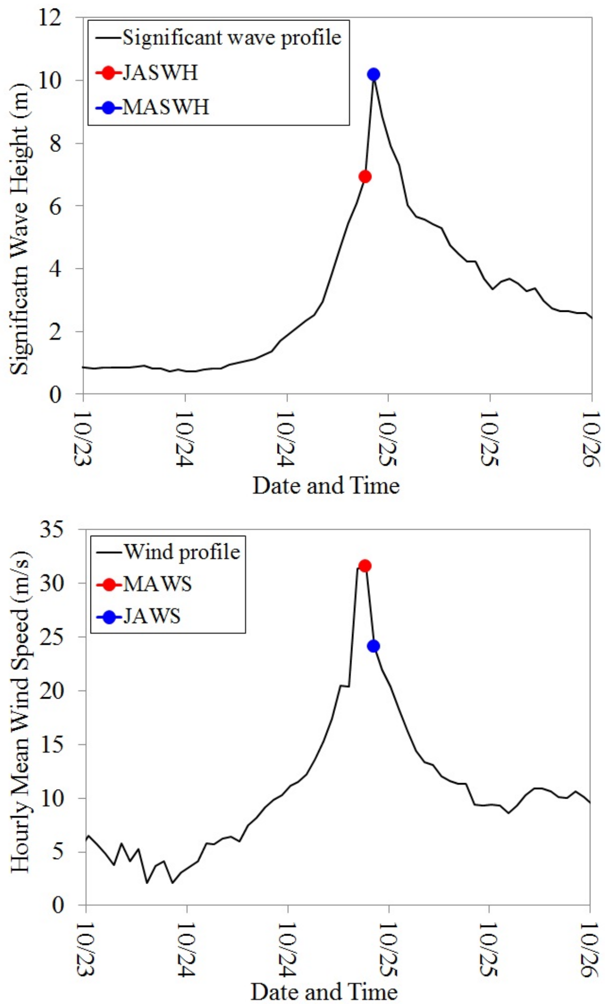

Figure 1. Hourly measurements of significant wave height (top) and maximum annual wind speed (bottom) during Hurricane Wilma in October 2005.

Marginal GEV distributions are fit to the annual maxima from the measurement stations. Figure 2 shows the best-fitting GEV marginal distributions for both methods of considering simultaneous data. Red circles and solid lines correspond to data and theoretical distributions based on maximum wind while blue circles and solid lines correspond to data and theoretical distributions based on maximum wave. The distributions are significantly different based on the method for filtering the data with the median wind speed changing by 22% (from 19.0 m/s based on maximum wind to 15.6 m/s based on maximum wave) and the median significant wave height changing by 41% (from 6.9 m based on maximum wave to 4.9 m based on maximum wind). Figure 2 also indicates the measurements corresponding to hurricanes and these measurements are all in the upper tail of the fitted distributions. All four distributions shown in Figure 2 passed a Kolmogrov-Smirnov statistical test at 0.05 significance, suggesting that the GEV distribution is an appropriate distribution for the set of extreme values considered here.

Table 2 provides the correlation coefficients between the wind speed and the significant wave height for both methods in for the original space, ρ, and transformed into Gaussian space, ρ′, where ρ′ is calculated by solution of Eq. 6.

Table 2. Correlation coefficients of wind speed and significant wave height.

	ρ	ρ'
Method 1	0.61	0.64
Method 2	0.66	0.65

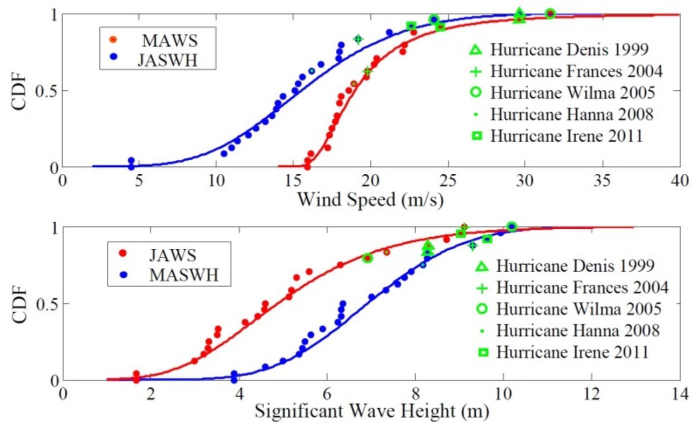

Figure 2. Annual extreme wind and wave data from 1988-2011 and the best-fitting GEV functions.

Following the Nataf model (Eqs. 2 to 7), contours of constant likelihood for the joint probability density function of significant wave height and wind speed are provided in Figure 3. In this figure, contours of constant likelihood based on the two methods for determining joint annual maximum values are superimposed on a scatter plot of the annual measurements. Measurements during hurricanes are indicated with green outlines.

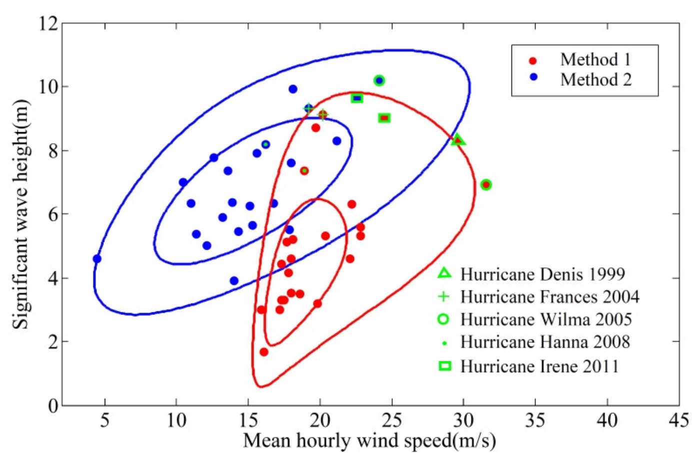

Figure 3. Contours of constant likelihood of the joint probability density function.

Based on the procedure outlined in the previous section for associating a mean return period with contours of wind and wave, Figure 4 presents contours corresponding to 50 and 100 year mean return periods for each of the two methods. As can be seen in the contours are significantly different for the two methods. The maximum wind speeds on the Method 1 contours provide the mean return period of wind speed when treated as an independent variable while the maximum wave height of the Method 2 contours provide the mean return period of the significant wave height when treated as independent variable. For the Method 1 and Method 2 100-year contours, these independent values (34.1 m/s for wind and 10.8 m for wave) are the basis for the combined wind and wave which API-2INT-MET would prescribe for the design of an offshore structure. Note that API prescribes a 0.95 reduction factor on these values, so, in practice, the design would be based on the 100-year independent wind and 0.95 times the 100-year independent wave or vice versa. As shown in the figure, the simultaneous occurrence of these magnitudes has a mean return period much greater than 100 years. For Method 1 the mean return period for the 100-year independent wind and 0.95 times the 100-year independent wave is 1500 years, while for Method 2 the return period is 2000 years. For the 100-year independent wave combined the 0.95 times the 100-year independent wind the return periods are 5,000 and 1100 years for Method 1 and 2 respectively.

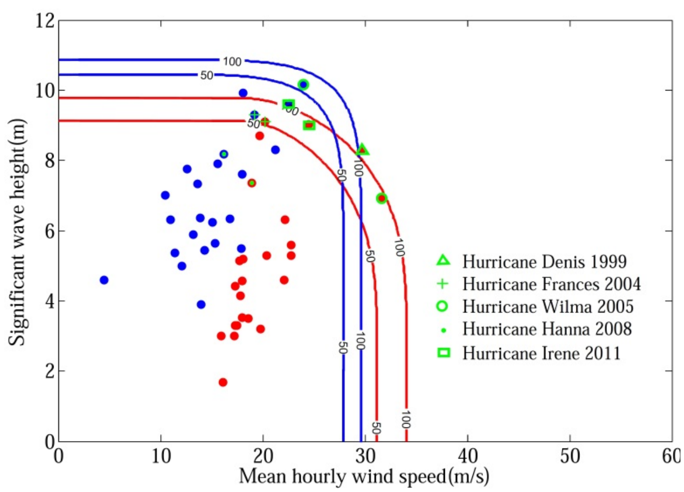

Figure 4. Wind and wave Contours of constant mean return period.

For this numerical example, the simultaneous values of wind and wave at a particular return period, determined with either Method 1 or Method 2, have intensities much less those for the same return period determined by treating wind and wave as independent variables. Moreover, it is also clear that the process for determining joint annual extreme values from simultaneous measurements is not obvious and has an important effect on the magnitudes of wind and wave for a particular mean return period. The differences highlight the coupling between the joint hazard and structural response. For example, an offshore structure with demands that are wind dominated should likely be designed for the hazard based on Method 1 whereas a structure with demands that are wave dominated should likely be designed for the hazard based on Method 2. For structures with demands that are more balanced between wind and wave loading, it’s possible that a third method that selects simultaneous annual maxima based on some maximum weighted average of wind and wave would be most appropriate.

5 CONCLUSIONS

The U.S. Atlantic coast is a prime location for potential installations of offshore wind turbines, but this area is also exposed to extreme wind and wave conditions induced by hurricanes. A better understanding of the extreme environmental conditions along the Atlantic coast would help to reduce the uncertainty currently present in offshore wind turbine design. Motivated by this need, this paper summarized existing methods for determining the design wind and wave conditions associated with the extreme loading during which the turbine rotor is parked and the blades are feathered. It is demonstrated that these existing methods recommend a conservative approach that overestimates the combined wind and wave intensity at a particular mean return period. The paper proposes a less conservative but more rational method, based on extreme value distributions of the wind and wave, the Nataf model and a prescribed joint exceedance condition, for estimating contours of wind and wave intensity with a constant mean return period. The paper calculated contours based on 24 years of hourly data measured at a NOAA buoy station located near the Atlantic coast of Florida and subjected to five hurricanes over a period of 24 years. The contours were calculated based on two different methods for selecting the joint annual extreme values for the wind and the wave. In the first method, the maximum annual hourly wind speed is paired with the significant wave height from that same hour while, in the second method, the maximum annual significant wave height is paired with the hourly wind speed from that same hour. Because maximum annual wave conditions rarely coincide with maximum annual wind conditions, these two methods result in contours with significantly different shape and central values.

6 ACKNOWLEDGEMENTS

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