

# CONSIDERATION OF HURRICANES AND TROPICAL CYCLONES IN THE DESIGN OF OFFSHORE WIND TURBINES

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## ABSTRACT

Variability in extreme loads is greater in areas in which hurricanes or tropical cyclones are likely. Therefore, it is expected that design standards for offshore structures in such areas should prescribe different load amplification factors if a uniform structural reliability is to be achieved at all locations. In this paper we examine and evaluate the adaptation of methods within the American Petroleum Institutions Recommended Practice (API RP 2A) that considers loads from hurricane conditions for the design of offshore structures for possible use in the International Electrotechnical Commission's standard, IEC 61400-3. Specifically, we address a method, termed the "Robustness Requirement" wherein load factors vary with both location and structural type and are calculated based on the ratio between a load effect calculated for design-level external environmental conditions and a load effect calculated for more extreme conditions. The impact of this method is assessed by comparing estimates for the weight of an offshore wind turbine designed based on this method with those based on the current edition of IEC 61400-3. Studies are carried out on the 5 MW offshore baseline wind turbine, developed by the National Renewable Energy Laboratory, and modeled using the United States National Renewable Energy Laboratory's aeroelastic simulation code, FAST.

## INTRODUCTION

Current international standards for the design of offshore wind turbines (OWTs) do not explicitly consider loading under hurricane or tropical cyclone conditions despite evidence that wind turbine structures are vulnerable to such loading (Bower & Mason 2011). The appropriateness of these standards to offshore wind turbines installed along the Atlantic and Gulf coasts of the United States and other regions of the world which are subject to tropical cyclones is questionable (Clausen 2007, Jha et al 2010). It is widely agreed that variability in extreme loads is greater in such regions. Therefore, the notion of hurricane- and site-specific design procedures and load factors is gaining attention in the offshore wind turbine engineering community, and an important step is to evaluate the effect of different design approaches (Dolan et al 2009). One goal of such approaches would be to harmonize the achieved structural reliability of offshore wind turbine support structures across the range of sites and structural types where development is likely.

This paper addresses the design of monopile support structures for OWTs at sites along the Atlantic coast of the USA by (1) evaluating the joint wind and wave hazard, (2) computing the ratio of the 500 and 50 year structural demands to arrive at a load factor based on the so-called API robustness requirement used in oil and gas platform design, (3) designing monopile support structures for the NREL 5MW offshore turbine at each of the sites using both IEC load factors (IEC 2005) and robustness requirement-based load factors while also considering stiffness considerations for resonance avoidance and avoidance of excessive rotation at the mudline, and (4) comparing the designs obtained from the IEC and API-robustness approaches in terms of total mass of steel used in the support structure,

The design of OWT support structures along the hurricane-prone US Atlantic coast can draw on the extensive knowledge bases developed by the oil and gas industry for design of platforms in the hurricane-prone Gulf of Mexico and by the wind energy community in Europe where extensive offshore deployment has occurred. Nevertheless, the situation along the US Atlantic coast is unique and new developments are necessitated by the significant structural differences between oil and gas platforms and OWT support structures. In summary this paper represents an initial attempt to translate some of the oil and gas knowledge base regarding partial load factor determination into the OWT domain by considering sites for likely OWT development and the specific structural characteristics of OWT support structures.

## **BACKGROUND**

Design standards based on load and resistance factor design philosophy have many attractive features, including the flexibility to independently prescribe amplification factors to different load types. This approach enables load effects which are more variable to be amplified more than load effects which are less variable. The objective of this approach is that each load type be considered in a consistent manner that results in a uniform structural reliability, regardless of the location or type of the structure.

Currently, the most comprehensive and widely-used design standard for offshore wind turbines is IEC 61400-3 (2005). This standard requires that offshore wind turbines be designed to resist extreme environmental conditions with a mean return period (MRP) of 50 years. The effects of these loads on the structure (e.g. the mudline moment and shear for a monopile foundation) are amplified by a factor equal to 1.35 to achieve an acceptable balance between risk tolerance and economic viability. The IEC 61400-3 standard has been designed to be compatible with the European offshore environment, and the writing committee for this standard is currently assessing the viability of applying this standard to the offshore environment in the U.S. One major difference between the European and U.S. offshore environments is the presence of considerable risk of hurricanes in the U.S. environment. It is widely agreed that variability in extreme loads is greater in areas exposed to hurricane risk, and therefore, it is possible that design standards for offshore structures in such areas should prescribe a higher load amplification factor if a uniform structural reliability is to be achieved at all locations.

In the U.S., there are currently no offshore wind turbines, however there are an enormous number of offshore oil and gas structures. These structures are typically designed in accordance with API RP 2A (2005) which, for medium consequence of failure structures, requires that the design consider the effect of extreme environmental loads amplified by a factor that is calculated on a site- and structure-specific basis. The concept, termed the "Robustness Requirement," requires that structures be designed to resist load effects under 50-year extreme environmental conditions amplified by the ratio of the load effect estimated at 500-year conditions to that at 50-year conditions. Sites and structures that have high load effect variability will then have higher load amplification factors, but similar overall structural reliability as sites and structures with lower load effect variability and lower amplification factors.

To provide some insight into the differences between the IEC and API approaches and the potential of each approach to prescribe a uniform structural reliability for offshore wind turbines installed along the U.S. coast, monopiles, supporting the NREL 5MW offshore baseline turbine, have been designed for four locations along the U.S. Atlantic Coast. Designs at each location are conducted twice, once based on the IEC load factor of 1.35 and once based on the API Robustness Requirement.

## **SITE INFORMATION**

Four sites, along the U.S. coast and exposed to hurricane risk, are considered in this paper. The sites are selected based on a combination of geographic features and the availability of metocean data. Specifically regarding geographic criteria, sites have been selected along the Atlantic Coast of the USA with added attention being given to the mid-Atlantic and Northeastern coasts where the wind resource is greater and which are targeted by the majority of current proposals for offshore wind energy development in the USA. Regarding data availability, sites have been selected to correspond to the location of metocean data buoys deployed and maintained by National Oceanic and Atmospheric Administration (NOAA) that have at least 10 years of data available or in one case to correspond to a site described in (MMI, 2009) based on metocean data obtained from Oceanweather Inc. Given these considerations and the desire to somewhat limit the scope of the paper, four sites have been selected that lie off the coasts of the states of Maine, Massachusetts, Delaware, and Georgia (Figure 1). In the remainder of this paper the sites are identified by their two letter postal abbreviation codes (ME, MA, DE, and GA).

Table 1 gives the general characteristics of the sites (shown in Figure 1) including their latitude and longitude, distance from shore, water depth, and, for the NOAA sites, the site identifier. The sites have water depths ranging from 15m to 30m which covers the range of moderate to large depths for which monopile support structures would be expected to be suitable. With the exception of the ME site the locations are all 20-30 km offshore, with the much closer ME site being reflective of the steeper bathymetry of coastal Maine when compared with the remainder of the Atlantic coast.

**Table 1. Site information.**

Site	Abbrev	NOAA ID	Lat	Long	Water Depth (m)	Dist. to Shore (km)
Maine	ME	44007	43.53 N	70.14 W	23.7	5.60
Massachusetts	MA	N/A	41.25 N	71.25 W	15.0	22.5
Delaware	DE	44009	38.46 N	74.70 W	30.5	30.3
Georgia	GA	41008	31.40 N	80.87 W	19.5	32.3

At the NOAA sites (ME, DE, GA), the data used in this paper consists of the wind speed measured at 5m above sea level and the significant wave height, defined as usual to be the average of the top one third of recorded wave heights in a given time interval. Wind speed measurements reflect the 8 minute average wind speed reported hourly and the significant wave heights are determined based on a 20 minute time interval and are also reported hourly. Before applying the wind data to OWT design, therefore, corrections must be made to account for the higher elevation above sea level of the rotor hub and the different averaging periods specified by the relevant design standards.



**Figure 1. Site locations and an image of a NOAA buoy (Image Ref: NOAA).**

At the MA site metocean conditions were gleaned from (MMI, 2009) which in turn were based on hindcast simulations conducted by Oceanweather Inc. that provide hourly average wind speeds at 10m elevation and significant wave height. The data reported in (MMI, 2009) come in two forms: a ‘tropical cyclone’ (TC) data set that is based on 100 years of tropical storms and a ‘continuous’ (C) data set that is based on 20 years of continuous data. The continuous data set can be considered equivalent in form and content to the data obtained for the NOAA sites.

**Table 2. Observation length<sup>1</sup> and general metocean conditions<sup>2</sup> given in terms of 50 year return period conditions.**

Site	Observation Length (yrs)	50 yr Vw (m/s)	50 yr Hs (m)
ME	31	25.8	10.0
MA (TC)	100	34.0	8.5
MA (C)	20	28.0	6.3
DE	27	28.1	8.1
GA	20	26.0	5.7

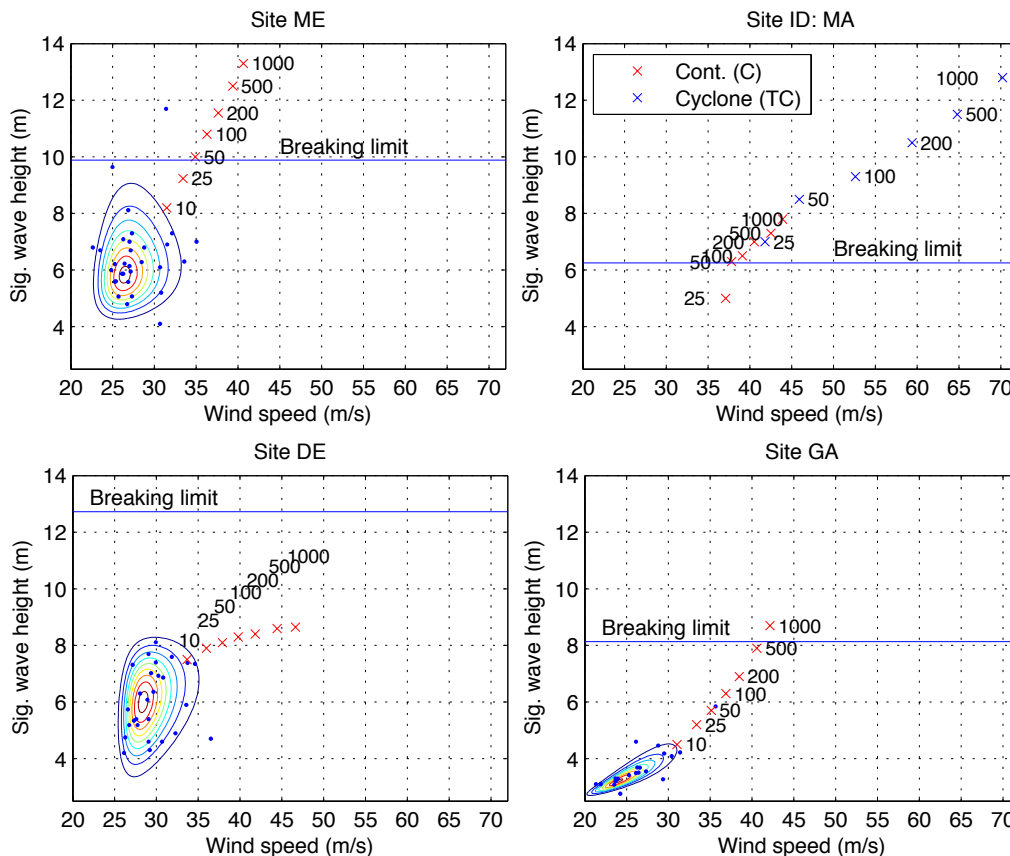
1. Observation length for MA is approximately 100 years and 20 years for tropical cyclone (TC) and continuous (C) data, respectively. MA data are based on hindcast stochastic storm simulations.
2. Wind speeds are hourly averages given at 10m elevation. Data for ME, DE, and GA sites have been adjusted from the 5m elevation of the NOAA data buoys at those sites.

### METOCEAN HAZARD AND SENSITIVITY

Design and risk assessment for OWTs depends on a robust model for the environmental loadings, or hazard. In this case the hazard consists of the wind speed and wave height, which are the two primary sources of structural loading—other sources that are neglected here are current, marine growth, thermal gradients, tide and storm surge. As described in the previous section, the metocean conditions at the ME, DE and GA sites are characterized by 20-30 years of continuous measurement obtained from the NOAA data buoy database and the MA site has two metocean characterizations obtained from simulated hindcast data. For the MA site, the value of the wind speed and wave height at mean return periods

(MRPs) up to 1000 years is provided directly in (MMI, 2009), and those values have been used here. For the NOAA sites, the buoy data have been used to fit a joint pdf for the wind speed and wave height and MRPs up to 1000 years have been extracted from the upper tails of the best fit distributions—in this case a generalized extreme value distribution has been used as the model distribution for each of the wind and wave hazard intensities at the NOAA sites.

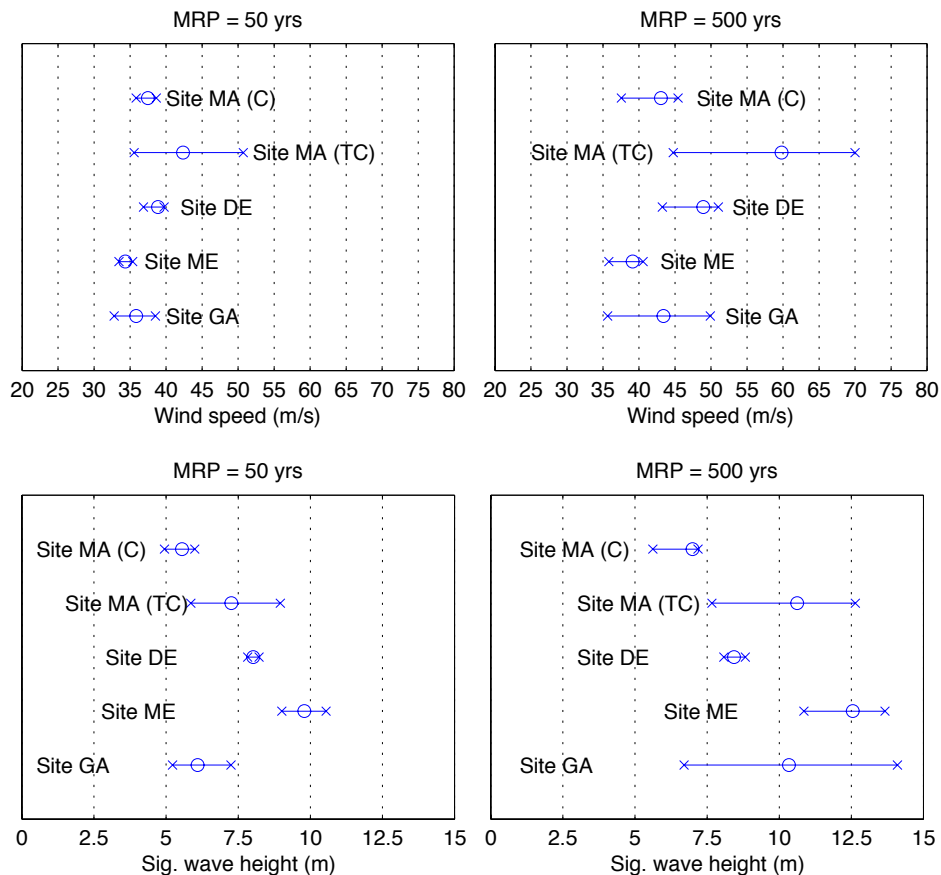
Figure 2 shows the joint wind/wave pdfs for the three NOAA sites and the corresponding MRP values. For the MA site only the joint MRP values are shown—one set for the tropical cyclone (TC) data and one for the continuous (C) data. Noting that the axis limits for all subfigures are the same it is readily apparent the large degree to which the wind/wave joint hazard is site-specific. Furthermore, the MA data show that developing joint hazard models based on relatively short duration continuous data can drastically underestimate the hazard when compared to models that are calibrated only to tropical cyclone events over a longer observation period. Furthermore, the pattern formed by the MRP data in the plots (the red x marks) provides information about the relative increase in the wind and wave hazard at long MRPs. For example, the MRP points at the ME site fall along a more or less straight line, indicating that the wind and wave hazard intensity increase proportionally as the MRP increases. On the other hand, at the DE site, the wave hazard appears to saturate and, at longer MRPs, the wind hazard intensity dominates. The opposite is true at the GA site. This added element of site specificity could have significant implications for design and risk assessment since wind and wave actions are expected to affect different OWT support structures in substantially different ways.



**Figure 2. Joint wind/wave hazard for selected sites. For the NOAA sites (ME, DE, GA) the joint pdf contours are shown along with the MRP values, while for the MA site, MRP values are shown for the tropical cyclone and continuous datasets. Access was not available to the full dataset at the MA site for estimation of the joint pdf. Wind speeds shown are 10 min average at hub-height, adjusted from actual measurement intervals and elevations. Data points are shown as a scatter plot superimposed on the contours, and a breaking wave limit is also indicated.**

Also shown in Figure 2 is an indication of the breaking wave limit for the particular sites. The breaking wave limit as shown in the figures has been calculated by assuming an extreme wave that has amplitude  $H_{\text{extreme}} = 1.87H_s$  and that the extreme wave breaks when  $H_{\text{extreme}}/D > 0.78$ , that is, when the ratio of the extreme wave height to the water depth,  $D$ , exceeds 0.78. Therefore the limits shown in Figure 2 are placed at a significant wave height of  $H_{s,\text{breaking}} = 0.78D/1.87$ . For the ME, MA (C) and MA (TC) locations the breaking limit is lower than the 50 year wave. Since the breaking limit imposes, to some degree, a limit on the height of waves that can impact the structure, the breaking limit indicates the wave height after which further increases in wave intensity may have little added effect on the structural demands. That is, if a hazard model based on a pdf for wave height that is based on non-breaking waves is used to extrapolate wave height at long return periods, the breaking limit sets something akin to a maximum physically possible wave height. Specifically for the ME site, then, imposing a breaking limit that is slightly below the 50 year wave height means that the structural loads due to the 50 year and all longer MRP waves will be essentially the same.

Another feature of the hazard model that is apparent from Figure 2 is that the hazard intensity at MRPs used in design (50-100 years) and risk analysis (500-1000 years and possibly beyond) lie far into the upper tails of the wind speed and wave height distributions. Therefore the estimates of the wind speed and wave height intensities at those MRPs based only on 20-30 years of continuous data have a high level of uncertainty. This uncertainty has been quantified, as shown in Figure 3, using the following procedure (described here in terms of the wind speed intensity, but performed identically for the wave height): (1) fit a GEV distribution to the data, consisting of  $n$  observations, (2) sample  $n$  realizations from the distribution fit in (1), (3) fit a new GEV distribution to the  $n$  points sampled in (2), (4) compute values of the wind speed intensity at desired MRP values based on the GEV fit in (3), and (5) repeat steps 2-4 20 times and estimate the 25% and 75% values for estimates of the wind speed at the desired MRP.



**Figure 3. Variability of estimates of wind speed and wave intensity at MRPs of 50 and 500 years. Circles show mean value of twenty samples and x markers indicate 25% and 75% values.**

Although this procedure, which is somewhat akin to estimation of bootstrap quantiles, does not provide rigorously defined confidence intervals on the estimates it does provide a sense of the range of values possible. Figure 3 shows that this range is large indeed, and that at both the 50 and 500 year MRPs there is significant overlap among the ranges of hazard intensities at the different sites. Despite this finding, which indicates that site specificity in the hazard intensity at MRPs of interest may not be statistically significant, the tremendous difference in the joint pdf contours (Figure 2) provide evidence that the lack of statistical significance for the site specificity is primarily due to small sample size rather than an underlying statistical similarity among the sites.

## DESIGN METHODOLOGY

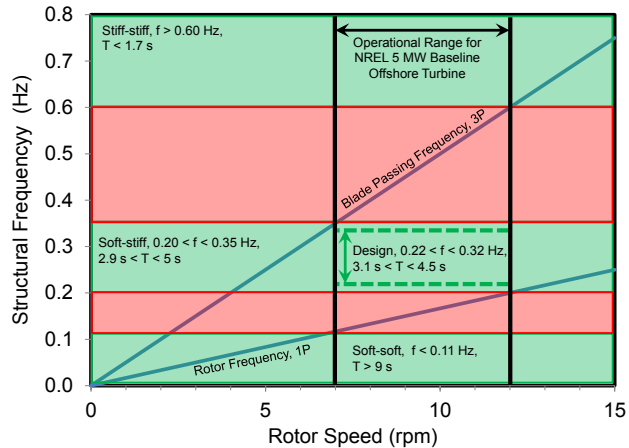
For each of the considered sites, monopiles supporting the NREL 5 MW baseline offshore turbine were designed to meet three design criteria: (1) avoidance of resonance with operational frequencies, (2) ultimate mudline bending and (3) minimum mudline stiffness, each of which is explained in more detail below. Basic specifications of the NREL 5MW turbine are provided in Table 3 with more detail available in (NREL, 2009). Each site is designed based on two methods, the first is based on amplification of extreme environmental moment and shear demands per the IEC 61400-3 (2005) load factor of 1.35 and, the second is based on amplification of these same demands, per the API Robustness Requirement, described previously.

**Table 3. Specifications of the NREL 5 MW baseline offshore wind turbine.**

Parameter	Value
Power Rating (MW)	5
Rotor Diameter (m)	126
Hub Height above MSL (m)	90
Cut-in, Rated, Cut-out Wind Speeds (m/s)	3.0, 11.4, 25.0
Cut-in, Rated Rotor Speeds (rpm)	6.9, 12.0
Tower Diameter Base, Top (m)	6.0, 3.9 (linear variation)
Tower Thickness Base, Top (mm)	35, 25 (linear variation)

Resonance Avoidance. This condition ensures that the natural frequency of the entire turbine structural system, comprising the rotor nacelle assembly, the tower, the monopile and the soil, is outside the range of the turbine's operational frequencies. The operational frequencies considered are the rotor frequency (i.e. the 1P frequency, the number of rotations per minute of the rotor) and the blade passing frequency (i.e. the 3P frequency for a three-bladed turbine such as that considered here). Both the 1P and 3P frequencies vary with the wind speed, and so the range of frequencies which are avoided are calculated for the operational range of rotor frequencies between the cut-in and cut-out wind speeds for the turbine. This requirement creates three ranges of acceptable structural frequencies, the soft-soft range, which ensures that the structural frequency is always less than the 1P frequency, the stiff-stiff range, which ensures that the structural frequency is always greater than the 3P frequency, and the soft-stiff range, which ensures that the structural frequency is between the 1P and 3P frequencies. It is generally not practical to design a turbine for the soft-soft or stiff-stiff regions, so, for this study, the structural frequency of the turbine is designed to fall within the soft-stiff region. Moreover, the boundaries of the soft-stiff region are restricted by 10%, as recommended by ASCE/AWEA (2011), to account for both tolerances in the design assumptions and for an appropriately safe separation distance. These requirements are conveniently visualized in a so-called Campbell Diagram, which has been provided in Figure 4 for the NREL 5 MW baseline offshore turbine. As shown in the figure, the design requirement for this turbine is that the natural frequency of the turbine system be between 0.22 and 0.32 Hz.

The natural frequency of the turbine system is calculated by an eigenvalue analysis. The tower and monopile are discretized into elements with distributed mass lumped at the nodes and an additional concentrated mass at the top of the tower to represent the rotor nacelle assembly. Soil-pile stiffness is modeled using P-Y springs with stiffness defined by the initial, linear stiffness of the API P-Y curves (API, 2005). The soil spring stiffness is calculated assuming the soil is a uniform deposit of medium dense to dense sand with a friction angle of  $40^\circ$ , a relative density of 0.55, a submerged soil unit weight of  $10 \text{ kN/m}^3$  and an initial modulus of subgrade reaction of  $20.8 \text{ MPa/m}$ . The base of the monopile is modeled with a roller support.



**Figure 4. Campbell diagram for the NREL 5 MW baseline turbine ( $f$  = structural frequency.  $T$  = structural period).**

Ultimate Bending and Minimum Stiffness at the Mudline. This study assumes that the moment and shear demands at the mudline control the strength and stiffness requirements of the monopile. Based on this assumption, the second and third design criteria require the calculation of a design moment and shear at the mudline for extreme environmental conditions. Design mudline moment and shear demands were determined as the more severe of two IEC 61400-3 (2005) design load cases (DLC), DLC 6.1b and DLC 6.1c. Both load cases consider the simultaneous and aligned loading of a steady wind and a single deterministic wave under parked and feathered conditions with the rotor plane oriented perpendicular to the wind and wave directions. The first load case (DLC 6.1b) considers an extreme wind, defined as 1.4 times the 50-year, 10-minute averaged wind speed at the hub height, combined with a reduced wave height, defined as 1.3 times the 50-year significant wave height, while the second load case (DLC 6.1c) considers a reduced wind, defined as 1.1 times the 50-year, 10-minute averaged wind speed at the hub height, combined with an extreme wave height, defined as 1.87 times the 50-year significant wave height. The period of both the extreme wave height and the reduced wave height is calculated based on the lower bound of the range of wave periods required to be considered by IEC 61400-3. Thus, the wave period is set equal to  $11.1(H/g)^{1/2}$ , where  $H$  is the wave height (either extreme or reduced) and  $g$  is gravity. For the turbines considered in this study, the lower bound of the wave period range provided by IEC always resulted in the largest moment and shear demands at the mudline. Moreover, DLC 6.1c always resulted in larger moment and shear demands than DLC 6.1b.

Up until the breaking wave limit, which was assumed for all sites to be at a wave height to depth ratio of 0.78, the kinematics of the waves are modeled nonlinearly using stream function theory developed by Dean (1965) and a numerical solution method developed by Fenton (1988). The kinematics calculated based on the method of Fenton were interpolated into a uniform grid that was used to generate a user-defined wave input in FAST, an open source program developed by NREL for the analysis of wind turbines. For wave height to depth ratios greater than 0.65, the numerical solution developed by Fenton becomes numerically unstable. For cases where the wave height to depth ratio exceeds 0.65 but doesn't exceed the breaking wave limit of 0.78, we artificially cap the wave height at a height to depth ratio of 0.65. For waves that exceed the breaking wave limit, the kinematics are modeled by adding "equivalent accelerations" to the accelerations developed by the stream function theory for a wave with height equal to the breaking wave limit. Slam forces from the breaking waves were calculated according to the model proposed by Wienke (2005), which specifies the duration and magnitude of the slam forces for a breaking wave. The slam forces are converted to "equivalent accelerations" using the Morison equation for a cylinder. These "equivalent accelerations" provide inertial forces that are identical to the magnitude and duration of the slam forces as predicted by the Wienke model. The "equivalent accelerations" are then added to the non-breaking accelerations and a user-defined wave input for FAST is created. The resultant sum of the accelerations produces inertial plus slam forces that are equivalent to the



combination of inertial forces from a nonlinear stream function wave and the slam forces from a breaking wave modeled per the Wienke model.

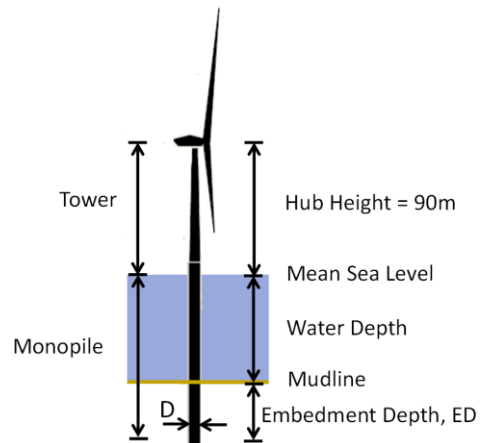
For all conditions, the aerodynamic and hydrodynamic loads are calculated based on a static structural model. The effect of structural dynamics is considered by amplifying nonbreaking wave loads based on the ratio of the wave period to the structural period and by de-amplifying breaking wave loads based on the ratio of the structural period to the breaking wave duration. For the considered ranges of structural periods and non-breaking wave periods, the dynamic amplification factor is, on average, equal to 1.2. For the considered ranges of structural periods and breaking wave durations, the dynamic amplification factor is, on average, equal to 0.2. This large deamplification of the static loads is a result of the short duration (around 0.1 seconds for all considered breaking waves) of the breaking wave slam loads according to the Wienke model. The combined bending and shear capacity of the monopile is calculated per ISO 19902.

The minimum mudline stiffness is a serviceability requirement that limits the mudline rotation of a monopile under extreme environmental loads to 0.25 degrees. This rotation is the maximum rotation permitted by the turbine manufacturer. Mudline rotation has been assessed using a nonlinear p-y Winkler spring type model to compute the deformation of the soil-pile system under design loads. A total of 20 soil springs has been used, evenly spaced along the length of the pile and nonlinear solution of the pile deformation has been performed using simple step integration of the load deformation response with the soil spring stiffness at each load step calculated as the tangent stiffness of the corresponding p-y curve.

## RESULTS

Before presenting the results, it is important to emphasize a few of the simplifying assumptions of the design methodology described above. First, the methodology assumes that ultimate mudline bending demand and maximum mudline rotation for design is caused by DLC 6.1b (extreme steady wind model + reduced wave height) or DLC 6.1c (reduced steady wind model and extreme wave height), ignoring the possibility that larger demands could occur during operational conditions, when environmental conditions are much less severe, but the average drag coefficient of the rotor is much larger. Second, loads are calculated for a static model and dynamic effects of the wave loading are considered through an amplification factor based on the ratio of wave period/duration to the structural period. Third, non-breaking waves are capped at a wave height to depth ratio of 0.65 because estimation of the kinematics of waves with heights greater than this limit were found to be numerically unstable. Finally, the design methodology completely ignores any design issues related to constructability (for example monopile dimensions which are too slender to be installed are not excluded).

Following the methodology described in the previous section, ten monopiles were designed, two each for the three NOAA sites and four for the MA site (two based on the TC data and two based on the C data). The first design is based on amplification of extreme environmental demands per a load factor of 1.35, as specified in IEC 61400-3 and, the second is based on amplification of these same demands, per the API Robustness Requirement. Each monopile is designed as a circular, hollow cross-section with constant diameter and thickness. The considered design parameters of the monopiles include the diameter  $D$ , thickness  $t$  and embedment depth  $ED$ , as shown schematically in Fig 5. The range of considered design parameters is:  $D$  from 4.5 to 7.0m in increments of 0.5m,  $t$  from 30 to 100mm in increments of 10mm, and  $ED$  from 20 to 45m in increments of 5m. The combination of these parameters that is selected for the design is that which satisfies the three design criteria described in the previous section and has the lightest mass. The selected design for each of the ten monopiles is presented in Table 4.



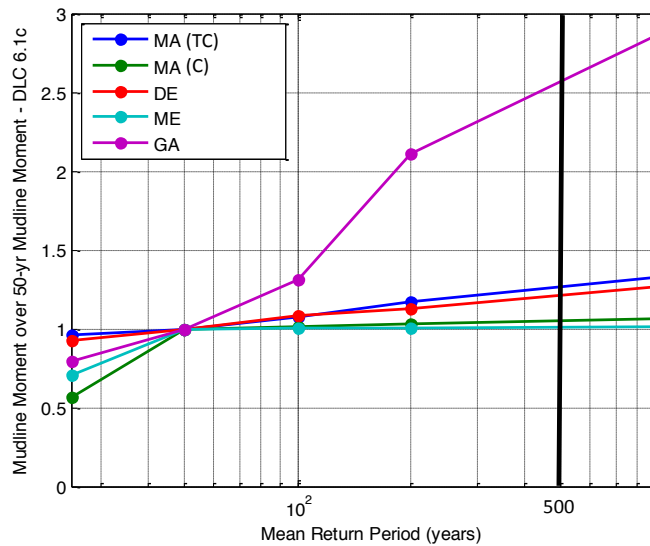
**Figure 5. Schematic representation of the NREL 5MW turbine. The design of the monopile considers three parameters, diameter  $D$ , embedment depth  $ED$ , and thickness  $t$  (Image modified from MMI, 2009).**



**Table 4. Monopile designs and resulting mass for five sites and two different load factors.**

Site ID	IEC Load Factor = 1.35				API Load Factor = Robustness Requirement			
	D (m)	t (mm)	ED (m)	Mass (kg)	D (m)	t (mm)	ED (m)	Mass (kg)
MA (TC)	5.5	40	25	9.3e5	5.5	40	25	9.3e5
MA (C)	5.5	40	25	9.3e5	5.5	40	25	9.3e5
DE	7.0	40	25	1.1e6	7.0	40	25	1.1e6
ME	7.5	80	35	1.6e6	7.0	60	30	1.3e6
GA	6.0	40	25	9.8e6	6.0	40	25	9.8e6

All designs, with the exception of the ME site, are controlled by resonance avoidance and, for these sites, there is no difference between the designs with different load factors. The ME site is an exception because the wave loading is the most severe due to a combination of relatively deeper water and the presence of breaking waves at the 50-year mean return period. For the ME site, the design based on the API robustness requirement is nearly 20% lighter than that based on load factor of 1.35. As seen in Figure 6, this is because the load factor due to the robustness requirement is less than 1.35. Since waves at the ME site are already breaking at a 50 year return period and since the impact load of breaking waves, as modeled here, is a constant value and since the demands due to waves are much greater than those due to wind, the ratio between the demand at a 500 year return period and the demand at a 50 year return period is barely greater than 1.0. Identical reasoning can explain why low ratios for the robustness requirement are observed at all of the sites where waves break at a 50 year return period (ME, MA-C and MA-TC). Low ratios of the robustness requirement are also observed at the DE site because the wave hazard at this site increases negligibly with increasing MRP, see Figure 2. For the GA site, the robustness requirement results in a large load factor of 2.6. This is because, at this site, waves break at the 500 year MRP resulting in a large difference between the moment at 500 years and that at 50 years. Had the design at this site not been controlled by resonance avoidance, then the robustness requirement would have required a much heavier monopile than that designed with a load factor of 1.35.



**Figure 6. Mudline moment, calculated based on DLC 6.1c, at various MRPs, divided by the mudline moment at a 50 yr MRP for monopile designed at each site per API (see Table 4).**

## CONCLUSIONS

This paper has described an investigation into two different approaches to determining the proper load factors for the design of offshore wind turbine support structures, specifically monopiles. The two approaches are to use the 1.35 load factor in IEC 61400-3 or to determine a site-specific load factor using the API Robustness Requirement that defines the load factor as the ratio of the demands generated by the 500 and 50 year metocean conditions. Design of the monopile has been conducted with respect to the criteria of resonance avoidance, bending capacity at mudline, and permanent rotational drift at

mudline, and designs have been made for four sites off the Atlantic coast of the USA. It was found that at every site the Robustness Requirement resulted in load factors that differed greatly from the IEC value of 1.35. In most cases the robustness criterion load factor was actually lower than 1.35 due to the presence of the breaking wave limit at a wave height lower than the 50 year storm, meaning that minimal increase in wave load occurred between the 50 and 500 year storms. One of the major conclusions of this paper is that when examining loading at expected mean return periods of 50 years and greater the possibility for breaking waves at sites of likely monopile development is great, and that the design loads are highly sensitive to the presence or absence of breaking waves. Due to the great variability in the literature in how to determine the breaking wave limit and how to calculate loads from breaking waves, the issue of loading on OWT support structures by breaking waves is in need of substantial further inquiry. Another important finding of this paper is that, at all but one of the sites, the design of the monopile was governed by the stiffness required to avoid resonance with the operational frequencies of the turbine. At sites where resonance avoidance determines the design the choice of load factor does not affect the final design. These conclusions are based on a study that considered only monopile support structures, and they could very well be different for other support structures such as jackets or tripods or guyed monopiles.

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## **REFERENCES**

- API (2005). "API RP 2A-WSD: Recommended Practice for Planning, Designing and constructing Fixed Offshore Platforms-Working Stress Design," American Petroleum Institute.
- ASCE/AWEA (2011). "Recommended Practice for Compliance of Large Land-based Wind Turbine Support Structures." AWEA Washington, DC and ASCE Reston, VA.
- Bowater, D., Mason, R. (2011). "Blown away: Gales Wreck Wind Turbines as Scottish Storm Wreak Havoc." The Telegraph (UK), Dec. 9, 2011 (accessed January 2012).
- Clausen, N. (2007). "Wind farms in regions exposed to tropical cyclones." European Wind Energy Conference, Athens, Greece.
- Dean, R.G. (1965). "Stream Function Representation of Nonlinear Ocean Waves." Journal of Geophysical Research. 4561-4572.
- Dolan, D., A. Jha, T. Gur, S. Soyoz, C. Alpdogan, and T. Camp. (2009). "Comparative study of offshore wind turbine standards." MMI Engineering, MMS Project No. MMW528 June 2009.
- Fenton, J.D. (1988). "The Numerical Solution of Steady Water Wave Problems." Computers and Geosciences, 357-368.
- IEC. (2005). "IEC 61400-1 Ed. 3: wind turbines - Part 1: design requirements." International Electrotechnical Commission, Geneva, Switzerland.
- Jha, A., Dolan, D., Musial, W. and Smith, C. (2010). "On hurricane risk to offshore wind turbines in US Waters." Proceedings from the 2010 Offshore Technology Conference, Houston, TX, 2010.
- Jonkman, J., Butterfield, S., Musial, W., and Scott, G. (2009). "Definition of a 5-MW Reference Wind Turbine for Offshore System Development." National Renewable Energy Laboratory, U.S. Dept. of Energy.
- Wienke, J and Oumeraci, H. (2005). "Breaking Wave Impact Force on a Vertical and Inclined Slender Pile – Theoretical and Large-scale Model Investigations." Coastal Engineering. 435-462.