



Strength, stiffness, resonance and the design of offshore wind turbine monopiles



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ABSTRACT

A monopile supporting an offshore wind turbine (OWT) is currently designed for both strength and stiffness. Regarding strength, the monopile is designed to have sufficient capacity to withstand demands under both 50-year operational conditions, when the rotor is spinning and blades are oriented to optimize power generation, and 50-year extreme conditions, when the rotor is parked and the blades feathered to minimize aerodynamic loads. Regarding stiffness, the monopile is designed to have sufficient stiffness such that the first structural frequency of the OWT falls between the 1P and 3P frequencies (rotation frequency and blade passing frequency for a three-bladed turbine). For six case studies, including three sites along the U.S. Atlantic coast and two mudline conditions (fixed and compliant), this paper delineates the conditions under which stiffness and strength govern the design of the monopile. This distinction has important implications for the overall risk profile of an OWT, as monopiles controlled by stiffness will have more reserve capacity than monopiles controlled by strength. The six case studies are intended to consider a range of water depths, metocean environments and mudline conditions that is representative of conditions suitable for installing OWTs supported by monopiles along the U.S. Atlantic coast. The monopile designs are controlled by stiffness for two of the six cases studied and, for these two cases, a modest (6–8%) reduction in monopile area (and mass) could be achieved if dynamic design requirements were achieved through means other than increasing monopile stiffness. Monopile designs for the remaining four cases are controlled by operational moment demands.

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1. Introduction

This paper attempts to delineate the conditions under which resonance avoidance (i.e. stiffness) and strength govern the design of the offshore wind turbine (OWT) monopile, using an idealized utility scale 5 MW wind turbine as an example. Since the mass of a turbine is fixed and since the mass and stiffness of the support structure cannot be treated as independent design parameters, the resonance avoidance condition is satisfied primarily by designing the support structure stiffness such that the first structural frequency of the OWT is between the 1P and 3P frequencies (rotation frequency and blade passing frequency for a three-bladed turbine) [1], and preferably also significantly above the peak spectral content of the wind and wave loading frequencies. Although the character of the dominant design criterion is not particularly important

for the specification-based design of the support structure, it has important implications for the overall risk profile of an OWT. For example, if the support structure design is driven by stiffness considerations, it may have significant reserve capacity at the design loads (typically related to the 50-year conditions) and therefore a substantially lower risk profile with respect to more extreme events than a similar structure controlled by strength considerations. While the overall risk profile of an OWT does not directly affect design, it does have meaningful implications for financing, underwriting, and regional and national scale energy security planning.

The support structure of an OWT extends from the bottom of the foundation, which is embedded below the mudline, to the hub of the turbine. Offshore, the design of the support structure takes on added importance because of the additional total structural height from mudline when compared with height above land for onshore turbines, the greater uncertainty in soil conditions [2,3], and the additional loading induced by the sea state particularly for extreme storms such as hurricanes. The complexity of the OWT structural system—soil conditions, foundation, support

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Nomenclature

1P	rotor frequency	$f_{X(x)}$	probability density function of random variable X evaluated at $X = x$
3P	blade passing frequency for a three-bladed turbine	f_{n1}	first natural frequency of structure (Hz)
A	cross sectional area of monopile (m^2)	g	gravitational acceleration (m/s^2)
D	diameter of monopile (m)	t	thickness of monopile (m)
$F_X(x)$	cumulative distribution function of random variable X evaluated at $X = x$	u_1, u_2	standard normal random variables
GEV	generalized extreme value distribution	Φ	standard normal cumulative distribution function
H_s	significant wave height (m)	β	radius of circle in standard normal space
I	turbulence intensity	κ	shape parameter of GEV distribution
MRP	mean recurrence period (year)	μ	location parameter of GEV distribution
OWT	offshore wind turbine	σ	scale parameter of GEV distribution
T_p	peak spectral period (s)		
V	wind velocity, hourly at hub height, 90 m above mean sea level (m/s)		

structure, hydrodynamic loads, operational loads, aerodynamic loads—means that a myriad of design load cases (DLCs) and design objectives must be considered. Loads must be evaluated for a large variety of conditions such as normal operational conditions, abnormal operational conditions (e.g. start-up, shut-down, or emergency shut-down) and extreme conditions during which the rotor is parked and the blades feathered. These conditions are considered through a suite of more than 20 DLCs specified in IEC 61400-3 [4]. The structure must be designed to have sufficient strength and fatigue life under these DLCs, but an additional requirement that differentiates the design of OWT support structures from traditional structures is that the first natural frequency of the OWT must be separated from the operational frequencies of the rotor to avoid resonance. Depending on site conditions, strength, fatigue lifetime, and resonance avoidance may all govern the final design of the support structure.

The most common support structure for OWTs is the monopile, a circular hollow steel tube that is embedded into the seabed and extends above sea level where it connects to the OWT tower. Roughly 66% of the 318 GW of worldwide offshore wind capacity installed as of late 2013 is generated by turbines supported by monopiles [5]. Most (63%) of this capacity is located in shallow water (water depth < 30 m) [5] where monopiles have been found to meet the structural requirements of IEC 61400-3 at lower cost than alternatives.

OWT support structures fall into a design category that sits between essentially public civil structures, governed by governmentally prescribed design codes, and electro-mechanical devices that are typically designed based on proprietary and market-driven criteria; consequently, there has been relatively little discussion in scholarly literature of the design drivers of OWT support structures, with much of the information regarding this issue being held as proprietary by OWT designers, manufacturers and developers. The conclusions of what has been published [6,7] is ambiguous regarding the relative importance of strength and stiffness in OWT support structure design, with perhaps some preponderance of the evidence favoring the importance of stiffness. If stiffness is indeed a design-driver for most monopiles, an obvious question is whether this situation allows for the most efficient development of the offshore wind resource or whether it would be preferable to avoid resonance through methods other than increasing stiffness (e.g. tuned mass dampers), thereby opening the potential for more efficient monopiles.

In an attempt to provide an answer to whether OWT monopile design is driven by resonance avoidance or strength considerations — putting aside fatigue life as a design driver — this paper takes the following approach:

- (1) three sites are selected along the U.S. Atlantic coast that are amenable to offshore wind energy development and are representative of a range of geographical, oceanic, and meteorological conditions appropriate for monopiles;
- (2) a wind-wave hazard model is developed that uses buoy measurements to calculate operational and extreme wind and wave conditions at each site corresponding to the design (50-year) mean recurrence period (MRP);
- (3) operational and extreme dynamic loads on the OWT along with natural frequencies are calculated for an extensive range of monopile diameters and wall thicknesses and for two types of mudline boundary conditions (fixed and compliant) for each of the three sites. Using these results, a determination is made as to whether stiffness or strength drives the design and what margin exists between the two;

The paper begins by providing details on the three offshore sites considered, including a description of the available measurements of wind and wave conditions at these sites. The next section describes the two methods employed for using measurements of wind and wave to calculate intensities for operational and extreme conditions at a MRP appropriate for design (50-years, per IEC 61400-3). In the following section, the structural model which is employed to convert wind and wave conditions to structural demands (i.e. load effects) is introduced and the method for selecting a monopile diameter and thickness which satisfies both strength and stiffness requirements for each site is described. Next, the numerical results for the wind and wave conditions and the monopile designs are provided for each site along with discussion of the results. The paper concludes with a summary of the findings.

2. Site descriptions

Three sites along the U.S. Atlantic coast are considered in this paper, selected based on a combination of geographic features and the availability of metocean data. Sites located along the mid-Atlantic and Northeastern coasts were favored because the majority of proposals for offshore wind energy development in the U.S. are located there. The three selected sites correspond to the location of metocean data buoys maintained by National Oceanic and Atmospheric Administration (NOAA) with at least 20 years of data available and where water depths are in the reasonable range for monopile support structures (15–30 m). Given these considerations, three sites have been selected that lie off the coasts of the states of Maine, Delaware, and Georgia (identified

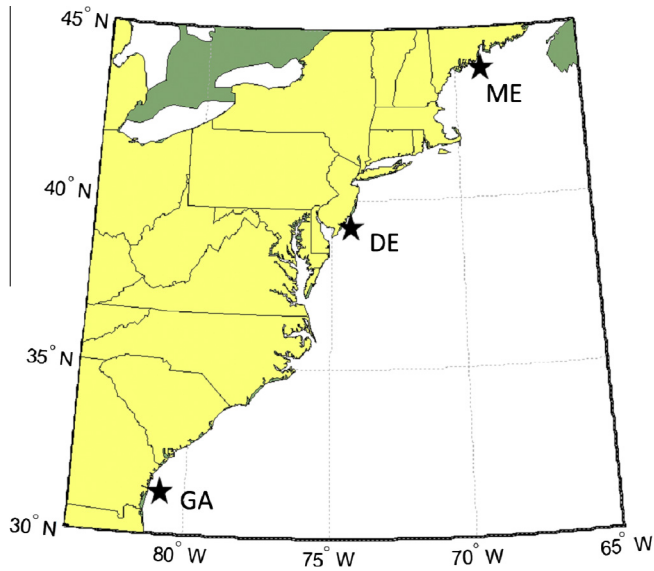


Fig. 1. Locations of three NOAA buoys considered in this paper.

in this paper by their two letter postal abbreviation codes, ME, DE, and GA; Fig. 1 and Table 1 for locations and general characteristics).

The data used in this paper consist of the wind speed measured at 5 m above sea level and the significant wave height, defined as the average of the top one third of recorded wave heights in a given time interval. Wind speed measurements reflect the 8 min average wind speed reported hourly and the significant wave heights are determined based on a 20 min time interval and are also reported hourly. Before applying the wind data to OWT design, 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. Specifically, the wind speed data are amplified by a factor of 1.42 [8] to represent the hourly wind speed at the hub height (90 m) of the considered turbine.

Site-specific soil conditions are not available at the three sites. Soil conditions play an important role in the design of monopiles supporting OWTs, as the soil stiffness influences the restraint condition at the mudline which in turn influences the natural frequency of the structure. Design requirements related to soil strength are not considered in this paper, but the effect of soil stiffness on natural frequency is considered through analysis of two cases: (1) a structural model which has a fixed condition at the mudline; (2) a structural model with non-zero mudline compliance broadly representative of realistic soil conditions. The latter case is described in more detail in the analysis methodology section.

3. Hazard calculation methods

The international standard, IEC 61400-3, prescribes a suite of more than 20 DLCs that require an estimation of loads during a variety of operational and environmental conditions. Wind and wave conditions are characterized by statistical measures, typically the hourly mean wind speed at hub height V , the significant wave

height H_s , and the peak spectral period T_p (i.e. the period associated with the highest power spectral density). This paper focuses on only two specific DLCs. The first, DLC 1.6a, considers turbulent winds under operational conditions combined with a severe sea state. The sea state is defined by H_s corresponding to a 50-year MRP conditioned upon the wind speed V being within the operational range of the turbine. The second, DLC 6.1a, considers extreme turbulent winds with a 50-year MRP combined the 50-year extreme sea state. In this paper, the 50-year values of V and H_s are calculated independently and assumed to occur simultaneously. Although this assumption is known to be a coarse approximation of joint wind-wave conditions [9], it is an assumption used in practice and listed in IEC 61400-3 as a conservative option in the absence of sufficient joint data. For these reasons and for simplicity, this assumption is also used here.

In the following two subsections, details are provided on how H_s and V are calculated for the two considered DLCs. In both cases, the turbulence intensity (i.e. the ratio of the standard deviation of the wind speed to the mean) is considered with a deterministic value ($TI = 0.16$) reflective of type A or “higher” turbulence characteristics [10]. The peak spectral period is also considered deterministically through a range of values conditioned on H_s and gravitational acceleration g ,

$$11.7\sqrt{H_s/g} \leq T_p \leq 17.2\sqrt{H_s/g} \quad (1)$$

This range is not provided explicitly by IEC 61400-3, but rather is established by converting a range provided by IEC 61400-3 for the period of an individual extreme wave within a sea state to T_p [11]. This conversion process is required because IEC 61400-3 states that the designer should “take account of the range of peak spectral period appropriate to each significant wave height” and to base design calculations on “values of the peak spectral period that result in the highest loads acting on an offshore wind turbine,” but does not elaborate nor provide a method to calculate the range. For all cases considered here, the lower bound of Equation (1) is assumed to lead to the highest loads because the lower bound is closer to the fundamental period for all of the structures considered in this paper that satisfy resonance avoidance requirements. While the authors have proposed a method for probabilistically calculating the appropriate value for T_p [11], a simpler approach is employed here.

3.1. Calculation of operational hazard

IEC 61400-3 recommends use of the Inverse First Order Reliability Method (IFORM) for estimating combinations of H_s and V which have a 50-year MRP and are conditioned on operational wind speeds. IFORM [13] is a general method for determining combinations of multiple random variables that correspond to a given MRP. IFORM starts by transforming the joint distribution of measurements of V and H_s into the uncorrelated standard normal variables u_1 and u_2 . In this paper, this transformation is constructed with the Rosenblatt transformation, summarized below,

$$v = F_V^{-1}[\Phi(u_1)] \quad (2)$$

$$h_s = F_{H_s|V}^{-1}[\Phi(u_2)|v] \quad (3)$$

where Φ denotes the standard normal cumulative distribution, F_V denotes the cumulative distribution function of random variable V

Table 1
Site information.

Site	Abbrev	NOAA ID	Latitude	Longitude	Observation length (years)	Water depth (m)	Dist. to shore (km)
Maine	ME	44007	43.53°N	70.14°W	31	23.7	5.60
Delaware	DE	44009	38.46°N	74.70°W	27	30.5	30.3
Georgia	GA	41008	31.40°N	80.87°W	20	19.5	32.3

and $F_{H_s|V}$ denotes the cumulative distribution function of random variable H_s conditioned on V . In this paper, the marginal distribution of the hourly mean wind speed $F_V(v)$ is modeled with a Weibull distribution, which is fit to the hourly measurements of the mean wind speed using maximum likelihood estimation. The distribution of the significant wave height conditioned on the mean wind speed $F_{H_s|V}(h_s|v)$ is modeled with a normal distribution fit to hourly measurements using the method of moments. In total, for each site, conditional distributions of H_s are calibrated for twenty-two evenly spaced values of V between 3 m/s, the cut-in wind speed, and 25 m/s, the cut-out wind speed. The cut-in and cut-out wind speeds are the minimum and maximum wind speeds of the operational range of the turbine. Once these distributions are established for each site, a circle in u_1-u_2 space, centered at the origin with radius β is transformed using Eqs. (2) and (3), into a contour in $V-H_s$ space. The magnitude of β determines the MRP associated with the contour according to

$$\beta = \Phi^{-1}(1 - 1/N) \tag{4}$$

where N is number of independent sea states in the MRP of interest. In this case, $MRP = 50\text{-years} = 438,000 \text{ h}$ and the sampling frequency is once per hour giving $N = 438,000$ and $\beta = 4.6$.

Scatter plots of hourly measurements of V and H_s for the ME, DE and GA sites are shown in Fig. 2. Superimposed on these plots are the top portion of a 50-year contour calculated using IFORM and assuming a normal distribution for $F_{H_s|V}$. Annex G of IEC 61400-3

suggests the use of either normal or lognormal distributions for $F_{H_s|V}$, but cautions that there may be difficulties using either choice and that considerable judgment is required when using this method. For this paper, the normal distribution was selected for use in the design investigation because a lognormal distribution provided illogically high estimates of the 50-year conditional wave height. The normal distribution also results in some illogical estimates (e.g. at low operational wind speeds, the 50-year contour appears too low relative to the data), however, even still, these results were much more reasonable than those based on a lognormal distribution. The point on the 50-year contour corresponding to the rated wind speed (i.e. the wind speed at which the turbine generates maximum power) is highlighted for each site because this was the combination of wind and wave conditions which was assumed to control the operational demands. The peak spectral period corresponding to the wave height at this point is also indicated.

3.2. Calculation of extreme hazard

As with the operational hazard, the extreme hazard for each site is calculated using wind and wave measurements from NOAA buoys. Using limited durations of measurements to estimate extreme wind and wave conditions is challenging as such conditions are likely to be influenced by ‘direct-hit’ hurricane events which are unlikely to have occurred even once during the NOAA

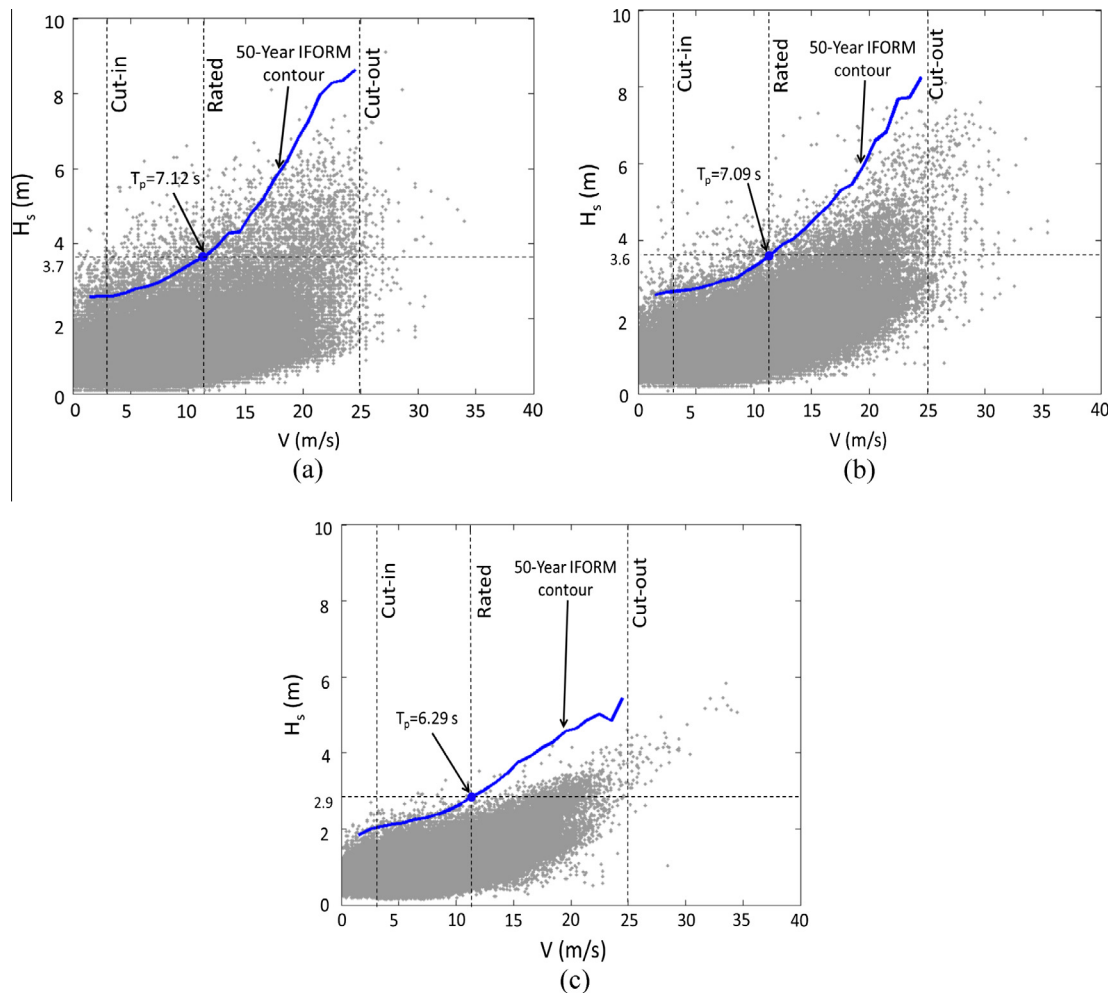


Fig. 2. Measurements of H_s and V along with the top portion of the 50-year environmental contour based on IFORM and a normal distribution for $F_{H_s|V}$ for the three considered NOAA buoys, (a) ME, (b) DE and (c) GA.

data record. While a preferred method estimates wind and wave intensities under a catalog of synthetic hurricane events which is designed to represent the possibilities of future hurricane activity, a straightforward model based only on NOAA buoy measurements is selected here for simplicity and because such stochastic synthetic hurricane catalogs are typically proprietary and not publically available. At long MRPs, there are often significant differences between hazard calculated based on decades of measurements and hazard calculated based on a synthetic hurricane catalog, however those differences are less pronounced for design MRPs (e.g. 50-years). As an example, for a site off the coast of Massachusetts, a 2009 report [6] stated that the ratio between the one-hour, 10 m wind speed calculated based on a hurricane catalog and the same wind speed calculated based on 20+ years of measurements was 1.25 for a 50-year MRP and 1.34 for a 100-year MRP.

The extreme hazard is estimated based on a maximum likelihood fit of the generalized extreme value (GEV) distribution to the annual maxima of wind speed and significant wave height extracted from the NOAA data. The form of the GEV distribution is given by

$$f_x(x, k, \mu, \sigma) = \left(\frac{1}{\sigma}\right) \exp\left(-\left(1 + k\frac{x-\mu}{\sigma}\right)^{-\frac{1}{k}}\right) \left(1 + k\frac{x-\mu}{\sigma}\right)^{-1-\frac{1}{k}},$$

$$1 + k\frac{x-\mu}{\sigma} > 0 \quad (5)$$

with k is the shape parameter, σ is the scale parameter, and μ is the location parameter. It is important to note that annual maxima measurements of V and H_s may not be simultaneous, and therefore this approach may result in more severe combinations of V and H_s than what was measured.

The values of V and H_s at a 50-year MRP for the three sites considered here, the best-fit GEV distributions, and the annual maxima of the NOAA data are shown in Fig. 3. For the ME site, 3 out of the 31 annual maxima measurements are taken during hurricane events (Gloria in 1985; Bob in 1991; Noel in 2007), while for the DE site, the number is 6 out of 27 (Charley in 1986; Bob in 1991; Josephine in 1996; Floyd in 1999; Irene in 2011; Sandy in 2012) and, for the GA site, the number is 2 out of 20 (Floyd in 1999; Gabrielle in 2001). Note that although some of the annual maxima data is caused by hurricane events, in most cases, it does not mean that the site experienced hurricane wind speeds, as the site is unlikely to have been hit directly by the hurricane at its peak intensity.

4. Analysis methodology

A set of schematic monopile designs for the NREL 5 MW reference turbine [12] (summarized in Table 2), characterized in terms of their diameter D and thickness t , are considered for each of the three sites and evaluated with respect to strength and stiffness requirements to delineate the regions of the design space in which resonance avoidance or strength controls. The set of conceptual designs includes 104 combinations of D (between 3 m and 10 m spaced at 1.0 m) and t (between 0.03 m and 0.09 m spaced at 0.005 m).

4.1. Frequency analysis

Each of the conceptual support structures (i.e. each combination of D and t) at each site was modeled using prismatic Euler–Bernoulli beam elements with the number of elements chosen to ensure convergence of the natural frequencies and mode shapes of the structure. For each model, a fixed and compliant boundary condition at the base (i.e. mudline) of the model is considered. The mudline stiffness of the compliant boundary condition is chosen to represent a uniform deposit of medium dense to dense sand with a friction angle of 40° , a relative density of 0.55, a submerged soil unit weight of 10 kN/m^3 , and an initial modulus of subgrade reaction of 20.8 MPa/m . These parameters are selected to be broadly representative of the U.S. Atlantic coast. Soil–pile interaction is incorporated through a series of springs that provide lateral support to the pile, assuming a 20 m embedment depth and assuming that each soil spring has stiffness equal to the initial

Table 2
Specifications of the NREL 5 MW baseline offshore wind turbine.

Parameter	Value
Power rating (MW)	5
Rotor diameter (m)	126
Number of blades	3
Hub height above sea level (m)	90
Cut-in, Rated, Cut-out Wind speeds (m/s)	3.0, 11.4, 25.0
Rated rotor speeds (rpm)	12.1
Tower diameter base, top (m)	6.0, 3.9 (linear variation)
Tower thickness base, top (mm)	35, 25 (linear variation)
Nacelle mass (t)	240
Tower mass (t)	350
Rotor mass (t)	110
1P frequency band for operational RPMs (Hz)	0.12–0.20
3P frequency band for operational RPMs (Hz)	0.35–0.60

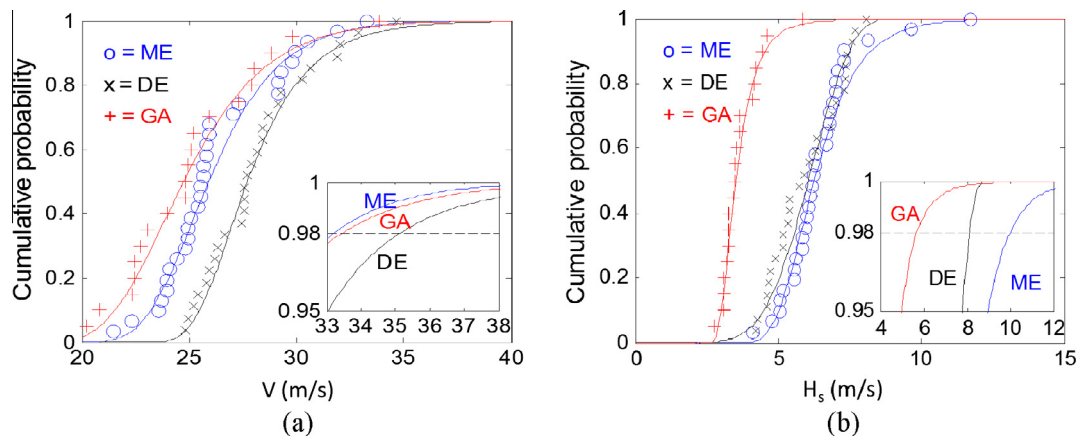


Fig. 3. Best fit GEV distributions to the adjusted NOAA data for the ME, DE, and GA sites for (a) wind speed and (b) significant wave height. Inset figures show detail of the upper tail of the distributions with the 0.98 cumulative probability level, which corresponds to the 50-year MRP, shown as a dashed line.

tangent stiffness of a P – Y curve calibrated for the soil properties and depth below grade [14]. The toe of the monopile is modeled with a roller support. It is important to note that the P – Y curve method was originally developed for small diameter, flexible piles, and has been shown to be too stiff when applied to large diameter piles such as those considered here [15–17]. For this reason, researchers have advised using caution when applying P – Y curves to large diameter OWT monopiles [15]; nevertheless, P – Y curves are the recommended method for modeling lateral soil–pile resistance by the design standard DNV-OS-J101 [18] and they are used here for their simplicity and since this paper is considering flexible mudline conditions that are broadly representative rather than a specific case.

The first structural frequency f_{n1} of each candidate design is calculated for each site and mudline condition by performing an eigenvalue analysis on the finite element model of the OWT. For the purposes of the eigenvalue solution, the rotor nacelle assembly is modeled with a lumped mass and the tower and monopile are modeled with distributed lumped masses. Each of these frequencies is compared to the appropriate operational frequencies of the rotor for the NREL 5 MW turbine, specifically 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). Both the 1P and 3P frequencies vary with the wind speed, and there are usually three acceptable ranges of structural frequencies that avoid these operational frequencies: the soft–soft range, which includes frequencies that are always less than the 1P frequency, the stiff–stiff range, which includes frequencies that are always greater than the 3P frequency, and the soft–stiff range, which includes frequencies between the 1P and 3P frequencies. For design purposes ASCE/AWEA (2011) [1] recommends a 10% margin around the 1P and 3P frequency bands to account for uncertainty. These requirements are visualized in a so-called Campbell Diagram (Fig. 4 for the NREL 5 MW baseline offshore turbine). It is generally not practical to design an OWT structure for the soft–soft or stiff–stiff regions, so the structural frequency of the turbine is usually designed to fall within the soft–stiff region. That is, the design requirement for the NREL 5 MW turbine is $0.22 \text{ Hz} < f_{n1} < 0.32 \text{ Hz}$.

4.2. Time history analysis for calculating moment demands

Moment demands used in evaluation of strength-based design criteria require dynamic time history analysis of the OWT under stochastic wind and wave loads. Each conceptual support structure

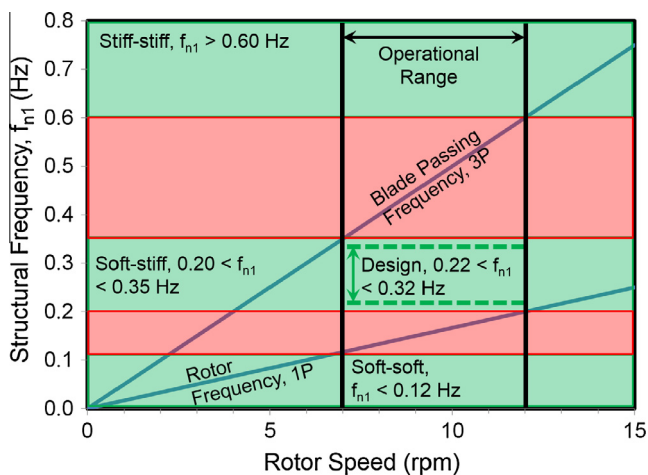


Fig. 4. Campbell diagram for the NREL 5 MW baseline turbine.

design is modeled for each of the three sites in FAST, an open source program developed by NREL for the analysis of wind turbines subjected to aerodynamic and hydrodynamic loading. The compliant mudline boundary condition is modeled in FAST similarly to the “apparent fixity” approach outlined in [19] by artificially extending the model of the monopile to a depth that, when modeled with a fixed boundary condition, has an identical natural frequency to that calculated with the actual depth and a compliant boundary condition. A fixed-bottom monopile with this equivalent depth is then modeled in FAST with all wave kinematics set to zero at depths below that corresponding to the actual mudline and with moment demands evaluated at the depth of the actual mudline.

This FAST model is used to convert environmental conditions characterized by H_s , T_p , V and I into moment demands, specifically the mudline moment for operational and extreme design conditions. The model is subjected to simultaneous, one hour time series of irregular waves and turbulent winds. The irregular waves are modeled using a JONSWAP spectrum with 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 [20]. The second-order component results in the wave series being non-Gaussian with non-zero skewness. As described by Agarwal and Manuel [20], this approach results in a more accurate wave time series and higher structural loads than a linear model, especially for shallow water. The turbulent winds are modeled as a Gaussian process using a Kaimal spectral model and an exponential coherence model, as specified in Annex B of IEC 61400-1 [10]. Such a modeling approach is predominant for structural design applications, however non-Gaussian models have been shown to more realistically represent measured data and to result in loads on wind turbine blades with differences on the order of $\pm 10\%$ compared to Gaussian models [21]. The turbulent wind and irregular wave time series are each simulated independently, as is standard practice in the design of offshore structures. Each structural model is analyzed for six, one hour realizations of the wind and wave time histories. The design mudline moment for a particular conceptual design at a particular site and for a particular mudline condition is taken to be the average of the maximum mudline moment in each of the six simulations, as specified in IEC 61400-3 [4]. For each conceptual design, the same six seeds are used for the random number generators implemented in the wind and wave time history simulation code in order to focus on the conceptual design parameters rather than load variability.

In practice and as specified by IEC for DLC 1.6a, operational loads for 50-year wind and wave conditions are evaluated for several wind speeds between cut-in and cut-out. For this study, operational loads are evaluated only under the rated wind speed (11.4 m/s). This is a simplification that was employed to limit the number of analyses required by this study—104 (D , t) combinations, 3 sites, 2 mudline conditions, 6 simulations per combination yields 3744 total FAST runs for the operational case. Although it is possible that a moment demand will be higher at other operational wind speeds, it is noted that, for operational conditions, aerodynamic loads are much greater than hydrodynamic loads and that aerodynamic loads are greatest at the rated wind speed. Extreme loads are evaluated for a yaw error of 0° . Although IEC 61400-3 requires consideration of a range of yaw errors, in the authors’ experience, the magnitude of the aerodynamic thrust for the NREL 5 MW turbine does not vary significantly over this range.

5. Numerical examples

Following the hazard and structural modeling frameworks described in the previous two sections, results for structural frequencies and mudline moments under operational and extreme conditions for all sites, conceptual designs and mudline boundary

Table 3
Range of parameters considered in this numerical study.

Parameter	Range	Number of models
Monopile diameter	3.0–10 m, evenly spaced at 1.0 m	8
Monopile thickness	0.03–0.09 m, evenly spaced at 0.005 m	13
Site	ME, DE & GA (see Table 1 and Table 4 for details)	3
Mudline boundary condition	Fixed, Compliant	2
Total number of models		624

Table 4
Operational and extreme hazard with a 50-year MRP based on hourly wind and wave measurements at three NOAA buoys.

Site	50-year operational hazard			50-year extreme hazard		
	V (m/s)	H_s (m)	T_p (s)	V (m/s)	H_s (m)	T_p (s)
ME	11.4	3.66	7.12	32.9	10.0	11.8
DE	11.4	3.63	7.09	35.8	8.10	10.6
GA	11.4	2.86	6.29	33.1	5.70	8.90

conditions are presented in this section. In total, these quantities are assessed 624 times for all combinations of 8 monopile diameters (from 3 m to 10 m spaced at 1.0 m), 13 monopile thicknesses, (from 0.03 m to 0.09 m spaced at 0.005 m), two mudline boundary conditions (fixed and compliant) and three locations (ME, DE, and GA), see Table 3. Table 4 summarizes the statistics that characterize operational and extreme hazard for each of the three sites calculated using the methods described previously.

The results from all analyses are presented in Fig. 5, which is comprised of six sub-figures representing all combinations of the three sites and two mudline boundary conditions. The contours represent the factored demand to capacity ratio of the mudline moment for each combination of monopile diameter and thickness. A ratio greater than 1.0 indicates that the strength design criterion has not been met. Demand is calculated for operational and extreme environmental conditions as described above and the larger of these two demands is amplified by a 1.35 load factor per IEC 61400-3 and plotted relative to the critical moment per DNV-RP-C202 reduced by the specified resistance factor [22]. A bold dashed line separates regions of the design space in which operational moment demands exceed extreme moment demands and vice versa. A bold solid line separates regions of the design space in which $f_{n1} < 0.22$ Hz from those in which $f_{n1} > 0.22$ Hz. All considered conditions had $f_{n1} < 0.32$ Hz, and so any condition with $f_{n1} > 0.22$ Hz satisfied resonance avoidance conditions for the soft-stiff frequency range (Fig. 4). For each plot in Fig. 5, two regions are shaded, one with coarse shading indicating combinations of diameter and thickness which have a demand to capacity ratio less than 1.0 (safe with respect to strength) and a structural frequency $f_{n1} > 0.22$ (safe with respect to resonance) and another with fine shading indicating combinations with demand to capacity ratio less than 1.0 (safe with respect to strength) and a structural frequency $f_{n1} < 0.22$ (unsafe with respect to resonance).

6. Discussion

The finely shaded regions indicated in the plots in Fig. 5 show combinations of monopile diameters and thicknesses that satisfy strength requirements, but do not meet resonance avoidance

requirements, while the coarsely shaded regions show combinations that satisfy both requirements. It is important to recognize that the results in this Figure have ignored fatigue considerations as a potential limit state in the design of monopiles. The boundary of the coarsely shaded regions, referred to herein as the design boundary, specifies combinations that exactly satisfy design requirements and the figures readily show whether these combinations are controlled by resonance avoidance, operational moment demand or extreme moment demand. As a general trend, smaller diameters and larger thicknesses on the design boundary tend to be controlled by resonance avoidance compared to moment demand. For all three sites, the portion of the design boundary controlled by resonance avoidance is larger for the compliant mudline boundary condition compared to the fixed boundary condition. This effect is caused more so by the shift in the resonance avoidance line (towards the right, larger diameters) for the compliant mudline condition than by changes to the demand to capacity ratio contours. For moment demands, smaller diameter monopiles tend to be controlled by operational conditions while larger diameters tend to be controlled by extreme conditions. This diameter dependency can be explained by noting that (1) extreme conditions are wave-dominated and the wave loads scale monotonically with monopile diameter according to Morison's equation [23] and (2) operational conditions are wind-dominated and the monopile diameter has minimal influence on the aerodynamic loads.

Another observation from Fig. 5 is that structural frequency influences moments due to extreme condition more than moments due to operational conditions, which can be observed by the significant shift in the boundary between extreme and operational moment demand when comparing the results for compliant vs. fixed mudline conditions. This can be explained by noting that (1) operational conditions are wind-dominated, and the wind spectrum is relatively broad banded with peak spectral density at frequencies much lower than any of the structural frequencies considered here and (2) extreme conditions are wave-dominated and the wave spectrum is more narrow-banded with peak spectral density at a frequency close to but less than the structural frequencies considered here. This means that, for a given monopile diameter and thickness, changing the mudline conditions from fixed to compliant, shifts the structural frequency to a portion of the wave spectrum with notably higher spectral density.

Differences in the plots for the three sites in Fig. 5 are caused by differences in water depth and environmental hazard. Changes in the resonance avoidance line from site to site are due to changes in water depth, with the resonance avoidance line shifting right (towards larger diameters) for deeper sites and left for shallower sites. As mentioned before, the operational moment demand is wind-dominated and not sensitive to the range of structural frequencies considered here. Since the operational wind hazard in this study does not vary from site to site, the operational moment demand contours vary minimally with changes caused only by difference in moment arm due to water depth (moment arms vary by 10%, from ~110 m at GA to ~120 m at DE) and from differences in the 50-year H_s for operational conditions. The extreme moment demand contours are wave-dominated and influenced by many factors which change from site to site, including changes in the natural frequency, moment arm length, loading distribution for fixed H_s , and environmental hazard. These factors combine such that there is significant variability from site to site in the extreme moment demand contours. Disregarding the changes in environmental hazard from site to site and considering only the range of parameters in this paper, deeper sites have larger extreme moment demands due to decreased natural frequency from longer moment arms and a larger portion of the monopile loaded hydrodynamically.

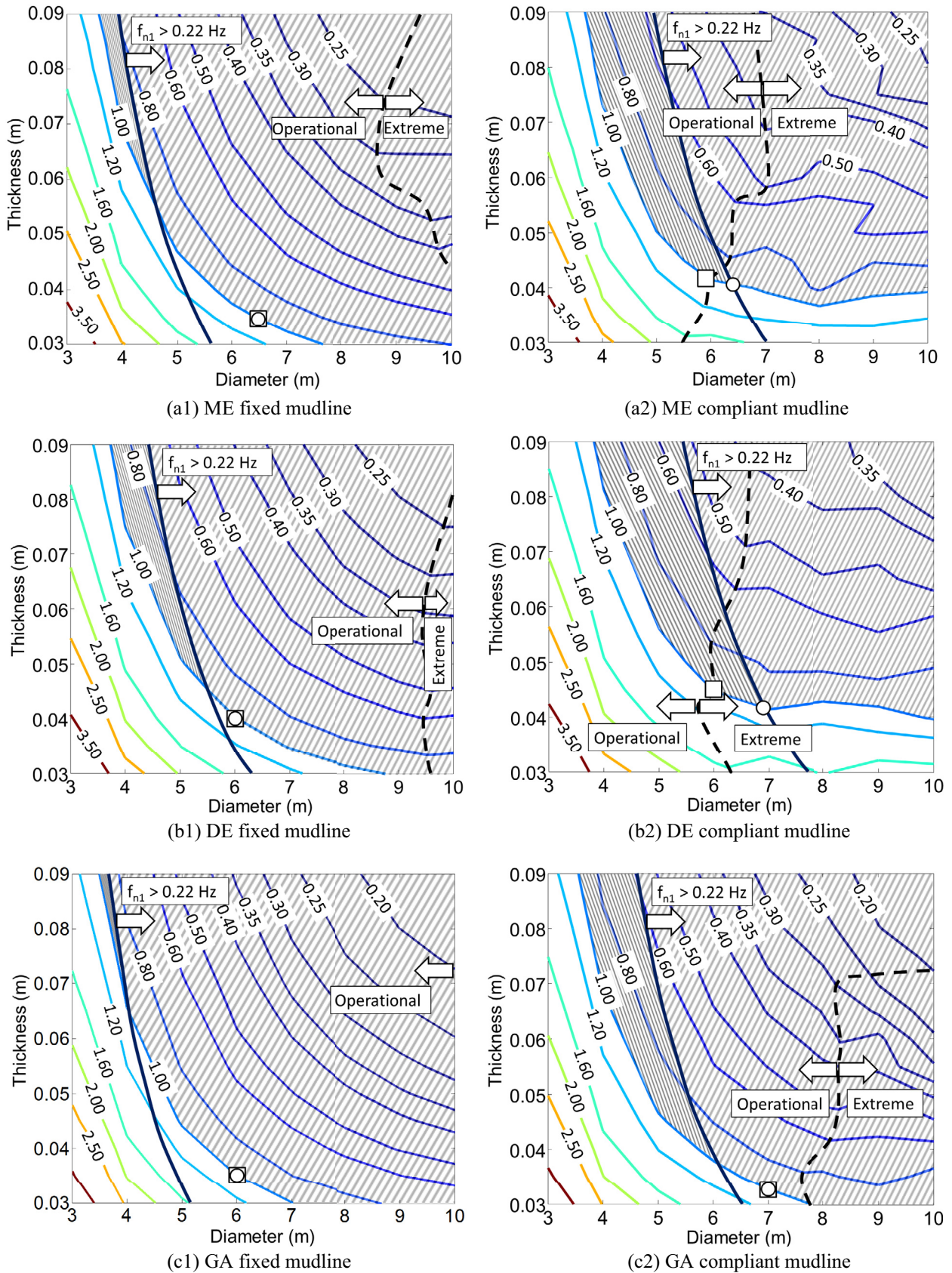


Fig. 5. Contours of constant demand to capacity ratio for the mudline moment of a monopile for the six considered case studies, (a1)–(c2). A solid bold line distinguishes between monopiles with f_{n1} less than and greater than 0.22 Hz. A dashed bold line distinguishes between monopiles with a demand to capacity ratio controlled by operational and extreme conditions. An open circle indicates the monopile with least area that satisfies strength and resonance requirements. An open square indicates the monopile with least area that satisfies only strength requirements.

Table 5
Monopile diameter D and thickness t with minimum cross-sectional area A on the design boundary for (1) designs satisfying mudline moment demands and resonance avoidance and (2) designs satisfying only mudline moment demands. RA = Resonance Avoidance. EM = Extreme Moment. OM = Operational Moment.

Site	Mudline boundary condition	Design satisfying resonance avoidance and moment demands				Design satisfying moment demands only			
		D (m)	t (m)	A (m ²)	Controlling condition	D (m)	t (m)	A (m ²)	Controlling condition
ME	Fixed	6.5	0.035	0.70	OM	6.5	0.035	0.70	OM
	Compliant	6.4	0.041	0.82	RA	5.9	0.042	0.77	OM
DE	Fixed	6.0	0.040	0.76	OM	6.0	0.040	0.76	OM
	Compliant	6.9	0.042	0.91	RA	6.0	0.045	0.84	OM–EM
GA	Fixed	6.0	0.035	0.66	OM	6.0	0.035	0.66	OM
	Compliant	7.0	0.033	0.72	OM	7.0	0.033	0.72	OM

OWT designers would typically try to minimize the mass of the support structure to minimize material costs and possibly reduce transportation costs. Such minimization must be done, of course, subject to a large number of constraints related to design criteria, manufacturing, and installation, etc. In the context of this study, it is of potential interest to identify the point along the design boundary for which the cross sectional area (and mass) of the monopile are minimized. Table 5 gives the monopile diameter and thickness that minimize area subject to the constraint of the design boundary and indicates which condition controls the monopile design at this location in the design space (these combinations are also indicated with an open circle in Fig. 5). Also included in the table is the diameter and thickness on the design boundary if resonance avoidance were not a design requirement (these combinations are also indicated with an open square in Fig. 5). Monopile area minimization at all sites is controlled by operational moment demands if the mudline is fixed and strength and resonance conditions are satisfied, whereas, when the mudline is compliant, only the GA site is controlled by operational moment demands and the other two sites are controlled by resonance avoidance. At these two sites and for a compliant boundary condition, the minimum monopile area would be reduced by 6–8% if resonance avoidance were not a design consideration or if it could be satisfied by means other than stiffening the monopile. When resonance avoidance is not considered as a design condition, the least area monopile at all sites and boundary conditions is controlled by operational moment demands except for the DE site with a compliant boundary condition which has identical controlling demands from both operational and extreme conditions.

7. Conclusions

The design of OWT monopile support structures has been investigated in a limited design space consisting of the pile diameter and thickness under operational and extreme loading conditions as specified by IEC 61400-3 and considering both a fixed and compliant mudline. For simplification, the design did not consider the fatigue limit state. Three sites along the U.S. Atlantic Coast were used to generate the illustrative examples and represent a range of conditions prevalent in regions of likely OWT development in the U.S. The purpose of the investigation was to define regions of the design space in which strength or stiffness controls design. Stiffness is of particular concern in the design of OWT support structures because dynamic loading caused by the rotation of the rotor and because the combined wind/wave loading has frequency content near the natural frequencies of the structure. Specifically, design practice requires that the first natural frequency of the structure fall in a relatively narrow band between the 1P and 3P operational frequencies of the turbine. Understanding whether strength or stiffness controls design is of potential importance to OWT development because, when stiffness controls, the structure

will have reserve capacity that results in reliability larger than target design reliabilities and more material usage than needed for strength considerations.

The key finding of this paper is that strength requirements control design in four of the six cases investigated. The two cases for which stiffness controls are for the two sites with deeper water depths (ME and DE) and for which the mudline has been modeled as compliant. For these two cases, the area (and mass) of the monopile would be reduced by 6–8% if strength instead of stiffness controlled the design. It therefore seems that for deeper water depths and more compliant boundary conditions, there is modest room for increasing the structural efficiency of OWT monopiles by controlling structural dynamics through means other than stiffness. For such conditions, investigation into alternate means of controlling the structural natural frequencies or increasing damping could be warranted. The remaining four cases were found to be controlled by moment demands under operational conditions.

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