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Incremental wind-wave analysis of the structural capacity of offshore wind turbine support structures under extreme loading

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ABSTRACT

Offshore wind turbine (OWT) support structures are subjected to non-proportional environmental wind and wave load patterns with respect to increases in wave height and with respect to wind and wave combined loading. Traditional approaches to estimating the ultimate capacity of offshore support structures are not ideally suited to analysis of OWTs. In this paper, the concept of incremental wind-wave (IWWA) analysis of the structural capacity of OWT support structures is proposed. The approach uses static push-over analysis of OWT support structures subject to wind and wave combined load patterns corresponding to increasing mean return period (MRP). The IWWA framework can be applied as a one-parameter approach (IWWA1) in which the MRP for the wind and wave conditions is assumed to be the same or a two-parameter approach (IWWA2) in which the MRPs associated with wind and wave conditions are related to a joint probability density function characterizing the wind and wave conditions at the site. Example calculations for monopile and jacket supported OWTs at Atlantic marine sites are performed under both one parameter and two parameters IWWA framework. The analyses illustrate that: the results of an IWWA analysis are site specific; and structural response can be dominated by either wind or wave conditions depending on structural characteristics and site conditions. Finally, reliability analyses for both examples excluding uncertainties in structural resistance are estimated based on their IWWA results and probabilistic models for site environmental conditions.

1. Introduction

In recent decades, offshore wind energy has been experiencing rapid worldwide development as an attractive renewable energy source [1]. Europe and Asia already have significant installed capacity on the order of gigawatts in Europe to hundreds of megawatts in Asia [2,3], and it is planned that 20% of the electricity demand of the United States will come from wind energy by 2030 [4]. Compared with onshore wind energy, offshore wind energy resources are emphasized both to exploit the tremendous potential of the offshore wind resource and mitigate the impact of wind turbines development on human populations [5]. The middle and northern Atlantic coast, the region in which offshore wind energy development will likely be focused in the U.S., is regarded as a natural place for offshore wind energy due to the large wind resource and short distance to population centers [6]. This paper introduces a novel method for assessing the structural capacity of fixed-bottom offshore wind turbine (OWT) support structures. The method accounts for varying load patterns that can affect OWT capacity due to non-proportional scaling effects of increased wind speed, wave height and their combinations. At this point in its development, the approach does not account for additional loads due to current or changes in mean water level. This approach, by accounting for variation in load pattern with increasing intensity of environmental conditions, contrasts with the approach typically used in seismic analysis of structures in which a single parameter, e.g. peak ground or spectral acceleration, can reasonably parameterize the seismic loading and the load distribution or pattern can be assumed to be constant with increasing acceleration. The approach described here is particularly useful for risk analysis of OWTs under extreme loading since current design practice attempts to ensure elastic response under design conditions.

To install wind turbines at marine sites, many configurations of offshore support structures are used. The monopile is the most common solution for OWTs in shallow water up to 25–30 m, and jackets are being investigated and proposed for the use of

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fixed-bottom turbines in deeper water (30–80 m depth) which promises to open up even more of the offshore wind resource [7,8].

Although movement towards offshore wind development is potentially transformative for the nation's energy portfolio, challenges become apparent almost at once: the Atlantic coast is at a considerable risk from severe hurricanes [9] and the environmental (wind and wave) loadings on OWTs lead to greater forces in the structure than those that would occur onshore, exacerbating the civil engineering problem [10–12]. Furthermore, the greater cost of OWTs means that analysis of the probability of failure under extreme loadings is even more important for OWTs than for onshore turbines.

Many studies have been carried out for load and response estimation for wind turbines with monopile and jacket foundation under uncoupled or combined wind and wave conditions from a given environmental model or measured data, e.g. Seidel et al. [13], Agarwal and Manuel [14], Jensen et al. [15], Haselbach et al. [16], Mardfekri and Gardoni [17] and Saha et al. [18]. These studies were mainly concentrated on dynamic time history simulation of support structures in the elastic or operational range of response. Ultimate capacity of OWTs has been seldom mentioned in the literature, yet an accurate prediction of capacity is needed to allow rational risk assessment.

Pushover analysis with material and geometrical nonlinearities is an efficient approach to evaluate nonlinear behavior and ultimate capacity of offshore structures [19] and has been recommended by industry standards [20]. Although pushover analysis techniques are well developed for offshore oil and gas platforms, that framework is not ideally suited to analysis of OWTs. First, traditional pushover analysis assumes that the lateral loading can be parameterized by a single variable and assumes that the lateral loading distribution or pattern remains constant as the load parameter is scaled up. This assumption does not hold for OWTs since wind and wave loads on the structure are imperfectly correlated [21] and wave loads are non-proportional with an increase in wave height [22]. This variability of the load pattern with increasing loading intensity leads to a dependence between the load intensity and the capacity [23,24]. The simplest way to understand this load pattern variability is that a taller wave, even if generating the same base shear as a shorter wave, will generate greater base moment due to the greater moment arm associated with the greater wave height. Second, extreme conditions which only appear at longer return period can contribute significantly to the lateral load in ways that could not be accounted for if a shorter return period load were simply scaled up as in a traditional pushover analysis. For example, wave-in-deck forces arise when the wave height is such that the wave interacts with the deck of a jacket structure [25]. A recent study of capacity analysis of oil and gas jacket platforms has introduced the idea of Incremental Wave Analysis (IWA), which arrives at capacities for jackets and accounts for load pattern variation with wave height [26]. This paper extends the idea of IWA to include the wind loads that act on an OWT and describes a new approach to fixed-bottom OWT capacity analysis called Incremental Wind-Wave Analysis (IWWA).

The remainder of this paper is organized as follows: The IWWA framework is introduced as a general approach in both single and double parameter versions in which the wind and wave loading intensities are scaled by a single parameter or two joint parameters; specific methods for calculating wind and wave loads on OWTs used in the examples are summarized; two example support structures, a monopile and a jacket, are introduced and site environmental conditions are specified; the results of IWWA analysis are presented for both structures and those results are discussed; finally, the main conclusions of the paper are summarized.

2. Incremental wind-wave analysis framework

The incremental wind-wave analysis (IWWA) framework provides a systematic and efficient approach to evaluate the capacity of OWT support structures subject to arbitrary combinations of wind and wave load. Dynamic effects of wind and wave loads, such as wind turbulence, wave irregularity, wave-structural interaction, time-dependent variance of loading direction and amplitude, etc., that require time history analysis would affect the operational and ultimate results. The question of dynamic versus static capacity estimation has ever been addressed by Golafshani et al. [26] for offshore oil and gas support structures and they found, for two different example platforms, that the difference between the dynamic and static results was either negligible (less than 0.5%) or approximately 14% such that the static analysis provided a conservative estimate. The difference is dependent on the dynamic behavior of the platform. Dynamic effects have been neglected here to provide initial insights into the load pattern dependence of OWT support structure capacity and because of the large computational demands of the multiple nonlinear time history analyses required for an analogous dynamic approach. In this section two forms of the IWWA framework are described in general terms, one–called: IWWA1–in which a scalar hazard measure is used for combined wind and wave loading and one–called: IWWA2–in which separate hazard measures are used for the wind and wave loading. Although the scalar, single-parameter IWWA has significant limitations due to the assumption that wind and wave conditions at equivalent, independently estimated return periods occur simultaneously, it is still described first followed by the vector-valued two-parameter IWWA to provide maximum clarity and accessibility of the methods.

2.1. Single-parameter IWWA

Consider an OWT support structure that occupies a space denoted by $S \subset \mathbb{R}^3$, has material properties $M(x)$ where $x \in S$ gives a position in the structure, and subject to point and distributed loading $L(x; \text{wind}, \text{wave})$, where $x \in \partial S$ denotes a point on the surface of the structure. For the purposes of developing the IWWA framework, the support structure is assumed to be fully fixed at the mudline and consist of the entire OWT assembly up to the bottom of the rotor-nacelle assembly (RNA). In the single parameter IWWA the environmental conditions are parameterized by the mean return period (MRP) of the wind and wave conditions so that the loading can be expressed as $L(x; \text{MRP})$ and to maintain a direct connection to the probabilities of occurrence of the environmental conditions. In principle, any method for estimating the wind and wave conditions at various MRPs can be used in the IWWA framework including approaches such as the inverse first order reliability method (IFORM) that generates a contour in the probability space corresponding to an MRP for a set of combinations of wind and wave conditions. Here the joint pdf of the wind and wave conditions is just that, but the independently assessed return periods have been substituted for the wind speed and wave height to maintain a closer connection to the probabilities of occurrence of the wind and wave conditions. The specific approach used here is developed as follows: (1) We calibrate the probability models for wind and wave based on the historical database from the NOAA data buoy center, annual maxima selected from hourly measurements of the wind speed at 5 m elevation W_s (60 min, 5 m) and significant wave height H_s; (2) We adjust those values to correspond to 1-min wind speeds at 90m elevation $W_s\ (1\ \text{min},\ 90\ \text{m}) = 1.608 W_s\ (60\ \text{min},\ 5\ \text{m})$ or extreme waves $He = 1.87 H_s$; (3) We develop independent joint probability models for these annual maxima of the 1-minute wind and extreme wave and use these models to associate MRP with specific values of wind and wave. It should be emphasized that the approach described here considers some very long MRPs. Such values, lying far in the tail of the distribution, are not generally used in the engineering design and can also be difficult to estimate. However, they have an important contribution to the total failure probability which is needed by decision makers regarding the risk profile of the structure and attendant financing. Another important consideration is the method for converting wind and wave conditions into the structural loads L. As with the MRP calculation, many approaches for converting environmental conditions into loads are possible within the IWWA framework. The specific approach used in this paper is described along with the example calculation.

Once the structure and loading have been defined, the single-parameter IWWA procedure entails the generation of a family of nonlinear static pushover curves for the structure as follows:

1. Select a series $\mathrm{MRP}_i,\ i = 1,\ldots,n_{\mathrm{MRP}}$ hazard intensities of interest.
2. For each MRP_i, perform a static, geometrically and material nonlinear pushover analysis that yields two load deformation curves: $D_{trans,i}(\gamma)$ which gives the lateral displacement at the support structure-tower transition as a function of a load factor γ and $D_{top,i}(\gamma)$ which gives the tower top displacement as a function of the same load factor. The role of the load factor γ is crucial in this definition, and it is defined such that when $\gamma = 1$ the applied load to the structure is exactly equal to $L(x;\mathrm{MRP}_i)$. The material nonlinearity will generally be modeled in such a way that the ultimate capacity is reached when a fully developed static mechanism is formed in the structure.
3. Define the capacity of the support structure at hazard level MRP_i to be $\gamma_{max,i} = \min(\gamma : d\gamma/dD_{trans,i} \approx 0 \text{ or } d\gamma/dD_{top,i} \approx 0)$ where $d\gamma/dD_{trans,i}$ and $d\gamma/dD_{top,i}$ are the slopes of the load displacement pushover curves and the structural capacity is reached whenever the pushover curves reach a plateau of near zero slope.
4. Define the single-parameter IWWA curve $\mathrm{IWWA1}(\mathrm{MRP}_i) = V_i \min(1,\gamma_{max,i})$ where V_i is the base shear generated by the wind and wave conditions at MRP_i. The IWWA1 curve can be represented graphically as a plot of IWWA1 against $D_{trans,i}, D_{top,i}$, or MRP_i. The use of $\min(1,\gamma_{max,i})$ ensures that the IWWA1 curve does not exceed the actual demand base shear applied to the structure and that a clear failure plateau appears in the curve. Note that base moment M_i could be substituted for base shear V_i without changing the fundamentals of the approach.

This procedure is illustrated in Fig. 1 which shows a schematic of an OWT supported by a jacket foundation with wind and wave loads applied, schematic pushover curves for such a structure with load patterns corresponding to two different MRPs, and a schematic IWWA1 curve that could result from such analyses. It is particularly important to note that, conceptually, each point on the IWWA1 curve contains information from a full static pushover analysis of the OWT support structure corresponding to the load pattern of a given MRP. It is also important to note that the vertical axis in frame (b) is the load factor rather than the total applied load. Therefore, at longer MRPs, corresponding to greater load magnitudes, the pushover curve capacity can eventually be lower than $\gamma = 1$ indicating failure of the structure under the demand at that particular MRP. In frame (c), on the other hand the vertical axis is the product of the load factor and the base shear induced by the actual wind and wave loading at a given MRP. Therefore, although the load factor at MRP = 100,000 is lower than that at MRP = 50, the base shear at MRP = 100,000 is much greater than at MRP = 50.

The procedure described above treats failure of the tower ($d\gamma/dD_{top,i} \approx 0$ and $d\gamma/dD_{trans,i} \gg 0$) and failure of the support structure below the transition ($d\gamma/dD_{trans,i} \approx 0$) as equivalent, and indeed both cases would correspond to catastrophic failures of the OWT, though perhaps with different consequences. Specifically, failure of the tower without concurrent failure of the substructure, would allow for the possible survival and reuse of that part of the structure. The possibility of tracking the mode and location of failure will be discussed as part of the two-parameter IWWA analysis and in the numerical examples. The IWWA1 procedure, as it is described here, defines failure in terms of the formation of a fully developed mechanism when $d\gamma/dD_{trans,i} \approx 0$ or $d\gamma/dD_{top,i} \approx 0$ which correspond to the formation of fully developed mechanisms in the structure. An alternate definition would track the inception of first yield in the support structure by replacing the mechanism formation criterion of step 3 with a check for first yield in the structure. Using first yield in the IWWA procedure would have the advantage of including all cases where some repair or rehabilitation of the structure would be needed, even when a fully developed mechanism had not formed.

2.2. Two-parameter IWWA

The single-parameter IWWA framework has the important advantage of using a scalar measure, MRP, of the hazard intensity and corresponding load pattern, and hence, requiring fewer simulations and less information about the environmental conditions. Here, the MRP is calculated independently for wind and wave conditions and wind and wave conditions at equivalent independent MRP are assumed to occur simultaneously. Many industry standards for offshore structures such as API and IEC documents prescribe methods for determining joint wind and wave conditions that use independently determined distributions when appropriately jointly distributed data is not available. Under realistic offshore conditions, however, it is generally unreasonable and conservative to assume that the wind and wave conditions will simultaneously correspond to the same independently determined MRP. A joint probability model could be used to calculate joint wind-wave conditions corresponding to a specific MRP. In general it is not possible to define a single combination of wind and wave conditions for a specified MRP. To allow flexibility and consideration of the full suite of possible wind and wave combinations, the two-parameter IWWA (IWWA2) framework is now introduced. Many features of the IWWA2 approach are similar to those of the IWWA1 approach, and the key difference is that the structural load induced by the wind and wave conditions is now expressed as $L(x;\mathrm{MRP}_{wind},\mathrm{MRP}_{wave})$, where MRP_{wind} and MRP_{wave} denote the hazard intensity of the wind and wave conditions, respectively. Once the hazard and loading are parameterized in this way, the procedure for generating a two-parameter IWWA surface IWWA $2(\mathrm{MRP}_{wind},\mathrm{MRP}_{wave})$ is (Fig. 2):

1. Select a series $\mathrm{MRP}_i,\ i = 1,\ldots,n_{\mathrm{MRP}}$ hazard intensities of interest.
2. For each possible pair of MRPs $(\mathrm{MRP}_{wind} = \mathrm{MRP}_i, \mathrm{MRP}_{wave} = \mathrm{MRP}_j)$ perform a static, geometrically and material nonlinear pushover analysis that yields two load deformation curves $D_{trans,ij}(\gamma)$ which gives the lateral displacement at the substructure-tower transition as a function of a load factor γ and $D_{top,ij}(\gamma)$ gives the tower top displacement as a function of the same load factor. The load factor γ is defined as for the IWWA1 approach.
3. Define the capacity of the support structure at hazard level $(\mathrm{MRP}_{wind,i}, \mathrm{MRP}_{wave,j})$ to be $\gamma_{max,ij} = \min(\gamma : d\gamma/dD_{trans,ij} \approx 0 \text{ or } d\gamma/dD_{top,ij} \approx 0)$ where $d\gamma/dD_{trans,ij}$ and $d\gamma/dD_{top,ij}/d\gamma$ are

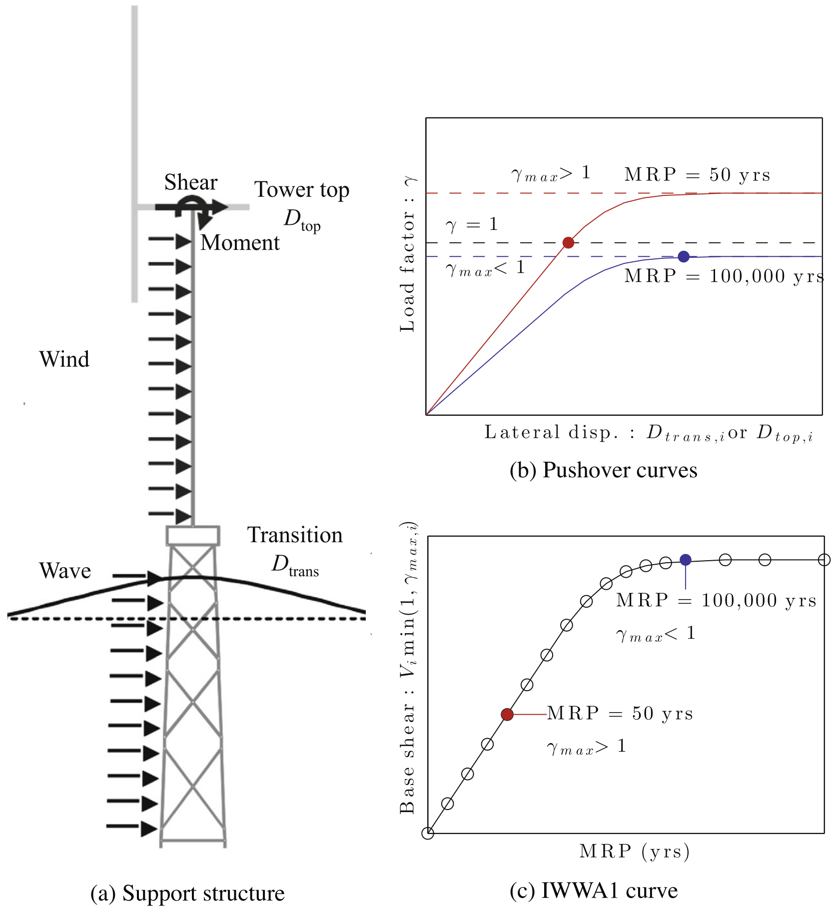


Fig. 1. Schematic illustration of the single-parameter IWWA procedure (IWWA1). (a) Schematic of an OWT support structure, in this case a jacket, with wind and wave loads indicated, including wind loads on the tower. (b) Schematic static pushover curves for load patterns corresponding to MRP = 100,000 and MRP = 50. Note that the vertical axis is the load factor γ defined such that $\gamma = 1$ corresponds to the actual loads generated by the wind and wave conditions at a given MRP. The data taken from the pushover curves for the purpose of constructing the IWWA1 curve is the point along the curve at $\gamma = 1$ when failure does not occur at the given MRP, or the point at which the pushover curves reaches the failure plateau in cases where the full load cannot be carried by the structure. (c) Schematic of an IWWA1 curve with the points taken from the pushover curves in (b) indicated. Note that the vertical axis in this case is not simply the load factor but the product of the load factor and the target base shear at a given MRP.

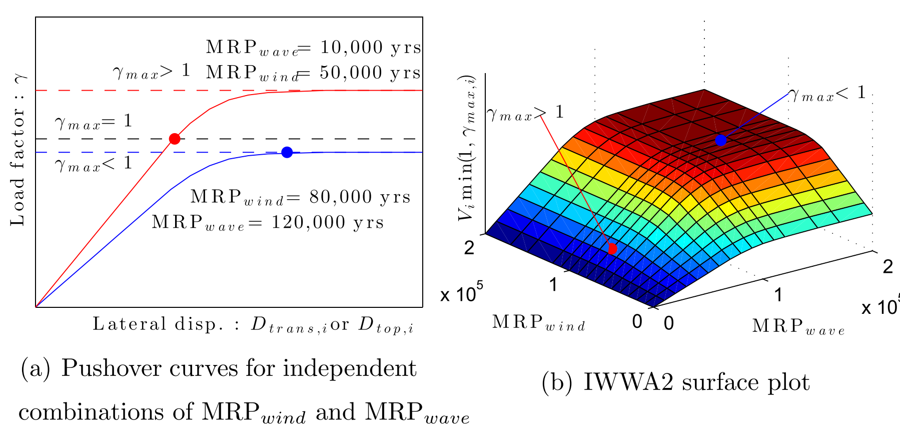


Fig. 2. Schematic illustration of the two-parameter IWWA approach. (a) Pushover curves for two different combinations of wind and wave hazard intensity. The vertical axis is the load factor which is equal to 1 when the full demand is applied to the structure. (b) The two dimensional IWWA2 surface shown on the MRP_{wind} and MRP_{wave} axes. Red and blue dots correspond to points taken from pushover curves in (a). (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

the slopes of the load displacement pushover curves and the structural capacity is reached whenever the pushover curves reach a plateau of near zero slope.

4. Define the two-parameter IWWA curve IWWA2 $(MRP_{wind,i}, MRP_{wave,j}) = V_{ij} \min(1, \gamma_{max,ij})$ where V_{ij} is the base shear generated by the wind and wave conditions at $(MRP_{wind,i}, MRP_{wave,j})$.

The IWWA2 surface is represented graphically as a plot of IWWA2 against $(\mathrm{MRP}_{wind,i}, \mathrm{MRP}_{wave,j})$. Note as before that the base moment M_{ij} could be used in place of the base shear V_{ij} in the case of a moment-dominated structure.

2.3. IWWA-based failure probabilities

The IWWA1 and IWWA2 described in the preceding subsections provide a straightforward approach for estimating the failure probability of an OWT support structure. Although the focus of this paper is on the generation of the IWWA functions themselves rather than reliability analysis, a basic approach to IWWA-based reliability calculations is provided to motivate the value of the IWWA framework.

For the single-parameter IWWA1 curve, the annual failure probability can be estimated by

$$P_f \approx \mathrm{MRP}_{fail}^{-1} \tag{1}$$

where

$$\mathrm{MRP}_{fail} = \min_{\mathrm{MRP}}(\mathrm{IWWA1}(\mathrm{MRP}) < V(\mathrm{MRP})) \tag{2}$$

in which $V(\mathrm{MRP})$ is the base shear demand under the given MRP conditions assuming no material failure. That is, the estimate of the failure probability is given by the minimum MRP hazard for which the capacity of the structure is less than the demand. Within the context of static pushover analysis of capacity, this estimate will tend to overestimate the failure probability since the IWWA1 curve has been constructed based on the assumption that wind and wave conditions with equal MRP occur simultaneously. Other considerations such as dynamic effects could in certain cases result in larger failure probabilities as could the existence of limit states other than the formation of a fully developed mechanism such as local or global member buckling, soil failure, *etc.*

For the two-parameter IWWA2 surface a similar but more intensive calculation can be made that will yield a more refined estimate of the probability of failure. The two-parameter calculation requires an estimate of the joint probability density function, denoted $f_{wind,wave}(\mathrm{MRP}_{wind}, \mathrm{MRP}_{wave})$, of the wind and wave hazard at the site in question. The joint pdf has as its arguments the MRP values for the wind and wave so that the construction of the IWWA2 surface is analogous to that of the IWWA1 curve. The joint pdf could be constructed using hub height wind speed and significant wave height as the arguments as was done in [27]. Given knowledge of the joint wind-wave pdf, the probability of failure can be estimated by

$$P_f \approx \iint_0^{\infty} f_{wind,wave}(\mathrm{MRP}_{wind}, \mathrm{MRP}_{wave}) \times I_{[\mathrm{IWWA2}(\mathrm{MRP}_{wind},\mathrm{MRP}_{wave})<V(\mathrm{MRP}_{wind},\mathrm{MRP}_{wave})]} d\mathrm{MRP}_{wind}\, d\mathrm{MRP}_{wave} \tag{3}$$

where $I_{[\mathrm{IWWA2}(\mathrm{MRP}_{wind},\mathrm{MRP}_{wave})<V(\mathrm{MRP}_{wind},\mathrm{MRP}_{wave})]}$ is an indicator variable that has value 0 when the condition $[\cdot]$ is not true, and 1 when it is. That is the indicator variable indicates whether the demand base shear is in excess of the capacity, indicating failure of the support structure.

2.4. Structural reliability index β

Standard design approaches require that the OWT structure remain elastic under the design loads (50- or 100-year conditions), while the emphasis of IWWA is on what happens to the structure during much longer MRP events. The IWWA approach is unlikely to affect the design in and out of itself, which is likely to continue to be based on 50- and 100-year loads and elastic response, but should serve to inform decision making regarding the risk profile of the structure and attendant financing, underwriting, and regulatory issues. Motivated by this purpose, a structural reliability index β is determined. Normally structural reliability analysis considers uncertainty in the loading and resistance, but here uncertainty in resistance is neglected and is assumed to be substantially smaller than uncertainty in the loading. Structural failure probability during a 20-year lifetime can be directly calculated from the 1-year lifetime failure probability obtained from IWWA analyses according to Eq. (4):

$$P_{f,20} = 1 - (1 - P_f)^{20} \tag{4}$$

When the safety margin is normally distributed, the reliability index β can be simply related to structural lifetime failure probability during 20 years $P_{f,20}$ by Eq. (5):

$$P_{f,20} = \Phi(-\beta_{f,20}) \iff \beta_{f,20} = -\Phi^{-1}(P_{f,20}) \tag{5}$$

where Φ is the standardized normal distribution function.

3. Load calculations

The IWWA procedure requires the calculation of structural loads generated by wind and wave conditions at arbitrary hazard intensities. This section describes the models used for the calculation of aerodynamic loads on the turbine and tower, and hydrodynamic loads on the support structure, transition piece or deck, and tower.

3.1. Aerodynamic loads

When the wind passes the wind turbine, it causes a lateral thrust force, a moment on the rotor and lateral forces on the tower. Aerodynamic forces generated on the rotor nacelle assembly (RNA) are transmitted to the tower as a concentrated moment and shear force pair. These forces can be calculated by blade element momentum theory [28]. In this study, the aerodynamic loads acting on the wind turbine are calculated based on a steady wind with magnitude equal to the one-minute wind at 90m hub height corresponding to the particular MRP. The aeroelastic computer-aided engineering tool FAST [29] is used to calculate the thrust force and moment through a static analysis. Although turbulence is not considered in the analysis, the probability model, which the authors use to estimate the steady state wind speed, is taken from a time-averaged measurement from a realistic turbulent wind condition. This approach taking a time-averaged wind speed and combining that with the maximum wave expected during a sea state, is similar to DLC 6.1c of IEC [30], a widely used standard to determine the magnitudes of wind and wave for the design of offshore structures. This DLC combines the maximum wave with a reduced wind to reflect the improbability of simultaneous maxima.

It is important to note that wind loads at the rated speed are larger than those at wind speeds just above cut-out due to feathering of the blades, however, it has been calculated that at higher wind speeds, the mudline bending moment generated by wind load on the parked and feathered turbine will exceed that generated at the rated wind speed [31]. For the wind loads generated at the rated wind speed to be important for the failure analysis of the support structure the rated wind speed would have to occur in concert with extremely large waves (e.g. 15–25 m waves). This is a very low probability event. Therefore, in the current study the aerodynamic calculation is always made for the parked and feathered rotor condition representing the likely state of the rotor during the extreme conditions considered in this paper.

The additional wind force per unit length f_w on the vertical wind tower member can be expressed as

$$f_w = \frac{1}{2}\rho_a C_f A(z) W_s(z)^2 \tag{6}$$

where ρ_a is the mass density of air, C_f is the shape coefficient of circular section which depends on Reynolds number and is set to be 0.7 for circular sections considered here according to the recommendation of the DNV specification for extreme offshore wind conditions [32], $A(z) = D(z)$ is the projected area enclosed by the boundary of the member normal to the direction of the force, which, for a unit length, is simply the tower diameter $D(z)$. Finally $W_s(z)$ is wind velocity at a height z above the mean water level and can be determined from the wind speed at hub height as

$$W_s(z) = W_s(z_{\text{hub}})\left(\frac{z}{z_{\text{hub}}}\right)^{\alpha} \tag{7}$$

in which for the normal wind conditions, the power law exponent, $\alpha = 0.14$ and $W_s(z_{\text{hub}})$ is the wind speed at the hub height.

3.2. Hydrodynamic loads

Beside aerodynamic loading, the other significant load on OWT support structures is hydrodynamic, induced by waves or current. Current is neglected in this study, and the following is a description of the approach used here for calculating wave loads on the OWT support structure. The approach is based on a modification of Morison's equation [33] to calculate wave forces on the members of the support structure, which are assumed to be cylindrical, and an approximation to the loads generated when a wave interacts with the transition piece of platform of the support structure. Breaking waves are not considered, and for the example site conditions used, none of the extreme wave heights considered exceed a standard threshold for wave breaking [21]. While calculation of such forces on monopiles is relatively straightforward, it is substantially more complicated for jacket structures. To allow approximate calculation of hydrodynamic forces on jackets, it is assumed that the wave crest acts simultaneously on all members of the jacket–this approximation is justified due to the long wavelength of extreme waves relative to lateral dimensions of the jacket–and that upstream elements do not disturb the flow field to which downstream elements are exposed.

3.2.1. Wave loading on the three dimensional inclined cylinder

It is assumed that the primary structural members of the support structure are cylindrical in cross section and that the wavelength of the incident waves is long compared to the dimensions of the structural member and, in the case of a jacket type structure, long also in comparison to the overall structural dimensions.

When the structural dimensions is smaller than one fifth of wavelength and the structure does not significantly affect the flow velocity field, the Morison equation [33] may be used in prediction of hydrodynamic forces. Morison's force for a fixed structure in an oscillatory flow consists of two force components due to inertia and drag, and may be approximated as

$$f = \frac{1}{2} C_D \rho D |u| u + C_M \rho \frac{\pi D^2}{4} \dot{u} \tag{8}$$

in which ρ is the fluid density, C_D is the drag coefficient, C_M is the inertia coefficient, u and $\dot{u}$ are the horizontal velocity and acceleration of the fluid particle, and f is the force per unit length acting on the cylinder. The estimation of drag and inertia coefficients is complicated and these quantities are often considered to be random variables [34]. Here, since the primary consideration is a study of the structural response and capacity of the support structure, deterministic values of $C_D = 1.2$ and $C_M = 2.0$ are used [35] and resulting hydrodynamic forces are calculated using the standard Morison equation modified as in [36] to account for inclination of the cylinder to the flow direction.

3.2.2. Wave-in-deck loading

Because the IWWA approach presented here is intended to capture response of OWT support structures in extreme environments, the possibility of waves that reach and interact with the transition piece of the support structure must be considered. In the case of a monopile structure, though there is a small platform and change in diameter at the transition piece, the wave forces generated by that interaction are neglected and it is assumed that the pile diameter is constant up to the base of the tower. For a jacket structure, on the other hand, the deck that transitions from the jacket to the tower is substantial, and wave-in-deck loading cannot be reasonably neglected. The presence of wave-in-deck loading may also substantially affect the load pattern, which in turn affects structural capacity. Vertical loads due to wave-deck interaction are not treated and only horizontal loads have been considered herein since the vertical loads have only a modest effect on capacity and are of greater concern in detailing of the deck [37].

The wave-in-deck force is calculated using a Morison-type approach that is recommended in many industry standard [20,30]. In this model, wave-in-deck load per unit length is given by

$$f_{\text{deck}}(z_0) = \frac{1}{2} C_D \rho B(z_0) u(z_0)^2 \tag{9}$$

where C_D is the drag coefficient of the deck and is set to be 2.0 according to API guidelines, z_0 is the height above the bottom of the deck, $B(z_0)$ is the width of the wetted area at z_0, and $u(z_0)$ is the horizontal particle velocity at z_0. This formula has to be used first by comparing the wave elevation relative to the deck vertical position and then judging whether the wave contacts the deck. Total wave-in-deck load can be calculated by

$$F_{\text{deck}} = \int_0^{z_c} f_{\text{deck}}(z_0)\, dz_0 \tag{10}$$

where z_c is the height from wave crest to bottom of transition piece.

It is possible that under very extreme conditions a wave may overtop the transition piece and interact with the OWT tower directly. Forces arising from such interaction can be approximately calculated using the classical Morison equation, noting that this approximation neglects the very real effect that the particle velocities and accelerations would be highly distorted above the transition piece.

3.2.3. Determination of wave kinematics

In order to use the Morison equation, one needs to obtain the wave kinematics, i.e. the velocities and accelerations of fluid particles in the wave. In general these kinematics vary in space and time. In this paper, the velocities and accelerations are deduced from a nonlinear wave elevation using stream function theory [38,39]. Loads on very small diameter components such as those of jacket platforms and jacket-template structures in deep water are generally drag dominated, but large diameter components such as monopiles in shallow water are large enough to incur both inertia and drag loads [40]. Therefore, the location of the fluid particle velocities and accelerations along the wave profile corresponding to the maximum wave force is different for jacket and monopile support structures.

Fig. 3 illustrates the location of the normalized wave induced base shear of a jacket and a monopile supported OWT on the wave profile. Although the location along the wave profile where maximum moment occurs can in principal differ from that where maximum base shear occurs, in practice the difference is very small and is therefore neglected here. Through comparison of the shear force along the wave length it can be seen that maximum forces for a jacket structure are generated at the wave crest, but slightly after

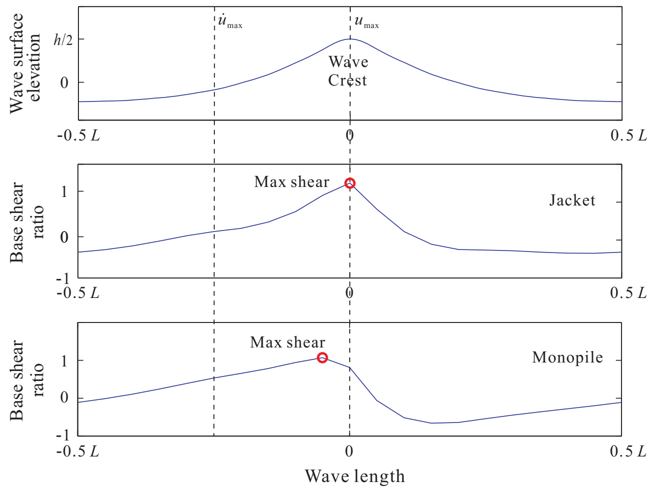


Fig. 3. Normalized wave-induced base shear on the wave profile.

the wave crest for monopiles. Forces generated at the location of maximum force are used for all calculations presented here.

The period T combined with the extreme wave height H_e from the environmental model of reference site are calculated following the formula

$$T = 11.1\sqrt{H_s/g} \tag{11}$$

in which significant wave height $H_s = H_e/1.87$. Wave length can then be calculated by stream function with the extreme wave height H_e and period T.

4. Example support structures and site conditions

Two support structures for the NREL 5MW turbine are described here and used to illustrate the IWWA procedure. The first, a monopile, is assumed to be located off the coast of the state of Delaware in 30m of water, while the second, a jacket, is assumed to be located off the coast of the state of Massachusetts in 50m of water.

4.1. NREL 5MW turbine

The 5MW baseline wind turbine designed by National Renewable Energy Laboratory (NREL) is used as the tower and rotor-nacelle-assemble (RNA) for both OWT support structures in this study. The baseline wind turbine is a variable-speed, collective pitch controlled horizontal axis wind turbine. The hub height is 90 m above the mean sea level. The nacelle above the hub has a diameter of 3 m and a total mass of 240,000 kg. The Rotor with three blades has a diameter of 126 m and weighs 110,000 kg. More detailed section configuration of tower and nacelle can be found in the report of NREL [41].

The density of steel of which the tower is constructed is taken to be 8500 kg/m^3 to include paint, bolts, welds and all other additional masses that are not otherwise considered [8]. The Young's modulus is 210 GPa and the effective yield stress is 230 MPa for the steel used.

In cases where a finite element discretization of the tower is needed, the tower is modeled using Euler-Bernoulli beam elements, and the RNA assembly is modeled as a concentrated mass with rigid links used to provide the correct offset of RNA masses from the tower centerline.

4.2. Monopile

4.2.1. General configurations and assumptions

The first support structure selected for study is a monopile to be placed in 30 m of water depth. The monopile has diameter of 7 m and a wall thickness of 0.03 m. The monopile is assumed to fully fixed at the seafloor and to extend approximately 20 m above the waterline to the point where the NREL tower begins. The transition piece is neglected in this model. Material properties are assumed to be the same for the monopile as for the tower. Fig. 4 gives overall dimensions of the NREL turbine, tower, and supporting monopile. Failure of the monopile is assumed to occur when the demand moment at a cross section equals or exceeds the plastic moment of the section as calculated to include interaction of moment and axial force due to gravity.

4.2.2. Reference site and environmental model

The site selected for study of the monopile supported OWT is off the coast of the state of Delaware, at location 38.461 N 74.703 W, where data buoy 44009 (Delaware Bay) of the US National Oceanic and Atmospheric Administration (NOAA) is located. The water depth is 30 m and the distance to shore is 30.3 km. Environmental conditions at the site are modeled based on 27 years of continuous wind and wave data collected by the buoy. The raw data consists of 8 min average wind speeds at 5 m elevation reported hourly and 20 min values of the significant wave height. These data have been converted to 1 min average wind speeds at 90 m elevation by multiplying by a factor of 1.608 that accounts for change in elevation and averaging time [42] and to extreme wave heights by multiplying the significant wave height by a factor of 1.87 [30]. To extract

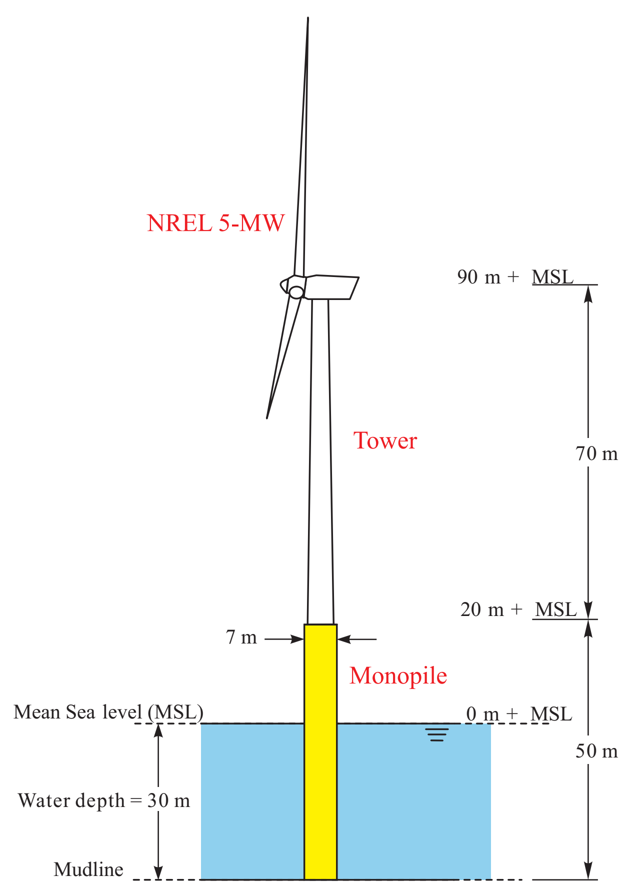


Fig. 4. Schematic of monopile supporting structure. Dimension of monopile supporting wind turbine.

wind and wave conditions at arbitrary mean return periods, as required in the IWWA procedure, the independent annual maxima of the wind and wave conditions have each been fit with a generalized extreme value distribution. Fig. 5 shows the site data (also for the site selected for the jacket) with values of the wind and wave intensity for a range of MRP values.

4.3. Jacket

4.3.1. General configurations and assumptions

The second support structure investigated here is a jacket. The jacket foundation support structure for the NREL 5.0 MW turbine is based on a reference design carried out by the UpWind project of European Union [43]. The UpWind jacket shown in Fig. 6 is designed for a 50 m deep water site. The jacket consists of four legs, four levels of X-braces and cross braces. The top and bottom widths of the jacket are 8 m and 12 m, respectively. A rigid concrete block with a weight of 666,000 kg and a dimension of $4 \times 9.6 \times 9.6$ m is positioned on top of the jacket as the transition piece or platform connecting the jacket with the tower of baseline wind turbine. The jacket is assumed to be rigidly fixed at the mudline. The properties of the jacket components are shown as described in Table 1 which corresponds to Fig. 7(a).

The jacket is made of a medium grade structural steel, of which the Young's modulus is 210 GPa and the minimum and effective yield stress is 345 MPa and 380 MPa, respectively and an ultimate stress of 510 GPa at a strain of 0.11. The density is assumed to be 8500 kg/m^3 for the same reason given in the monopile case. The turbine, transition piece and jacket are discretized into three dimensional (3D) beam elements by SAP2000 [44]. The geometric $P - \Delta$ nonlinearity and elastic-strain-hardening material model are included.

The joint connectivity of jacket members is considered as perfectly rigid. In order to simulate post-yield behavior in nonlinear pushover analyses, concentrated hinges that include full interaction between the axial force and bending moments [44] are assigned to the member joints, which may experience nonlinear behavior under wind and wave loads. Elastic behavior occurs over member length, then deformation beyond the elastic limit occurs entirely within the discrete hinge locations. The length L_p of the hinges at the ends of the jacket members are determined according to [45].

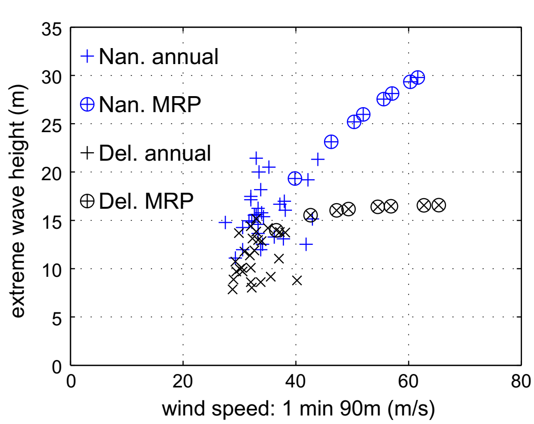


Fig. 5. Site environmental conditions for Delaware (Del.) and Nantucket (Nan.) sites used to investigate monopile and jacket response, respectively. Markers without circles give the scatter plot of annual maxima and circled markers give the values of the wind and wave conditions at MRP = [10, 100, 500, 1000, 5000, 10,000, 50,000, 100,000] years. MRP values are derived from independent best fit generalized extreme value distributions to the wind and wave data provided from NOAA buoys.

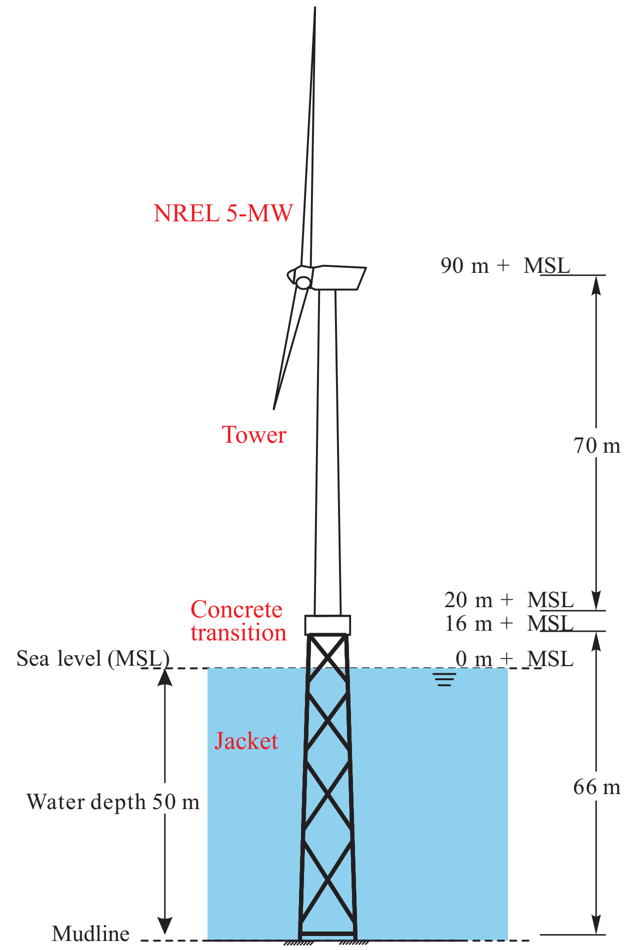


Fig. 6. Schematic of jacket supporting structure. Dimension of jacket supporting wind turbine (unit: m).

Table 1
Properties of jacket members.

Component	Color in Fig. 7(a)	Diameter (m)	Thickness (m)
Leg at lowest level	Gray	1.30	0.055
Leg 2nd to 4th level	Blue	1.15	0.030
Leg node	Green	1.28	0.043
Braces	Purple	0.73	0.020

$$L_p = \frac{L}{4}(1 - 1/f) \tag{12}$$

in which L is the total length of the member and f is the shape factor of circular section with a value of 1.27. For the purposes of this study the formation of a fully developed mechanism is the only lateral failure mode considered. Other failure modes such as member buckling or connection failures have not been considered and dynamic effects are excluded.

4.3.2. Reference site and environmental model

The site selected for study of the jacket supported OWT is off the coast of the state of Massachusetts, at location 40.502 N 69.247 W, where NOAA data buoy 44008 (Nantucket) is located. Water depth is 65.8 m, which is approximated as 50 m in this study to conform to the water depth for which the jacket structure was designed. The distance to shore is approximately 120 km. Environmental conditions at the site are modeled based on 31 years of continuous wind and wave data collected by the buoy.

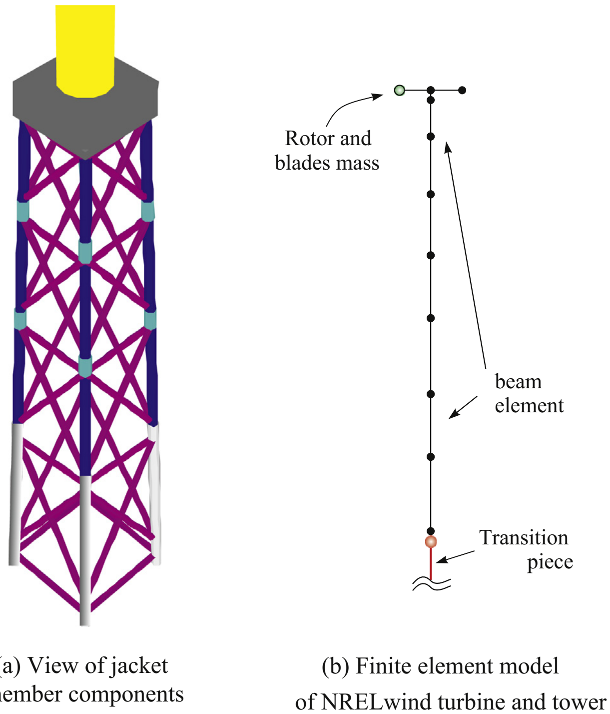


Fig. 7. Structural model of jacket supporting structure: (a) view of jacket member components in color and (b) finite element model of jacket.

The raw data are in the same format and converted in the same way as for the Delaware site, and wind and wave conditions at arbitrary MRPs have similarly been extracted from independent GEV distribution fits. Fig. 5 shows the site data and illustrates the highly site-specific nature of the environmental conditions and the IWWA procedure since the chain of wind-wave MRP values (circled markers) at the two sites not only do not fall on straight lines, but are quite separated from one another. Overall, the figure illustrates that the Nantucket site is relatively wave dominated while wind plays a greater role at the Delaware site.

4.4. Wave amplitudes and structural geometry

Fig. 8 shows, to scale, incident waves corresponding to MRP = [10, 100, 500, 1000, 5000, 10,000, 50,000, 100,000] years for the Delaware monopile and the Nantucket Jacket. The greater severity of the wave conditions at the Nantucket site is readily apparent and the importance of modeling forces generated by interaction of waves with the deck of the jacket is illustrated by the fact that the 100-year wave is high enough to reach the deck. The wavelength of the incident waves is quite long relative to the lateral dimensions of the monopile and jacket, supporting the assumption that the location along the wave crest that generates maximum Morison forces acts simultaneously on all members.

5. Results and discussion

5.1. Single-parameter IWWA

Fig. 9 shows the IWWA1 curves for the example monopile and jacket structures. The figure shows both the base demand (shear for the jacket and moment for the monopile since monopile failure is evaluated with respect to the plastic moment of the cross section) versus MRP and the load factor γ_{max} versus MRP. Downcrossing of 1.0 by the γ_{max} curve or plateau of the base demand curve indicate the MRP at which the support structural fails. In the case of the monopile this is approximately MRP $= 4 \times 10^8$ years, and in the case of the jacket it is approximately MRP $= 1.8 \times 10^5$ years.

5.2. Two-parameter IWWA

Fig. 10 shows the IWWA2 surfaces for the monopile and jacket examples as well as the corresponding γ_{max} surfaces. One can immediately discern that the jacket structure, both by virtue of its structural configuration and site environmental conditions, is a wave dominated structure while the monopile is a wind

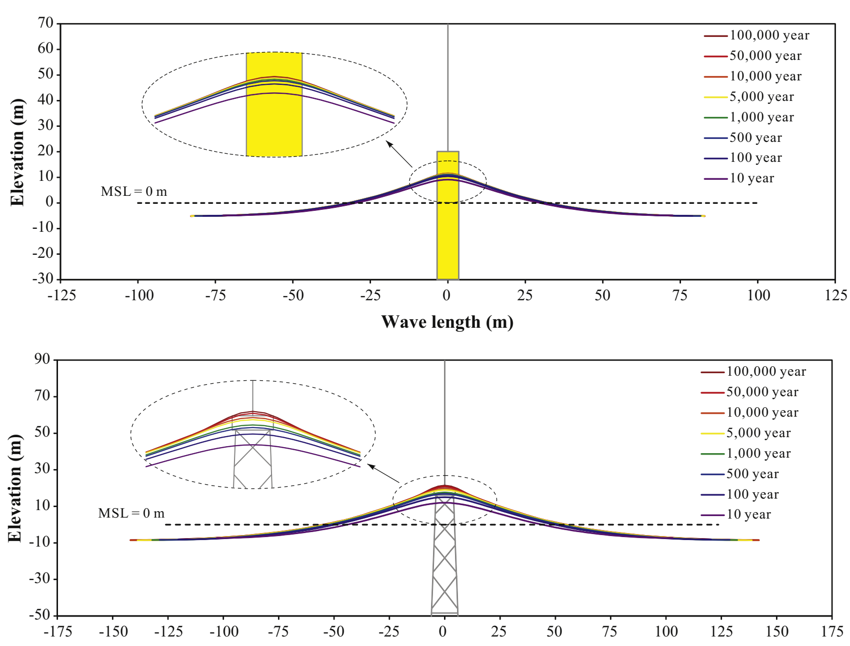
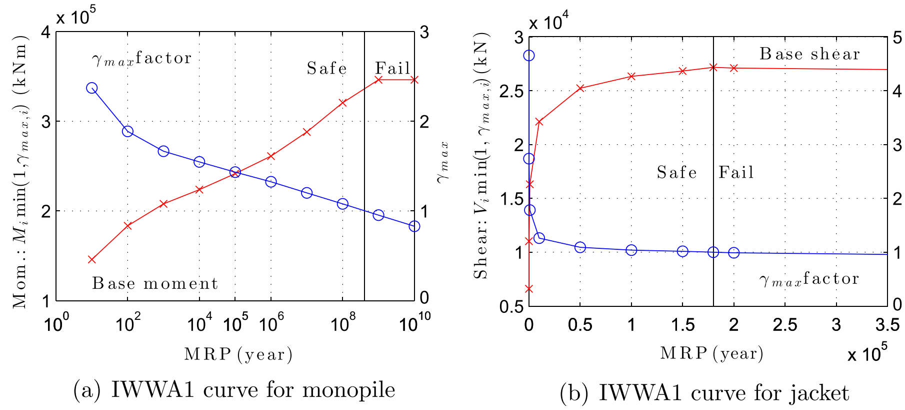


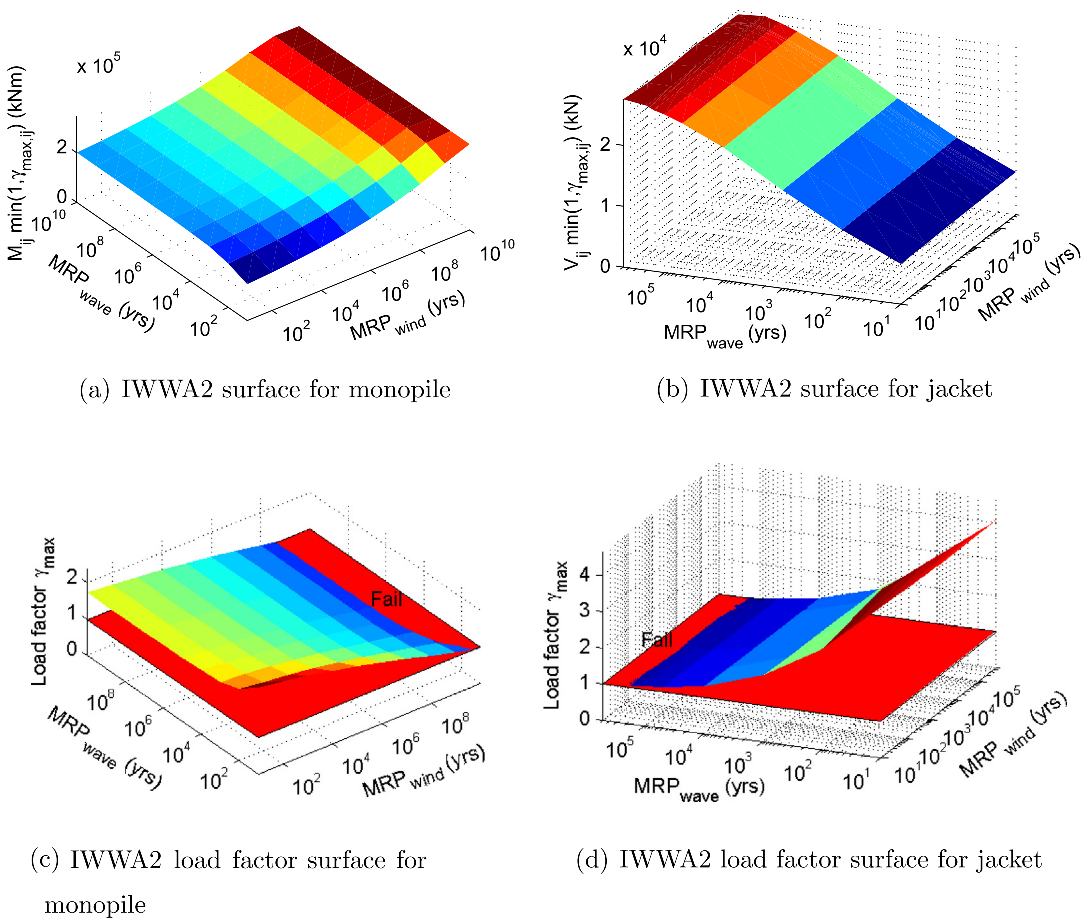
Fig. 8. Wave heights at various MRPs shown to scale against the monopile and jacket structures. Note specifically the interaction of waves with the deck of the jacket structure.



(a) IWWA1 curve for monopile

(b) IWWA1 curve for jacket

Fig. 9. IWWA1 curves for monopile and jacket examples. Lines marked with 'x' show the relationship between base demand and MRP of wind and wave conditions while lines marked with 'o' show the relationship between load factor γ_{max} and MRP. Failure occurs for MRPs greater than that at which the base force curve plateaus or the γ_{max} curve downcrosses 1.0.



(a) IWWA2 surface for monopile

(b) IWWA2 surface for jacket

(c) IWWA2 load factor surface for monopile

(d) IWWA2 load factor surface for jacket

Fig. 10. IWWA2 curves for monopile and jacket examples and corresponding γ_{max} surfaces. The γ_{max} surfaces are cut by a plane at $\gamma_{max} = 1.0$ to indicate the contour at which failure occurs.

dominated structure. The factors contributing to these findings are the difference in water depth and wave condition severity at the two sites. Essentially, the shallower (Delaware) site has less wave intensity and therefore becomes wind dominated.

5.3. Load pattern effects

The key motivation for the IWWA approach is that the load pattern applied to an OWT support structure is highly non-proportional with increased load intensity. Specifically, wave loading patterns are non-proportional as wave height increases and continuously varying combinations of wind and wave intensities can occur with non-zero probability. Fig. 11 illustrates the potential shortcoming of adopting a simple static pushover approach using a proportionally increasing load pattern to assess the capacity of an OWT support structure. The figure shows that if the capacity of the jacket structure were assessed using load patterns corresponding to wind and wave MRPs of 100 years the capacity would be overestimated by 5% to 10% compared to the capacity derived corresponding to other load patterns. To give an

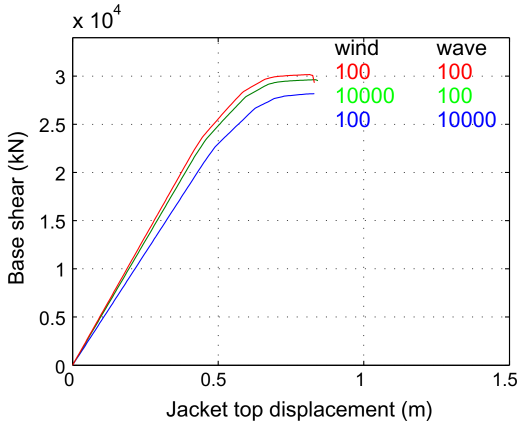


Fig. 11. Static pushover curves for jacket structure under different load patterns.

idea of how the load patterns themselves differ, Fig. 12 shows the wave and wind forces on the jacket structure for the corresponding wind and wave MRP pairs.

5.4. Structural reliabilities

Following the procedures described earlier in this paper, and for the particular structures and site conditions used in the examples, structural reliabilities have been estimated using the IWWA1 and IWWA2 approaches. Table 2 gives the reliabilities assuming a 20-year service life and independence between the years of that service life. The reliabilities for the jacket structure are reasonable while for the monopile they are extremely high, likely due to the fact that monopiles are often designed primarily to provide adequate stiffness, resulting in significant overstrength [21]. It should also be noted that the method used here for estimating extreme conditions based on 20–30 years of continuous data is a very coarse approximation of the long MRP hazard. Reliabilities obtained by the IWWA2 approach are slightly larger than those obtained using the IWWA1 approach. This is consistent with the assumptions of the two methods in that the IWWA1 approach assumes that wind and wave conditions of equivalent, independently estimated, MRP occur simultaneously, a more severe condition than assumed in the IWWA2 approach. The reliabilities obtained by IWWA1 and IWWA2 approaches are very close for the examples shown here because the jacket structure is heavily wave dominated and the monopile structure is heavily wind dominated. For a structure with more balanced response to wind and wave conditions, a greater difference between IWWA1 and IWWA2 reliabilities would be expected.

Table 2
Example structural reliabilities for monopile and jacket structures.

Structure	IWWA1 β	IWWA2 β
Monopile	5.49	5.52
Jacket	3.72	3.77

6. Conclusion

This paper has introduced the concept of an incremental wind-wave analysis (IWWA) of the capacity of OWT support structures. The IWWA approach uses static pushover analysis of the offshore structure subject to load patterns determined from wind and wave combinations corresponding to increasing mean return periods (MRP). Such an approach is required since these load patterns are non-proportional with respect to increases in wave height and with respect to wind and wave combined loading. The IWWA procedure allows the determination of the MRP (and corresponding wind and wave conditions) that leads to formation of a fully developed mechanism in the structure, and also provides a direct path to assessing the probability of failure provided that a probabilistic model for site environmental conditions is available. The IWWA approach can be applied as a single-parameter approach in which the MRP for the wind and wave conditions is assumed to be the same or a two-parameter approach in which the MRPs associated with wind and wave conditions are related to a joint

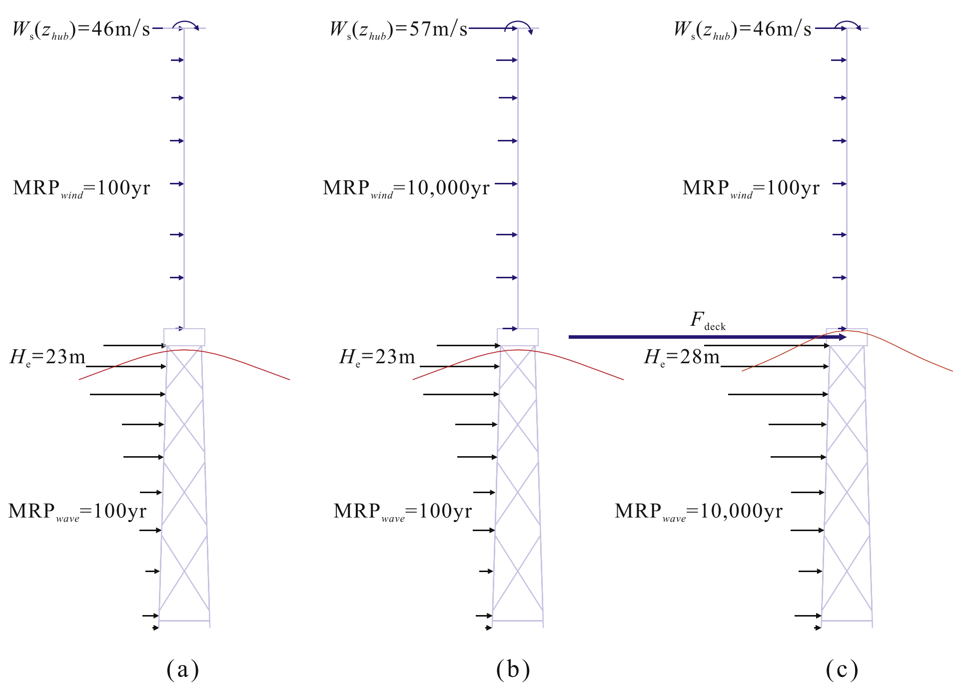


Fig. 12. Wind and wave load patterns on jacket structure. Note that for graphical clarity the scale of the wind forces is twice that of the, generally larger, wave forces.

pdf characterizing the wind and wave conditions at the site. Example calculations are performed within the constraints of static pushover curves, the single limit state of a fully developed mechanism that arises from the formation of plastic hinges, and assumed deterministic loading at a given MRP. These calculations illustrate that the results of an IWWA analysis are site specific and that structural response can be dominate by either wind or wave conditions depending on structural characteristics and site conditions. A study of the jacket structure shows that neglecting the variability of load patterns with changes in MRP can lead to significant error in estimation of the structural capacity.

Note that the forcing functions acting on the structure are relatively broad-band and that simple approximations to the dynamics such as the application of amplification factors are not likely to be accurate. Therefore, consideration of the dynamics in a realistic way requires nonlinear dynamic time history analysis, which is very time-consuming. The complex dynamic effect of wind and wave loads are therefore neglected in this study. It is important to note that the IWWA estimation without dynamic effects might over- or under-estimate the structural responses. The significance of these dynamic effects on structural ultimate response should be emphasized in the later research. Moreover, another important factor of the offshore structural capacity analysis is structural fatigue performance, which is beyond the scope of this paper. It should not be neglected in the global risk/failure analysis of OWT support structures.

Acknowledgments

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