Comparative Structural Reliability Analysis of Monopile-supported Offshore Wind Turbines – considering the Effect of Flexible Foundation and Simulation Time on Long-term Extreme Response Extrapolation

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ABSTRACT: This paper deals with the development of semi-probabilistic design formats for ultimate limit state design of monopile-supported Offshore Wind Turbines. The effect of the flexible foundation and simulation time on long-term extreme response extrapolation and their model uncertainties from structural reliability analysis perspective are addressed. The benchmarked NREL 5-MW monopile OWT, designed according to IEC 61400-3-1, is considered as a case study. Time-invariant structural reliability analysis is conducted for this case to illustrate the variation in implied safety level for various characteristic wave-to-wind moment ratio while considering the uncertainties in simulation time and foundation's flexibility on the extreme response extrapolation. The results indicate that the definition of bias in extreme response extrapolation due to simulation length can have an important effect on the implied safety level. The effect of foundation's flexibility model on the implied safety level is found to be marginal for this specific case, however, a systematic investigation is still required to document this effect. In general, structural reliability analyses indicate that the implied safety levels in current design standards of IEC-1400-3-1 should properly be harmonized to account for such uncertainties.

1. INTRODUCTION

Due to remote areas and harsher environment such as distance offshore (where stronger winds exist), offshore wind turbine development is far more challenging than onshore and requires careful consideration of environmental actions. While for offshore wind turbines the design is primarily based upon direct calculations of load effects and resistance and the use of safety factors and margins, it is however important that the safety format transparently reflects the uncertainties and variability inherent in loads and resistance. This aim can be achieved by applying structural reliability methods to calibrate the safety factors. Such an approach is required for the design of a robust support structure for offshore wind turbines, Cheng, P.W (2002), Sørensen et al. (2010), Sørensen et al. (2014).

The recent International Electrotechnical Commission design standard series, IEC 61400-1 (2019) and IEC 61400-3-1 (2019), are based on such a reliability approach. However, methods to account for the integration of analyses, especially including the nonlinear effects from dynamic analysis for both wave and wind for different types of support structures will imply model uncertainty that need to be assessed and documented, see e.g., Amlashi (2023) and

Amlashi et al. (2023). Only by performing such a systematic uncertainty assessment and the use of structural reliability analyses that a more explicit sets of partial safety factors can be developed.

The work in this paper focuses on monopile-supported OWTs, as this is a common type of foundation for as much as 80% of currently installed OWTs in Europe, as reported by Pineda and Tardieu (2017). The main aim of the paper is to obtain reliability estimates implied by a reliability-based design format for ultimate limit state checks of monopile-supported offshore wind turbines under global bending moment. The effect of pile-soil interaction and the uncertainty measures of long-term extreme response extrapolation are assessed.

2. RELIABILITY BASIS FOR OFFSHORE WIND TURBINES

Structural Reliability Analysis (SRA) methods have been used to make optimal decisions regarding safety and life cycle costs of offshore wind turbines, e.g., Sørensen et al. (2010). A more detailed discussion about the use of Structural Reliability Analysis (SRA) methods can be found in Amlashi (2023) and Amlashi et.al. (2023).

In general, the probability of failure is defined as the probability of occurrence in the failure domain:

$$P_f = P[g(\boldsymbol{X}) \le 0] = \iint_{g(\boldsymbol{X})\le 0} f_{\boldsymbol{X}}(\boldsymbol{x}) d_{\boldsymbol{x}} \tag{1}$$

Where $f_{\boldsymbol{X}}(\boldsymbol{x})$ represents the joint probability density function for $\boldsymbol{X}$ (a vector of $\boldsymbol{n}$ random variables) and represents the uncertainty in the governing random variables and $g(\boldsymbol{X})$ which defines the limit state function (a curved surface between the safe and failure domains in the space of basic variables). The reliability index (β) can be defined to express the safety defined in the space of random variables, as $\beta = \Phi^{-1}(P_f)$, see e.g., Madsen et al. (2006).

The present study is concerned with ultimate limit states for the monopile support structure of the offshore wind turbine. For a monopile-supported offshore wind turbine subjected to a fore-aft mudline global bending moment, the limit state function $g(\boldsymbol{X})$ is defined, such that $g(\boldsymbol{X}) < 0$ signifies failure, by:

$$g(\boldsymbol{X}) = M_u - (\varphi_w M_w + M_v) \tag{2}$$

where φ_w is a load combination factor. The load combination factor (φ_w) is used to account for the fact that the extreme values of wave-induced loads and wind-induced loads do not occur at the same time. The implication of this factor will be discussed later in this paper.

Normally, the resistance, M_u, and load effects, M_w and M_v, themselves are functions of other uncertain variables relating to geometrical and material properties, environmental conditions, and operational aspects. Therefore, the resistance and load effects would normally need to be determined by numerical procedures and are estimated in a hierarchy of analyses.

It is emphasized that the above limit state can be applied assuming that the global bending moment dominates over the (secondary) local load effects. This can be a fair assumption for most design load cases. However, the validity of this assumption should be documented when scaling up the offshore wind turbine design often using advanced numerical analysis, such as Nonlinear Finite Element Analysis. In the present study, SRA will be applied to investigate the failure probability implied by a semi-probabilistic design format for the ultimate strength of monopile-supported offshore wind turbine under global bending moment of the following form:

$$\frac{M_{uc}}{\gamma_r} - (\gamma_w M_{wc} + \gamma_v M_{vc}) \ge 0 \tag{3}$$

where M_{wc} is the characteristic wave bending moment and γ_w is the partial safety factor for the wave bending moment covering uncertainties in wave loads, M_{vc} is the characteristic wind bending moment, γ_v is the partial safety factor for the wind bending moment covering uncertainties in wind loads, M_{uc} is the characteristic bending capacity and γ_r is the partial safety factor for the bending capacity covering material, geometric and prediction uncertainties. The characteristic values are defined as a quantity associated with the probability distribution for load effects and resistance variables. Here, the characteristic value for both wave- and wind-induced bending moments refers to 50 years (IEC 61400-3-1). Different standards define different approaches for the design of structures. The main differences are associated with the definition of characteristic values for load effects and strength, quantity of partial safety factors applied, format of the limit state formulation and design procedure for defining the environmental conditions and/or calculating the loads and responses.

A design standard, in general, may be based on a set of expressions like Eq. (3) to achieve an implied failure probability as close as possible to the target failure probability. This implies that two or more sets of partial safety factors are applied. For instance, IEC 61400-3-1 wind turbine standard specifies several sets of safety factors for offshore wind turbines depending on different wind classes. The safety level obtained using a given code also depends on how the characteristic values are defined and the magnitude of the corresponding partial safety factors. The first set of safety factors would govern when wind loads dominate, and the second set governs the design when wave-induced loads dominate. The uncertainties in wave and wind loads for different sizes of wind turbines and different support structures may be different. This would in principle justify different safety factors. These issues should be reflected more transparently in a design code. The focus in this paper is on the model and statistical uncertainties in random variables.

3. LOADS

3.1. General

An offshore wind turbine, due to its functional requirements and its unique varied characteristics along the height, is exposed to different types of loads. These loads depend on many parameters, such as wind turbulence, wind fetch, wave heights and periods, size and shape of the structure and arrangement of the wind turbines, i.e., topology of wind farm, etc.

However, the loads, here mainly global, can be divided into two main categories: ***Primary (environmental-related) loads*** coming from wind, waves, currents, seismic, etc., and ***Secondary (geometry-related) loads***, i.e., rotor frequency load (1P), blade-passing frequency load (2P/3P). In this connection, the fundamental purpose of the support structure is to transfer these loads safely and effectively into the surrounding seabed via foundation in the case of a bottom-fixed offshore wind turbines or via a stationkeeping system in the case of a floating offshore wind turbine. In the following, these loads are briefly discussed, and the uncertainties associated with them are debated.

3.2. Wave loads

3.2.1. Wave kinematics and dynamic response

In general, wave kinematics can be described by either regular or irregular waves both linearly and nonlinearly. The wave in a sea state can be described with a variance spectrum. Among many spectral representations of the wave, the Pierson-Moskowitz is a classical one that can satisfactorily describe a fully developed sea state. For a partially developed sea state, however, the JONSWAP spectrum can be applied. The difference between these two spectra is the prediction of peak spectra, the JONSWAP being more pronounced, Barltrop and Adams (1991). This peak can be of importance for the response if the fundamental frequency of the support structure is close to the peak frequency, Cheng (2002).

In principle a long-term response analysis method should be applied to determine the characteristic extreme values, i.e., corresponding to the 25 years maximum for design or the annual maximum needed in the reliability analysis when annual failure probabilities are to be estimated. The long-term load effects may be obtained by combining short term distributions for all sea states and wave headings. While long-term analysis is fast for linear response, it is time-consuming for nonlinear responses. However, it has been demonstrated that only a few sea states along a contour line of the wave scatter diagram will be sufficient for design purposes, i.e., those sea states will normally govern the 20- or 100-

year maximum responses (Baarholm et al. 2002). The long-term extreme value should then be determined by considering the sea state with the largest contribution to the exceedance probability and using the 90 - 95% response fractile. In most cases the identification of important sea states can be based on linearized response analyses, while the prediction of extreme values in general should be based on nonlinear time domain analysis.

In the present calculation, the IEC61400-1& IEC61400-3-1 procedure is applied as the base case. Load effects are influenced by uncertainties relating to the environmental and operational aspects as well as hydrodynamic and structural modelling and will be discussed further in this paper.

3.3. Wind loads

Aerodynamic load on wind turbine blades due to wind is a complex phenomenon. Different numerical methods exist that account for the inflow wind velocity and the induced velocity due to the presence of the rotor, which are the two main features to be accounted for. In general, these methods can be categorized as: (1) Generalized Dynamic Wake (GDW) method, (2) Blade Element Momentum (BEM), (3) Navier-Stokes method, (4) Vortex method and (5) Simplified Panel method, see, e.g., Hansen et al. (2006), Gebhardt CG et al. 2014, Burton et al. (2011), Manwell et al. (2009), Hansen et al. (2011), Hansen MOL (2008).

Sometimes, it is of interest to establish simplified methods especially for use in conceptual studies. It has been shown that simplified methods give global responses within 10% accuracy compared with the model using the BEM method Karimirad M & Moan T (2012).

The wind can be described with a variance spectrum. Generally, there exists two families of spectrum that are most common in wind energy applications, namely: Von Karman spectrum and Kaimal spectrum. While DNV codes (DNV 2021) recommend using Kaimal spectrum, the IEC codes (IEC 61400-1) suggest using Mann spectrum (which is a modified version of Von Karman spectrum) or Kaimal spectrum.

3.4. Adapted model for characteristic wave- and wind-induced loads

From structural reliability analysis perspective, wave-induced responses are usually random and time-dependent, i.e., they are stochastic processes. It is usually assumed that the wave conditions remain stationary for a short period of time, in the order of a few hours. The wave-induced response may then be assumed stationary for the same period, provided that the system response is time-invariant in this period. Nonlinear wave theories should be used to simulate strongly nonlinear effects like impact from breaking waves. For moderate water depth and in deep water 2nd order irregular waves may be applied to model high sea states. In very shallow water the waves may significantly change their behaviour and become more nonlinear for high crests. For bottom-fixed wind turbines, high crests may be modelled using a nonlinear stream function or 5th order Stokes waves embedded in the irregular Airy wave model to give more accurate wave kinematics around the high crest, DNV-RP-0286 (2019). The model uncertainty in wave-induced loads will therefore depend on the method used for the estimation of the wave load effects and should carefully be estimated.

In the case of monopile supported offshore wind turbines, the Pile-Soil Interaction (PSI) has also an important impact on the dynamic responses of wind turbine’s structure. Among the four typical pile-soil interaction modeling methods, namely Coupled Springs (CS), Improved Apparent Fixity (IAF), Distributed Springs (DS) and 3D Solid Finite Element Model, the DS method is the most efficient one considering computational cost and accuracy, Feyzollahzadeh et al. (2016). Both systematic (bias) and random uncertainty of each method should properly be assessed.

Recently, Barreto et al. (2022) & Barreto et al. (2020) performed statistical analysis on the estimation of long-term extrapolated extreme responses in a monopile offshore wind turbine using FAST. Fore-aft bending moments (FABM) at the mudline were reported for different environmental condition sets for a reference site (Site 15) located in the North Sea center. The well-benchmarked NREL 5-MW wind turbine model supported on a monopile foundation was used for this purpose. It was found that the bias in the long-term extrapolated extreme response due to the simulation length (10 min versus one hour) is around 1.1 for speeds near to cut-out wind speed, while it can be reduced to 0.75 for wind speeds towards rated wind speed. Although the physical reason is unknown, the difference in bias could be due to the calibration of the design tools around the cut-out wind speed (design point). The bias in the long-term extrapolated extreme response due to foundation's flexibility model (IAF versus rigid foundation) can be in average around 1.05.

The uncertainty concerning the calculation of the aerodynamic loading due to wind is difficult to quantify. For instance, during storms, when the rotor is parked, the aerodynamic forces will be dependent on the final position of the blades in connection with the tower which can be uncertain due to uncertainty in control mechanism; therefore, affecting the wind turbine structural response.

As mentioned above, the limitations in nonlinearity prediction will impact the model uncertainty, for instance, the small deflection assumption in a linear aeroelastic code will provide less accurate results as compared to nonlinear aeroelastic methodologies in capturing extreme and transient loading, Gebhardt et al. (2013). Furthermore, the final position of the blades in parked position can impact the response of the structure as there will be some uncertainty in the azimuth angle of the rotor. This subsequently will influence the loading on the blades, as the wind profile is not constant. The same issue exists in yaw misalignment, therefore, influencing the load response. Moreover, the random seed used to generate phase angles used in the transform in FAST directly impacts the structural calculation.

Another issue is that the characteristic values of wind- and wave-induced loads for use in the design equation are defined as the most probable maximum values in 50 years. An alternative definition, however, is to use the maximum load case observed due to control mechanism in place. These values may be expressed by the sum of mean and k times the standard deviation, where k can be found from extreme value theory. Here, as explained before, we use an asymptotic extreme value theory (Gumbel distribution) to characterize the uncertainty related to the reference period and statistical uncertainty for both wave- and wind-induced load effects.

All the above-mentioned uncertainties are very difficult to estimate. Due to the limited data available, the measure should rather be given as an estimate in terms of a range of values for the different factors of influence. The aim here is to compare the relative impact of the level of bias and random uncertainty on the safety level.

3.5. Load combinations

Design load cases defined in IEC codes (IEC 61400-3-1), DNV code (DNV 2021) and ABS codes (ABS 2020) are based on several combinations of extreme environmental loads. The main design load cases include ULS, FLS, ALS and SLS assessment for various design conditions, e.g., power production, parked (idle), normal shutdown, emergency stop, installation, maintenance, etc. The most relevant load cases for ULS, however, covers a combination of four extreme wind loads and four extreme sea states. However, in some scenarios, such as idling, planned shutdown, and operating close to the cutout-wind speed, often the thrust force is significantly reduced, especially when pitch-controlled, i.e., the mudline bending moment will be reduced at speeds above rated-speed (Hald et al., 2009). Accordingly, the total extreme bending moment must be somewhat reduced. This calls for a load combination factor when two extreme wave- and wind-induced load effects are combined. A clear definition of this factor, however, is currently missing in offshore wind turbine codes, for example IEC 61400-3-1 does not specify any explicit load combination factor in the design code. For the sake of reliability analysis and due to lack of sufficient data, the load combination factor φ_w is assumed to be 1.0. A sensitivity study will be performed to document the effect of this on the implied safety level.

4. RESISTANCE FOR GLOBAL BENDING ULTIMATE LIMIT STATE

4.1. General

The ultimate strength of the monopile supported wind turbine can be estimated by a hierarchy of methods, such as: *Nonlinear finite element methods* of the entire wind turbine including blades, hub, nacelle, and tubular tower and monopile foundation; *Simplified linear finite element methods* of the wind turbine with calibrated correction factor to account for the nonlinearities; and *Simple closed-form formulations* without considering the progressive development of the collapse and load redistribution. Despite advances in computational methods and computer use, simplified closed-form methods are still preferable, simply because they are more practical as code formulations. However, it would be preferable, for the sake of obtaining a good calibration of codes, that the code formula is the same as the limit state function used in reliability analysis. Very few nonlinear finite element analyses have been performed for the complete wind turbine. The accuracy of the simplified method depends largely on the accuracy of predicting buckling collapse in the monopile or tower. The Main difficulties concern the modelling of initial imperfections (deflection and welding residual stress) and the boundary conditions (interaction between tower and rotor at the top and monopile and soil at the bottom). Therefore, the limited information about failures of monopile supported OWTs is of interest in judging the model uncertainty. In the following, some features relating to uncertainty modelling of structural resistance are discussed.

4.2. Uncertainty measures in resistance

From a reliability analysis point of view the true ultimate bending capacity of the monopile tower should be used in the reliability formulation. The true ultimate capacity, however, may be related to the ultimate strength determined by using simplified methods, by accounting for the model and parameter uncertainties of the strength as discussed previously. These uncertainty measures depend upon the method used to estimate the capacity. If the required characteristic capacity is used the uncertainty measures should account for the variability in the characteristic capacity. This implies that the true bending strength for the tower can be written as:

$$M_{FA,ut} = \hat{\chi}_{m,given.par} \cdot \hat{\chi}_{m,actual.par} \cdot \hat{\chi}_{m,charc} \cdot M_{FA,uc} \qquad (4)$$

in which $\hat{\chi}_{m,given.par}$ represents the random model uncertainty of the strength due to the given parameters. The mean value of this model uncertainty is obtained by values of the parameters close to the “design point” in a reliability sense. $\hat{\chi}_{m,actual.par}$ is the model uncertainty due to design parameters, i.e., it needs to be obtained by comparison to experimental results or advanced analyses.

If the given parameters are chosen close to the actual values, they will primarily represent the random uncertainty with a bias close to 1.0. $\hat{\chi}_{m,charc}$ represents the ratio of ultimate bending capacity due to predicted value based on actual design values and characteristic values. It is a random variable with a mean value and standard deviation which is determined by the difference in characteristic and actual parameters. The dominant effect is due to the yield strength, which implies a bias of 1.1 and a CoV of 0.1~0.15.

The ultimate bending strength check must be carried out in both fore-aft and side-to-side directions or any direction that the total load will dominate in the design cases. The required characteristic bending moment capacity of the tower is based on the governing failure mode in design. If fore-aft condition is governing the design, the required bending moment capacity should be used in side-to-side as well.

In lack of proper data, this correction factor for the required capacity, can be assumed to be 1.0 (deterministic).

5. STRUCTURAL RELIABILITY ANALYSIS

5.1. Structural reliability formulation and uncertainty measures

Starting out with the characteristic values for the wave bending moment and the wind bending moment together with their partial safety factors, the required characteristic capacity is estimated from Eq. (2) as follows:

$$M_{FA,uc} = \gamma_r\left(\gamma_w M_{FA,wc} + \gamma_v M_{FA,vc}\right) \tag{5}$$

where, $M_{FA,wc}$ is the characteristic fore-aft wave bending moment and $M_{FA,vc}$ is the characteristic fore-aft wind bending moment. Note that in the equation above the characteristic capacity depends on the relative magnitude of c=$M_{FA,wc}/M_{FA,vc}$. The limit-state equation with respect to ultimate failure under bending moment may then be written as:

$$g(\boldsymbol{X}) = M_{FA,u,req.} - \left(\varphi_w M_{FA,wt} + M_{FA,vt}\right) = \hat{\chi}_{m,given.par}\hat{\chi}_{m,actual.par}\hat{\chi}_{m,charc}M_{FA,uc} - \left(\varphi_w\hat{\chi}_{w,nl}B_{flex}B_{sim}\hat{\chi}_{w,stat.}\hat{\chi}_{w,lin.}\hat{\chi}_{w,env.}M_{FA,wc} + \hat{\chi}_{v,nl.}\hat{\chi}_{v,stat.}\hat{\chi}_{v,aero.}\hat{\chi}_{v,env.}\hat{\chi}_{v,geo.}M_{FA,vc}\right) \tag{6}$$

In the above the sub-indices *stat.*, *lin.*, *nl.*, *env.* and *geo* refer to uncertainties due to *statistics*, *linear*, *non-linear*, *environment-related*, and *geometry-related*, respectively.

Table 3 summarizes the random variables and parameters used in the reliability formulation. The uncertainties are chosen based on best available information as discussed before. However, it requires careful consideration regarding the distribution type as well as associated bias and randomness. The purpose here, however, is to perform a reliability comparison and to illustrate the effect of definition of random variables on the implied safety level.

Table 3. Variables in reliability formulation, values in brackets represent values used for the sensitivity analysis (CoV= Coefficient of Variation)

Variable	Uncertainty due to	Distribution	Mean	CoV
$\hat{\chi}_{m,given\ par}$	Given resistance parameters	Normal	1.0	0.1
$\hat{\chi}_{m,actul\ par}$	Actual resistance parameters	Normal	1.0	0.1
$\hat{\chi}_{m,charc.}$	Characteristics resistance	Lognormal	1.1	0.15
B_{flex}	Bias due to foundation flexibility	---	1.05	---
B_{sim}	Bias due to extreme response extrapolation (simulation length)	---	1.1 (0.75)	---
$\hat{\chi}_{w,stat.}$	Wave statistics	Gumbel	1.05	0.10
$\hat{\chi}_{w,nl}$	Wave nonlinearities	Normal	0.9	0.15
$\hat{\chi}_{w,lin.}$	Wave linear prediction	Normal	1.0	0.1
$\hat{\chi}_{w,env.}$	Wave environmental prediction	Normal	1.0	0.15
$\hat{\chi}_{v,nl.}$	Wind nonlinearities	Normal	0.9	0.15
$\hat{\chi}_{v,stat.}$	Wind statistics	Gumbel	1.05	0.10
$\hat{\chi}_{v,aero.}$	Wind linear prediction	Normal	1.0	0.1
$\hat{\chi}_{v,env.}$	Wind environmental prediction	Normal	1.0	0.1
$\hat{\chi}_{v,geo.}$	Secondary (geometry-related) loads	Normal	0.9	0.1
γ_r	Resistance safety factor	---	1.35	---
γ_w	Wave load factor	---	1.35	---
γ_v	Wind load factor	---	1.35	---
c = $M_{FA,wc}/M_{FA,vc}$	Normalized bending moment ($M_{yy,wc}/M_{yy,vc}$)	---	0.2 (0.5)	---
φ_w	Load combination factor	---	1.0 (0.85)	---

In the limit state equation above, the wave-induced bending moment is normalized by the wind-induced bending moment. The normalized wind-induced bending moment in this equation is thus equal to 1.0. As illustrated in Amlashi et al. (2023), the range of normalized wave-induced bending moment varies (c=$M_{FA,wc}/M_{FA,vc}$) for instance between 0.1 to 0.6. For the sake of reliability analysis performed here, a typical value of 0.2 and 0.5 is assumed. No load (reduction) combination factor (φ_w) has been applied in the design equation. Wave and wind load effects are assumed to be determined by direct calculations in FAST.

The reliability analyses reflect the effect of: foundation's flexibility modelling in extreme response extrapolation, statistical uncertainty in extreme response extrapolation due to simulation length and partial safety factors. In the following section, the sensitivity of annual failure probability for monopile-supported offshore wind turbine is studied.

5.2. Reference case

As a base case the design of NREL 5-MW monopile OWT according to IEC 61400-3-1 is considered. This is because it is, so far, the only well documented design case for which the design loads are benchmarked with. According to IEC 61400-3-1, there is no indication of the relative characteristic wave load to characteristic wind load ($c=M_{FA,wc}/M_{FA,vc}$). As an indicative value a relative $c=M_{FA,wc}/M_{FA,vc}=0.5$ is used. The partial safety factors given in IEC 61400-3-1 are (γ_r, γ_w, γ_v) = (1.35, 1.35, 1.35). The load combination factor is set to $\varphi_w=1.0$.

The structural reliability analysis is performed using the STRUREL program. The annual failure probability using FORM and SORM are 6.30×10^{-4} and 8.18×10^{-4}, respectively. A crude Monte Carlo simulation method with 10^6 simulations gives a value of 8.26×10^{-4} (with a CoV of 3.5%). This shows that the SORM method is accurate for this case. The results indicate that, for this base case with the safety factor of 1.35 for both loads and strength, the probability of failure is slightly higher than the IEC 61400-3-1 recommended target value of 5×10^{-4} or $\beta_t=3.3$. For the case with $c=M_{FA,wc}/M_{FA,vc}=0.2$ and $\varphi_w=1.0$ the corresponding probability of failure (SORM result) increases marginally, i.e., 9.27×10^{-4}.

5.3. Sensitivity studies

The annual failure probabilities are calculated to study the sensitivity to the definition of various parameters for monopile OWT. The parameters include the percentage ratio of characteristic wave bending moment to characteristic wind bending moment, uncertainty measures, load combination factor as well as partial safety factors. The sensitivity study is based on varying one of the safety factors at a time, while the two others are kept at their base case values, i.e., 1.35. The results are shown in Figure 1 – Figure 6.

Figure 1 shows the sensitivity of reliability level to the partial safety factors in the design format considering the effect of flexibility in foundation ($B_{flex}=1.05$) and effect of simulation time on extreme response extrapolation for different wind speeds. The bias $B_{sim}=1.1$ is assumed to be atypical value for simulations near cut-out wind speed while $B_{sim}=0.75$ is found more relevant for speeds near rated wind speed.

The base case is the one with rigid foundation with a typical 10min simulation time with no load reduction factor. For wind speeds near the cut-out, the shorter simulation length reduces the implied safety 2-3 times as compared to the base case, while accounting for the conservativism in the assessment with shorter simulation near the rated wind speed will increase the implied safety level 2-3 times compared to the base case. The effect of flexibility in foundation is, however, marginal. Use of load reduction factor of $\varphi_w=0.85$ is equivalent to non-conservatism in simulation time for near cut-out wind speeds.

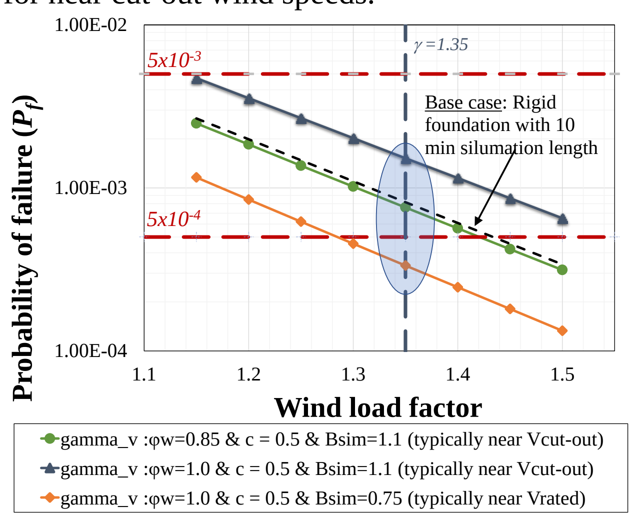

Figure 1. Sensitivity of annual failure probabilities to wind safety factor; *Solid lines*: cases with flexible foundation ($B_{flex}=1.05$) and effect of simulation time on extreme response extrapolation for different wind speeds; *Dashed line*: base case with rigid foundation and 10min simulation time and $\varphi_w=1.0$.

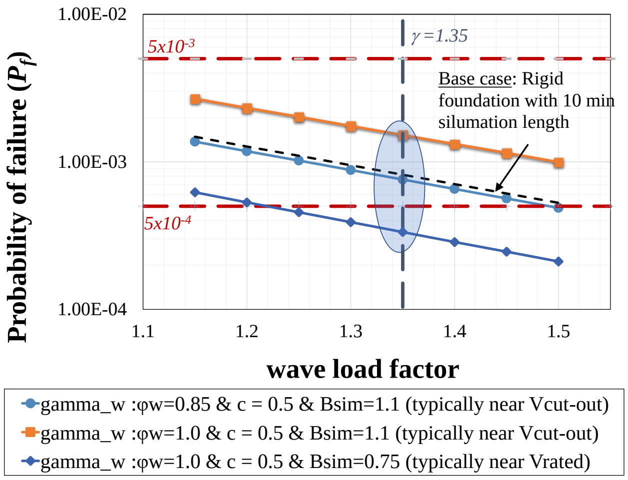

Figure 2. Sensitivity of annual failure probabilities to wave safety factor; <u>*Solid lines*</u>: cases with flexible foundation ($B_{flex} = 1.05$) and effect of simulation time on extreme response extrapolation for different wind speeds; <u>*Dashed line*</u>: base case with rigid foundation and 10min simulation time and $\varphi_w = 1.0$.

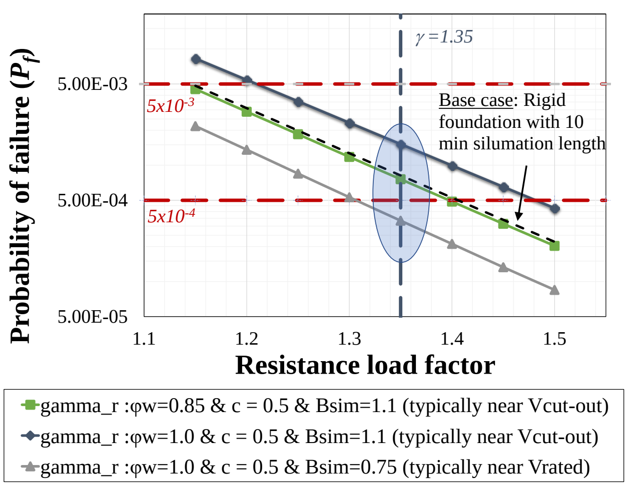

Figure 3. Sensitivity of annual failure probabilities to resistance safety factor; <u>*Solid lines*</u>: cases with flexible foundation ($B_{flex} = 1.05$) and effect of simulation time on extreme response extrapolation for different wind speeds; <u>*Dashed line*</u>: base case with rigid foundation and 10min simulation time and $\varphi_w = 1.0$.

From Figures 1, 2 and 3, considering typical IEC61400-3-1 safety factors, (γ_r, γ_w, γ_v) = (1.35, 1.35, 1.35), the safety level is seen to vary to some extent. As regards to the effect of the safety factors, the implication of γ_r, γ_w, γ_v differ. An increase of γ_r has bigger influence on the reliability level than the same percentage increase of the other two partial factors. To have the same safety level as for the case with $\varphi_w = 0.85$, the wind-induced load factor or resistance factor can be increased to 1.4 instead.

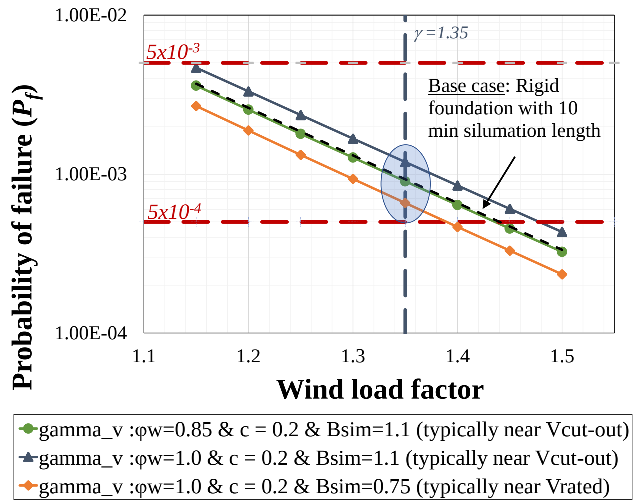

Figure 4. Sensitivity of annual failure probabilities to wind safety factor; <u>*Solid lines*</u>: cases with flexible foundation ($B_{flex} = 1.05$) and effect of simulation time on extreme response extrapolation for different wind speeds; <u>*Dashed line*</u>: base case with rigid foundation and 10min simulation time and $\varphi_w = 1.0$.

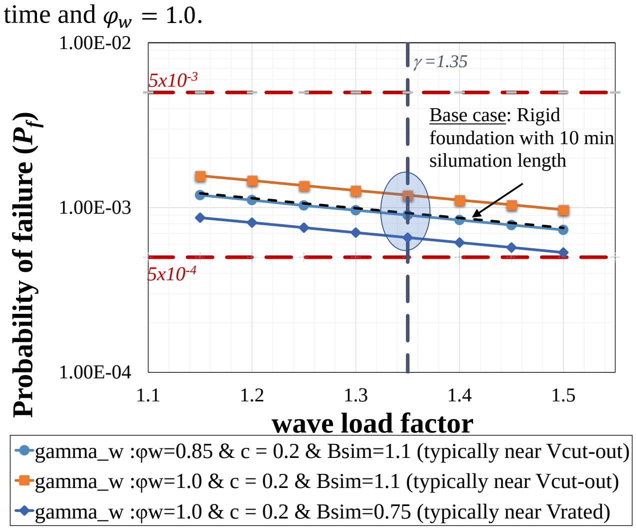

Figure 5. Sensitivity of annual failure probabilities to wave safety factor; <u>*Solid lines*</u>: cases with flexible foundation ($B_{flex} = 1.05$) and effect of simulation time on extreme response extrapolation for different wind speeds; <u>*Dashed line*</u>: base case with rigid foundation and 10min simulation time and $\varphi_w = 1.0$.

Figure 4-6 shows the sensitivity of the implied failure probabilities to partial safety factors for the corresponding case of c=$M_{FA,wc}/M_{FA,vc}$=0.2. It is seen that the effect of simulation length is less pronounced compared to the case with c=$M_{FA,wc}/M_{FA,vc}$=0.5. This can be related to the wave-to-wind moment ratio, i.e., wind-dominated design.

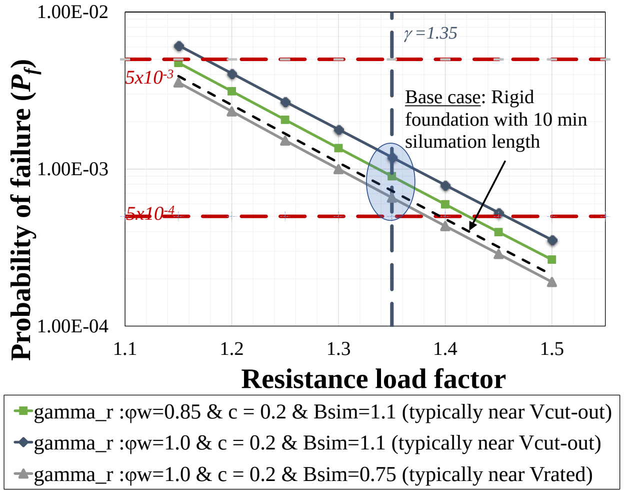

Figure 6. Sensitivity of annual failure probabilities to resistance safety factor; _Solid lines_: cases with flexible foundation ($B_{flex} = 1.05$) and effect of simulation time on extreme response extrapolation for different wind speeds; _Dashed line_: base case with rigid foundation and 10min simulation time and $\varphi_w = 1.0$.

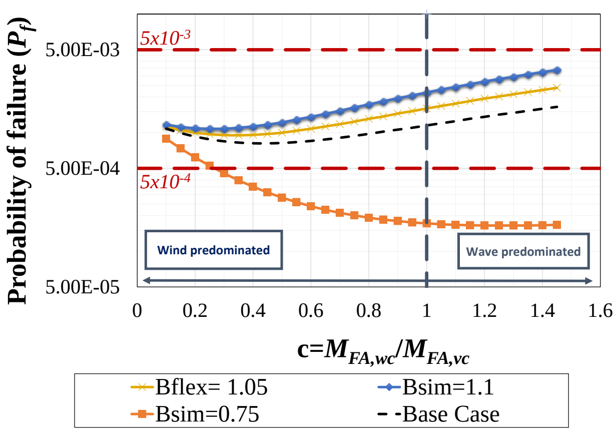

Figure 7. Sensitivity of annual failure probabilities to wave-to-wind moment ratio of $c=M_{FA,wc}/M_{FA,vc}$ for different biases associated with extreme response extrapolation due to simulation time and flexibility in monopile foundation. The partial safety factors used in the design equation are (γ_r, γ_w, γ_v) = (1.35, 1.35, 1.35).

Figure 7 below shows the sensitivity of implied failure probability to wave-to-wind moment ratio considering biases associated with extreme response extrapolation due to simulation time and flexibility in monopile foundation. It is obvious that, in general, implied safety level depends on the moment ratio and definition of systematic bias in the extreme response extrapolation. The higher the moment ratio, the larger the impact on the implied safety level will be, i.e., when wave bending moment predominates, the bias on response extrapolation near cut-out wind speed (B_{sim}=1.1) will increase the failure probability, while the bias on response extrapolation near rated wind speed (B_{sim}=0.75) conservatively reduces the implied failure probability. This shows that the extreme response extrapolation bias should be carefully defined based on the applicable range of moment ratio for the monopile OWT in hand.

6. CONCLUSIONS

A comparative reliability study of monopile offshore wind turbines for the ultimate bending strength criteria is carried out. The base case for loads and resistance factors are taken to be 1.35, i.e., IEC recommended value. The implied failure probability varies with increasing moment ratio depending on the definition of bias in response extrapolation due to simulation length, foundation's flexibility model and load combination factor. The effect of bias in response extrapolation due to simulation length and load combination factor on the implied safety level is considerable which requires careful definition of the bias through systematic investigations. The effect of foundation's flexibility model on the implied safety level is found to be not significant. However, a systematic investigation is still required to document this effect. Also, the variation on partial safety factors indicates that the implied safety levels should properly be harmonized to account for the uncertainties in random variables involved.

The effect of pitch/yaw control on the distribution of the wave- and wind-induced loads and especially the possible load truncation during storm is important. A possible truncation would affect both characteristic values of the design equation as well as the reliability model and results. Further work on the effect of control strategies and the influence of avoiding extreme loads on the wind turbine should be made. Moreover, long crested waves yield a response which might have some bias as compared to those obtained by the short-crested sea states. No bias factor of the environment has been used here, but evidence of correctness of this assumption is urgently needed.

The data on the ratio of wave to wind loads are based on the NREL 5-MW wind turbine model. To have a better statistical model for the relative magnitude of wave and wind loads, more data, especially for larger wind turbines, is needed. A load combination factor on the wave and wind loading is considered in the reliability analysis, but not in the design format. The effect of this on the implied safety level needs to be investigated properly.

7. ACKNOWLEDGEMENTS

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