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Uncertainty Modeling and Fatigue Reliability Assessment of Offshore Wind Turbine Concrete Structures

Joey Velarde and Claus Kramhøft
Marine & Foundation Engineering Department, COWI A/S
Aarhus, Denmark

Amol Mankar and John Dalsgaard Sørensen*
Department of Civil Engineering, Aalborg University
Aalborg, Denmark

In this paper, the propagation of uncertainties related to structural, environmental and fatigue damage model parameters is evaluated by performing Monte Carlo fatigue simulations of a concrete foundation for offshore wind turbines. Concrete fatigue damage models are formulated based on the S-N approach, where the resistance model uncertainty is calibrated against experimental fatigue tests. Results indicate that the resistance model uncertainty governs the concrete FLS assessment. This underlines the importance of improving estimates of model uncertainty by conducting experimental fatigue tests at lower stress cycle amplitudes and at different mean stress levels.

INTRODUCTION

In the detailed design of offshore wind turbine (OWT) foundations, the structure has to be evaluated for fatigue to ensure that the structure withstands environmental loads throughout its intended design life (typically 25 years). Current design standards are based on deterministic approaches, where partial safety factors are used to account for uncertainties in loads and resistance models. This approach, however, can either be over conservative or unsafe. It has been argued that target reliability level for OWTs can be lowered compared to other fixed offshore structures due to lower risks and consequences related to failure (Marquez-Dominguez and Sørensen, 2012). Moreover, uncertainties related to environmental inputs, which affect reliability assessments, are site-specific. To achieve more robust and cost-effective solutions, relevant sources of uncertainties have to be accounted for when performing reliability analyses and calibration of safety factors.

Several studies have been made on uncertainty analysis and its effect on structural reliability of onshore and offshore wind turbines. Toft et al. (2016a) investigated the effects of uncertainties related to wind climate parameters on fatigue loads of onshore wind turbines and concluded that these contribute to about 10%–30% of the total uncertainty in structural reliability analyses. Uncertainties due to wind resource variability were also investigated by Murcia et al. (2018), focusing on fatigue assessment of wind turbine components using polynomial surrogates. The effects of uncertainties in soil properties on dynamic response and reliability of monopile foundations has been previously investigated (Carswell et al., 2015; Damgaard et al., 2015). For OWT foundations, the relevant uncertainties have been outlined by Negro et al. (2014) and Velarde et al. (2019), which includes, among others, uncertainties related to selection of load combinations, soil properties, and wave load models. These uncertainties can have a huge effect on fatigue reliability assessment, as demonstrated by Muskulus and Schafhirt (2015), on design optimization of monopiles and jacket foundations. A study on fatigue reliability of a reinforced concrete foundation supporting an onshore wind turbine suggests that uncertainties in the material S-N curve are also important for reliability assessment, and that current design rules result in higher reliabilities than what is required for wind turbines (Marquez-Dominguez and Sørensen, 2013).

This study focuses on uncertainty modeling and reliability assessment of fatigue damage accumulation with focus on a reinforced concrete gravity-based foundation (GBF), and demonstrates the potential of using Monte Carlo-based linear regression models (Sin et al., 2009) for uncertainty and reliability analysis. Stochastic input parameters related to structural properties, soil properties, environmental wind and wave loads, and stochastic concrete fatigue damage based on the S-N approach are considered. The results provide insights on the sensitivity of fatigue loads to various input parameters, and evaluates the structural reliability with respect to concrete fatigue failure. The results of this study are relevant for reliability-based design and for calibration of safety factors for OWT concrete structures.

ASSESSMENT OF UNCERTAINTIES

In probabilistic design of structures, the load and resistance are modeled by stochastic variables to account for the uncertainties. These uncertainties are generally classified into two subgroups: (1) aleatoric uncertainties which are related to physical random processes, such as variability in soil properties, material strength, and metocean conditions; and (2) epistemic uncertainties which are related to uncertainties associated with models, measurements and statistics (due to a limited number of observations). The latter can be reduced by improving the models, by increasing the measurement accuracy, and by increasing the number of data samples (Sørensen and Toft, 2010). In fatigue design and assessment of OWT structures, both types of uncertainties have to be considered. Table 1 summarizes the sources of uncertainties considered in this study, which are uncertainties related to structural inputs,

*ISOPE Member.

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KEY WORDS: Uncertainty, reliability, probabilistic design, concrete fatigue, Monte Carlo, gravity based foundation, offshore wind energy.

Parameter	Unit	Dist	Mean	COV	Ref.*
Structural					
Steel mod. (E_s)	[MPa]	LN	2.10 E5	0.03	TH
Concrete mod. (E_c)	[MPa]	N	2.96 E4	0.06	DNV, E
Nacelle mass (M_{nac})	[kg]	N	2.95 E5	0.03	X_1
Hub mass (M_{hub})	[kg]	N	7.00 E4	0.03	X_1
Tower t_{wall} (t_{twr})	[mm]	N	1.00	0.66	EN
Damping ratio (ζ)	[%]	LN	1.10	0.12	DE
Soil					
Shear mod. (G_s)	[MPa]	LN	80	0.4	DA, DG
Poisson's ratio (ν)	[-]	LN	0.3	0.1	DG, X_1
Metocean					
Wind speed (U_w)	[m/s]	N	12.0	0.05	IEC, X_1
Turb. int. (TI)	[-]	LN	0.146	0.20	K, M, W
Wind shear exp. (α_w)	[-]	LN	0.150	0.66	T, W
Wave weight (H_s)	[m]	LN	1.55	0.07	ZI, IEC
Wave period (T_p)	[s]	LN	5.20	0.04	ZI, IEC
Water depth (h)	[m]	LN	25.0	0.03	X_1
CD factor (X_{CD})	[-]	LN	1.0	0.25	JC, MU
CM factor (X_{CM})	[-]	LN	1.0	0.10	JC
Fatigue damage					
Model (X_m)	[-]	N	1.5	0.5	X_2
Stress (X_{stress})	[-]	LN	1.0	0.1	T2, X_1
Post tension (X_{PT})	[-]	LN	1.0	0.03	E, X_1
Damage ratio (Δ)	[-]	LN	1.0	0.4	F, T2
Concrete str. (f_c)	[MPa]	LN	55	0.1	JC, X_1

*Reference abbreviations – DA: (Damgaard et al., 2015), DE: (Devriendt et al., 2013), DG: (DGGT, 2002), DNV: (DNV, 2012), E: (CEN, 2004), EN: (CEN, 2010), F: (Folsø, Otto, and Parmentier, 2002), IEC: (IEC, 2009), JC: (Joint Committee on Structural Safety, 2001), K: (Koukoura et al., 2016), M: (Müller and Cheng, 2016), MU: (Muskulus and Schafhirt, 2015), T: (Toft et al., 2016a), T2: (Toft et al., 2016b), TH: (Thöns et al., 2010), W: (Westerhellweg et al., 2014), X_1: Expert opinion/available data, X_2: Analysis of available data (see section: Probabilistic Model for Concrete Fatigue Resistance), ZI: (Ziegler et al., 2016).

Table 1 Uncertainties related to soil, structural, metocean and fatigue damage model parameters; N: Normal, LN: Log-Normal

soil properties, metocean conditions, and fatigue damage model for concrete structures.

Assumptions on probabilistic modeling of the parameters are based on existing research findings, design standards, available data, and practical experience. Uncertainties related to structural inputs generally represent both aleatoric uncertainties and tolerance levels during production. For soil and metocean inputs, the uncertainties represent not only those related to physical random variation, but also uncertainties coming from measuring devices and procedures, post-processing of measurements, derivation of key parameters, and representation of these parameters in the model and analysis. As an example, uncertainties in wave load propagate from the wave measuring device, post-processing into parameters H_s and T_p, representation in time domain analysis as lumped sea states with an assumed spectrum (as per IEC (2009) recommendations), and hydrodynamic load model used (i.e., Morison's equation). A notable reference to uncertainty modeling is the Probabilistic Model Code (Joint Committee on Structural Safety, 2001), which outlines the general principles of uncertainty modeling and reliability assessment. The model implementations of parameters summarized in Table 1 are further discussed in the succeeding sections.

METHODS

This section presents the OWT modeling, load calculation, and formulation of the stochastic concrete fatigue damage model. Monte Carlo method and fatigue reliability assessment using the S-N approach are also briefly discussed.

Wind Turbine Modeling

The Thornton Bank offshore wind farm is located in the Belgian North Sea about 30 km off the coast of Oostende, Western Flanders. The first phase consists of six 5 MW OWTs supported by reinforced concrete GBFs, which are chosen as reference OWT foundations for assessment of fatigue accumulation in concrete. The NREL 5 MW reference wind turbine (Jonkman et al., 2009) is taken as the representative of the 5 MW REpower wind turbine. The primary wind turbine properties, design elevations, and support structure design parameters are summarized in Table 2.

The OWT supported by GBF is illustrated in Fig. 1. Load calculation is performed using HAWC2 (Larsen and Hansen, 2015), which is developed for static and dynamic analyses of both onshore and offshore wind turbines. The structural analysis in HAWC2 follows a multibody formulation, where each body is represented by Timoshenko beam elements. The GBF, tower, shaft, hub, and the blades are all modeled as separate bodies. The steel tower is modeled with axis-symmetric pipes having a fixed outer diameter. Linear elastic material is assumed with mean Young's modulus $E_s = 210$ GPa and structural shear modulus $G_{\mathrm{twr}} = 80.8$ GPa. The tower has a varying thickness across the height, and the variation due to manufacturing tolerances is also represented by a stochastic parameter (t_{twr}). Similarly, the GBF body is represented by axis-symmetric pipes with increasing diameter from the ring beam down to the foundation base. Linear elastic material is assumed with mean Young's modulus $E_c = 29.6$ GPa and structural shear modulus $G_{GBF} = 15.0$ GPa. Wet and dry fill materials are represented as distributed masses along the GBF height. The structural and aerodynamic properties of the wind turbine blades are modeled according to the NREL 5 MW definition (Jonkman et al., 2009). Both nacelle mass (M_{nac}) and hub mass (M_{hub}) are modeled as concentrated masses at tower top and shaft bodies, respectively.

The structural damping in HAWC2 is formulated using Rayleigh viscous damping, also referred to as "classical damping" (Strutt, 1897). The damping matrix is expressed as a linear combination of both mass and stiffness matrices in terms of proportional coefficients α and β, respectively, as shown in Eq. 1. For simplicity, only stiffness contributions are considered in the model. The

Parameter	Unit	Value
Hub height (amsl)	[m]	91.7
Wind turbine rating	[MW]	5.0
Rotor diameter	[m]	126.0
Rated wind speed	[m/s]	11.4
Cut-in, Cut-out wind speeds	[m/s]	3.0, 25.0
Cut-in, Rated rotor speed	[rpm]	6.9, 12.1
Mean water depth	[m]	25.0
Interface elevation (amsl)	[m]	14.70
Tower height, Diameter	[m]	74.0, 5.5
Ring beam elevation (amsl)	[m]	−11.1
Ring beam diameter	[m]	6.5
GBF base diameter	[m]	23.5

Table 2 Main design parameters for the reference OWT and gravity based foundation

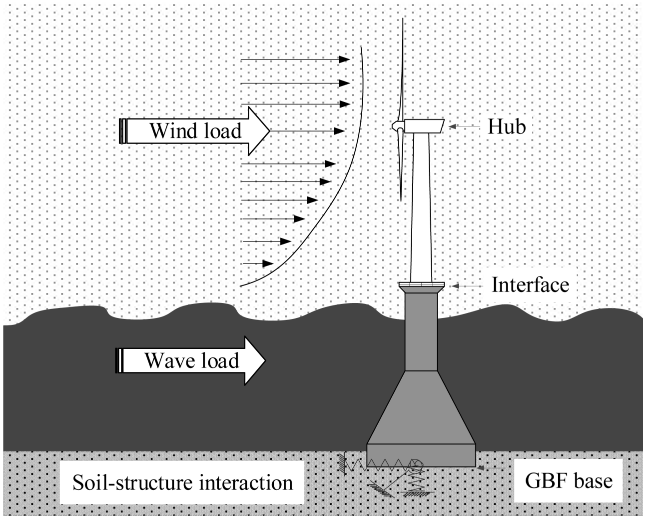

Fig. 1 Model illustration of an OWT supported by a GBF

damping coefficient (β) is tuned to represent both structural and foundation damping contributions. The assumed mean damping ratio is $\zeta = 1.10$ % for both 1st fore-aft and 1st side-side bending modes (Devriendt et al., 2013). The stochastic damping ratio (ζ) depicted in Table 1 is calculated based on structural Eigenanalysis, such that variations in other structural parameters (i.e., tower thickness, steel Young's modulus) which influence the structural damping are also considered. Note that aerodynamic and hydrodynamic damping are also included in HAWC2 when performing dynamic simulations.

$$C = \alpha M + \beta K \tag{1}$$

The foundation stiffness is determined based on recommendations outlined in DNV (2014). Based on elastic theory, the lateral stiffness (K_H) and rotational stiffness (K_R) of a GBF with base radius (R) are calculated as

$$K_H = \frac{8G_s R}{2-\upsilon}\left(1+\frac{R}{2H}\right) \tag{2}$$

$$K_R = \frac{8G_s R^3}{3(1-\upsilon)}\left(1+\frac{R}{6H}\right) \tag{3}$$

where G_s, ν, and H are the soil shear modulus, Poisson's ratio, and height of soil strata, respectively. Both G_s and ν are modeled as random parameters (see Table 1), while H is taken as a relatively large value as in the case of most sites in the North Sea. For simplicity, the foundation is modeled in HAWC2 using the apparent fixity (AF) approach, where the calculated lateral and rotational stiffness values are converted into an equivalent beam, which is fixed at a determined distance below the foundation base. The AF approach is assumed to represent the foundation accurately, particularly within fatigue load spectrum where the soil stiffness are still within the elastic range. The OWT model developed in HAWC2 is validated against the original project load model.

Fatigue Damage Assessment

IEC 61400-3 (IEC, 2009) describes requirements for the design of offshore wind turbines. The standard outlines an extensive number of design load cases to evaluate structural integrity of the wind turbines for both normal and extreme design conditions. Fatigue-relevant scenarios include design load case (DLC) 1.2, which evaluates fatigue loads during power production. For simplicity, co-directional and unidirectional wind and waves are assumed and current loads are not included. Fatigue simulations are performed using the HAWC2 model for 10 minutes at a timestep (Δt) of 0.02 seconds. Monte Carlo (Metropolis and Ulam, 1949) simulations with 250 realizations are performed.

A representative sea state with mean wind speed $U_w = 12$ m/s and correlated sea-state wave parameters ($H_s = 1.55$ m, $T_p = 5.20$ s) is chosen based on having the highest fatigue damage equivalent load, considering both load magnitude and probability of occurrence. The wind fields ($32 \times 32 \times 8192$ points) are generated at a timestep of 0.08 seconds based on the Mann turbulence model (Mann, 1998), which assumes that the energy spectrum is described by von Karman spectrum (IEC, 2005). Normal turbulence model (NTM) is applied for Class III-C wind conditions. A power law wind profile is assumed, with the wind shear exponent (α_w) also assumed as a stochastic parameter with mean equal to 0.15. For the hydrodynamic loads, linear irregular waves are generated based on JONSWAP spectrum with a peak enhancement factor (γ) set to 3.3. Wheeler stretching is applied on the wave kinematics. Random seed numbers are used to generate both wind fields and wave kinematics for each simulation.

Morison's equation is used for hydrodynamic load calculations, with the drag (C_D) and inertia (C_M) coefficients modified along the height of the foundation to account for diffraction and secondary steel. Note that the hydrodynamic coefficients (C_D, C_M) are also considered as stochastic parameters. The uncertainty associated to C_D and C_M, which is related to the calibration and the correctness of the diffraction analysis, is currently assumed based on recommendations from the JCSS (2001). A more thorough separate assessment of the wave load has to be performed to identify this uncertainty with more accuracy. This is outside the considered scope of work for this paper.

For each simulations, the concrete fatigue damage is evaluated using a modified DNV (2012) model (see Eqs. 5 to 6). The stochastic concrete damage model is calibrated against available tests as discussed in the following section.

Probabilistic Model for Concrete Fatigue Resistance

The design life for both steel and concrete structures is normally evaluated using cumulative linear damage theory by Palmgren (1924) and Miner (1945). The fatigue resistance of a material is represented by a relationship between the number of cycles to failure (N) and the stress range (S), commonly referred to as the S-N curve. The design criterion based on Miner's rule is expressed as

$$\sum_{i}^{k} \frac{n_i}{N_i} \leq \Delta \tag{4}$$

where k is the number of stress-blocks each with constant stress amplitude and number of cycles (n_i), and Δ is the cumulative damage ratio, which is equal to 1.0 for deterministic calculations. For probabilistic design, Δ can be expressed as a random variable (see Table 1) to account for the uncertainties related to linear damage accumulation (Miner's rule) as indicated by experimental fatigue tests under variable amplitude stress ranges (Folsø et al., 2002).

For concrete structures, the mean stress level is also an important parameter to consider, and the use fatigue damage models based on e.g., Goodman (1918) relation is necessary. The number of cycles to failure (N_i) is calculated based on the design guideline for Offshore Concrete Structures (DNV, 2012). For reliability assessment, it is important that "hidden safety" parameters

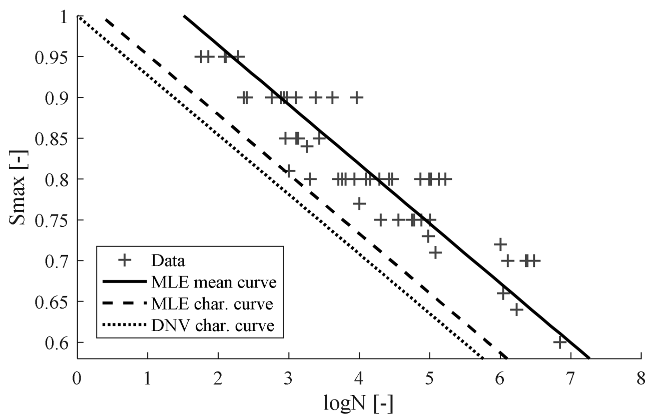

Fig. 2 Modified S-N curve based on concrete fatigue tests

are accounted for. Since the concrete fatigue resistance in DNV (2012) is a characteristic equation, the equation is modified to a limit state equation to account for both model uncertainty and physical uncertainties such that it can be used for reliability analysis. Using the Maximum Likelihood Method (MLM), calibration of the model uncertainty is performed using a compiled database of experimental tests by Lantsoght (2014), Lohaus et al. (2012), Sørensen (2011), and Thiele (2016). Test results with 20-60 MPa concrete characteristic compressive strength (f_{cck}) are used. A normally distributed stochastic variable X_m representing model uncertainty (see Table 1) is added to the fatigue resistance model as

$$\log_{10}(N_i) = C_1(1 - S_{\max_{i,j}}/(1 - S_{\min_{i,j}}) + X_m \tag{5}$$

$$S_{\max_{i,j}} = \frac{\sigma_{\max_{i,j}}}{f_c}, \qquad S_{\min_{i,j}} = \frac{\sigma_{\min_{i,j}}}{f_c} \tag{6}$$

where f_c is the compressive strength of the concrete, which is also represented as a stochastic parameter. The factor C_1 can be equal to 10 for structures "in water" having stress variation within the compression range, or equal to 12 for structures "in air." Since the database consists of concrete strength tests performed "in air," it is assumed that the same bias and model uncertainty (X_m) is applicable for offshore concrete structures exposed to seawater. In addition, experimental tests were carried out at larger cycle amplitudes than what is experienced by OWTs. It is therefore assumed that the same model uncertainty is present in the S-N curve at low amplitude cycles ($S_{\max} \approx 0.30$). The modified S-N curve (MLE mean curve) is shown in Fig. 2 with $X_m \sim \mathrm{N}(1.5, 0.5)$. Characteristic curves and available data are also illustrated.

For each realization (i), the stress time series was derived based on a simple combined flexure formula, which takes into account the axial loads (F_z), bending moment (M_x), and the post tensioning force (F_{PT}) at the critical section defined by cross sectional area (A_{cs}) and section modulus (S_{mod}). In addition, the uncertainty (X_{PT}) related to the degradation of the post tensioning force over time is accounted as follows:

$$\sigma(t) = \frac{F_Z(t) + F_{PT}X_{PT}}{A_{cs}} + \frac{M_x(t)}{X_{\mathrm{mod}}} \tag{7}$$

The additive sign indicates that the compression (upwind) side of the GBF and tower sections are considered critical for FLS analysis during power production, as illustrated in Velarde et al. (2018). Using rainflow counting to the stress histories obtained by Eq. 7, the means ($\bar{\sigma}$) and amplitudes (σ_{amp}) are calculated and are used to estimate the maximum ($\sigma_{\max}$) and minimum ($\sigma_{\min}$) stress amplitudes for each realization (i) and stress block (j) as

$$\sigma_{\max_{i,j}} = \bar{\sigma}_{i,j} + \sigma_{\mathrm{amp}_{i,j}} X_{\mathrm{stress}_i} \tag{8}$$

$$\sigma_{\min_{i,j}} = \bar{\sigma}_{i,j} + \sigma_{\mathrm{amp}_{i,j}} X_{\mathrm{stress}_i} \tag{9}$$

A stochastic parameter X_{stress} is introduced to account for the uncertainty in load and stress calculations. This two-step approach allows application of X_{stress} to the load amplitudes, which are more uncertain than the mean values. Following Eqs. 4 to 6, the total accumulated damage in concrete (D_f) can be calculated as

$$D_f(t) = \sum_i \frac{n_i(E_s, E_c, M_{\mathrm{nac}}, \ldots, X_{CM})}{N_i(X_m, X_{\mathrm{stress}}, X_{PT}, f_c)} t \cdot f_{\mathrm{occ}} \tag{10}$$

The number of stress cycles per year (n_i) is extrapolated from 10-minute simulations and is written as a function of structural, soil, and metocean parameters listed in Table 1. The parameter t is the structure lifetime in years. Ideally, the damage contribution from a range of representative environmental sea states shall be evaluated. For simplicity, only damage contribution from a single, representative sea state is considered. The total damage is estimated by introducing the pre-determined scale factor f_{occ}, which is equal to the ratio of the fatigue damage from all operational sea states to the damage contribution of the considered sea state.

The presented probabilistic fatigue damage model assumes linear damage accumulation for fatigue limit state. In addition, it is noted that the simplified concrete stress calculation applied in the study does not consider contributions from steel reinforcements and concrete confinement.

Damage Models

Concrete fatigue damage (D_f) is calculated based on DNV (2012). D_f is evaluated at the foundation-tower interface assuming that the structure is exposed to seawater. In order to evaluate the effect of the stochastic material damage model, two cases are considered: (1) D_f with Stochastic Load and Stochastic Resistance (SLSR), which considers all uncertainties defined in Table 1; and (2) D_f with Stochastic Load and Deterministic Resistance (SLDR), which does not consider the fatigue damage model uncertainty, but considers structural, soil, and metocean uncertainties as defined in Table 1. SLDR is evaluated using design values for the SN curve parameters ($f_c = 44$ MPa, $X_m = 0$). Simple linear regression is performed using least squares method to estimate the linear contributions of input parameters to D_f. To compare the sensitivity of different parameters with different dimensions, the standardized regression coefficients (SRC) are derived by performing linear regression on normalized inputs and outputs. The workflow and algorithm used for the Monte-Carlo-based linear regression is described by Sin et al. (2009).

Fatigue Reliability Assessment by S-N Approach

Based on calculated SRCs, the parameters with the largest influence are determined. The linear models for the annual fatigue damage are formulated using a reduced set of k parameters, as shown in Eq. 11. The stochastic variables and linear regression coefficients are represented by X_i and b_i, respectively. An error term (ε) is included to match the distribution of the Monte Carlo predictions. The error term is formulated by fitting a normal distribution (with $\mu = 0$) to the residual between the Monte Carlo output and preliminary linear model output.

$$D_f(\mathbf{X}, t) = \left(\sum_{i=1}^{k} X_i b_i + \varepsilon\right) \cdot t \tag{11}$$

$$\varepsilon \sim N(0, \sigma_{\text{error}}) \tag{12}$$

Based on Eqs. 4 to 12, the limit state equation based on S-N approach can be formulated as shown in Eq. 13. The parameter Δ represents model uncertainty related to Miner's rule for linear damage accumulation, which is assumed as constant for the deterministic case. Consequently, the cumulative probability of failure (P_F) is estimated using the First Order Reliability Method (FORM). The corresponding reliability index (β) is related to P_F as shown in Eq. 15, where Φ is the standard normal distribution function (Madsen et al., 2006).

$$g(\mathbf{X}, t) = \Delta - D_f(\mathbf{X}, t) \tag{13}$$

$$P_F(\mathbf{X}, t) = P(g(\mathbf{X}, t) \leq 0) \tag{14}$$

$$\beta(\mathbf{X}, t) = -\Phi^{-1}(P_F(\mathbf{X}, t)) \tag{15}$$

Given the structure's survival up to time t, the annual probability of failure (ΔP_F) and annual reliability index ($\Delta\beta$) are determined as follows:

$$\Delta P_F(\mathbf{X}, t) = \frac{P_F(\mathbf{X}, t+\Delta t) - P_F(\mathbf{X}, t)}{(1 - P_F(\mathbf{X}, t))\Delta t} \tag{16}$$

$$\Delta\beta(\mathbf{X}, t) = -\Phi^{-1}(\Delta P_F(\mathbf{X}, t)) \tag{17}$$

where $t > \Delta t$ and Δt is the time interval in years; here $\Delta t = 1$ year.

According to the DNV-OS-J101 standard for offshore wind turbine structures (DNV, 2014), the annual probability of failure, $\Delta P_F = 10^{-4}$ ($\Delta\beta = 3.7$), can be assumed for individual failure modes. The target safety level corresponds to a normal safety class for unmanned offshore structures. For fatigue reliability assessment, $\Delta\beta = 3.1$ to 3.7 are generally acceptable (Marquez-Dominguez and Sørensen, 2012), which corresponds to $\Delta P_F = 10^{-3}$ to 10^{-4}.

RESULTS AND DISCUSSION

The analysis is performed for two cases: (1) SLSR and (2) SLDR. Comparative results are presented in this section.

Uncertainty Analysis and Models for Concrete Fatigue Damage

The objective of this subsection is to investigate which parameters are the most important with respect to the 20-year fatigue damage (D_f). Predictions of D_f using Monte Carlo simulation are shown in Fig. 3. On average, the SLDR case predicts about ten times higher D_f than the SLSR case, since the S-N curve used for SLDR is based on design standards that use design values of the uncertain parameters, whereas the SLSR case is based on mean values and scatter associated with the uncertain parameters. Despite having a lower mean, the SLSR case predictions have higher variations in D_f, due to more uncertainties included.

The SLDR linear damage model ($D_{f\,SLDR}$) is based on Eqs. 11 and 12, with significant parameters (X), regression coefficients (b), parameter ranking, model R^2, and standard deviation of the error term ($\varepsilon \sim N(0, \sigma_{\_SLDR})$) summarized in Table 3. It has been previously shown that reduction of the number of parameters does not significantly affect the linear model accuracy (Velarde et al., 2018). The proposed linear model captures about 68% of the total variations in D_f.

The SLSR damage model ($D_{f\,SLSR}$), on the other hand, is significantly influenced by the fatigue damage model uncertainty (X_m). Thus, an exponential relation gives a better fit (see

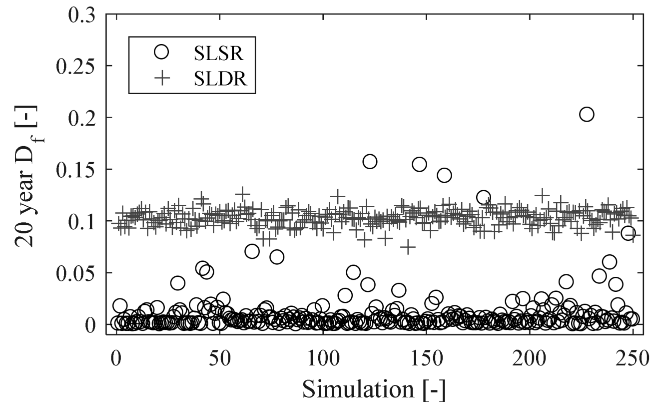

Fig. 3 Influence of damage model on fatigue damage distribution

Par.	Rank	SRC	b	Par.	Rank	SRC	b
G_s	1	0.59	0.0001	MSL	5	−0.22	0.0001
TI	2	0.30	0.0793	ζ	6	0.11	0.0106
H_s	3	0.24	0.0273	U_w	7	0.09	0.0027
T_p	4	−0.23	−.0017	$\sigma_{\_SLDR}$		0.005	
R^2				0.68			

Table 3 Parameter rankings, Sigma-normalized Regression Coefficients (SRC), Regression Coefficients (RC), and model R^2 for linear deterministic D_f model

Fig. 4) than a linear model. A normally distributed error term (ε_{SLSR}) is added with $\sigma_{\_SLSR} = 0.0025$. The exponential relation is defined by

$$D_{f\,SLSR}(X_m, t) = (D_{f0} e^{-|\lambda| X_m} + \varepsilon_{SLSR}) \cdot t \tag{18}$$

$$\varepsilon_{SLSR} \sim N(0, \sigma_{\_SLSR}) \tag{19}$$

Based on the damage models, variance decomposition is performed for both cases as shown in Fig. 5. The results indicate that the uncertainty in $D_{f\,SLSR}$ is governed by the damage model uncertainty (X_m), while $D_{f\,SLDR}$ is most sensitive to soil and metocean parameters ($G_{soil}, TI, H_s, h, T_p$). Note that other parameters (e.g., U_w, wave load coefficients) can also affect the mean fatigue loads, but the relatively lower uncertainties related to those parameters reduce the parameter influence. A visual validation of the damage models ($D_{f\,SLSR}, D_{f\,SLDR}$) against Monte Carlo simulations is shown in Fig. 6, with the x-axis set to the governing parameter. This also illustrates the random variations of $D_{f\,SLSR}$ and $D_{f\,SLDR}$ for a given realization of X_m and G_s, respectively.

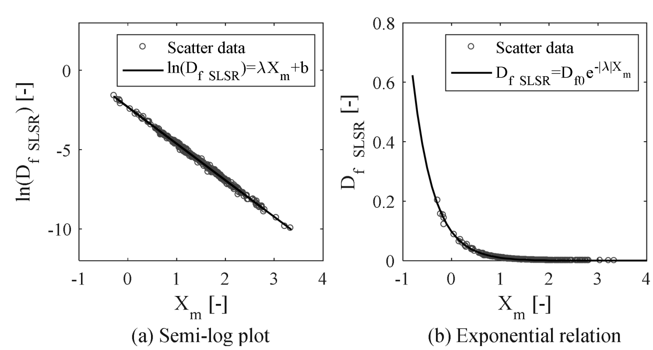

Fig. 4 Exponential relation ($D_{f0} = 0.10$, $\lambda = -2.30$) between model uncertainty (X_m) and fatigue damage (D_f) with $R^2 = 0.99$

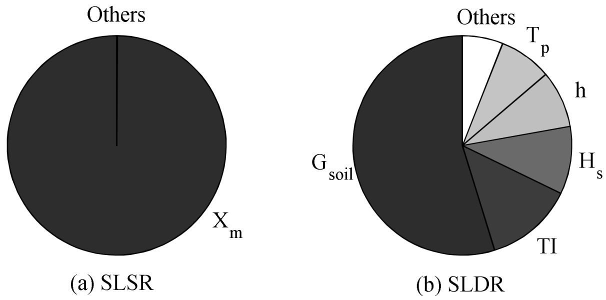

Fig. 5 Variance decomposition of D_f at interface based on linearized damage models

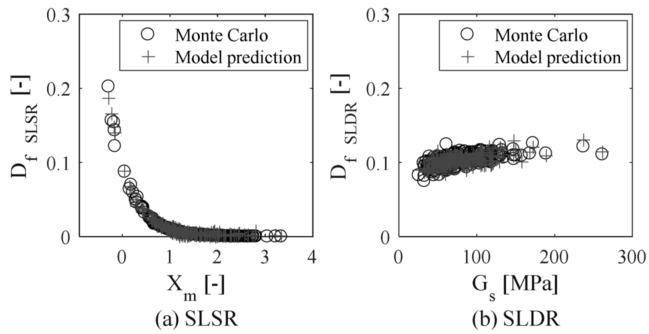

Fig. 6 Comparison of Monte Carlo and damage model predictions

Reliability Assessment

The first order reliability indices are estimated based on the limit state equation (Eq. 13) and the fatigue damage models (Eq. 11 and Eq. 18). The annual reliability indices ($\Delta\beta$) for both SLSR and SLDR cases are shown in Fig. 7. Under different assumptions on Miner's cumulative damage ratio (Δ), both cases have $\Delta\beta$ above the target reliability level ($\Delta\beta = 3.1$ to 3.7). For the SLSR case, different uncertainty levels (COV_Δ) are assumed for Δ (with mean $= 1.00$). For comparison, it is noted that typical COV_Δ for welded steel, cast steel, and composite materials are 0.30, 0.40, and 0.50, respectively (Folsø et al., 2002; Toft et al., 2016b). For the SLDR case, design values for Δ are assumed based on DNV (2012). For structures below or in the splash zone, Δ is reduced to $\Delta = 0.50$, while for structures exposed to harsh environments, such as North Sea conditions, a reduction of $\Delta = 0.33$ is normally adopted.

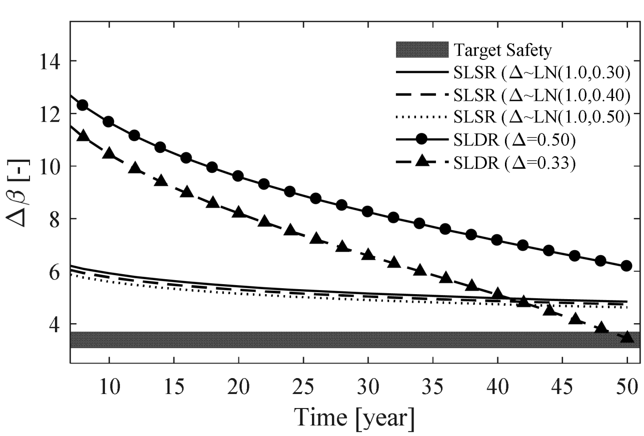

Fig. 7 Annual reliability index ($\Delta\beta$) for concrete fatigue failure showing influence of Miner's rule uncertainty (Δ) assumption

Reliability indices for SLSR and SLDR cases cannot be compared directly, since SLDR estimates are conditional to the assumed material model uncertainty (X_m). In this example, the SLDR case is used to obtain the probability of failure conditional to the design value ($X_m = 0$). The SLSR model predicts a more realistic $\Delta\beta$ with respect to time, with a slight decrease of the reliability with time, mainly due to the time-independent stochastic variable X_m being the dominating uncertainty parameter. This underlines the importance of a more accurate estimate of model uncertainty (X_m) for concrete fatigue reliability assessment. The same is not true for fatigue of welded details, where the loads can have a significant contribution since the load effects are raised to the power of the Wöhler's exponent (m).

CONCLUSIONS

In this study, uncertainty analysis and reliability assessment is performed for a reinforced concrete GBF supporting a 5 MW OWT. By using Monte Carlo procedure, it has been shown that uncertainties in concrete fatigue damage accumulation ($D_{f\,SLSR}$) during OWT power production is governed by the resistance model uncertainty (X_m). For uncertainty analysis conditional to the design value of X_m, soil and metocean parameters ($G_{soil}, TI, H_s, h, T_p$) are found to be more significant.

Reliability analysis also showed that X_m should be treated as a stochastic variable. To improve fatigue assessment of concrete OWT structures, it is recommended to conduct experimental campaigns for concrete fatigue failure at lower stress cycle amplitudes and different mean stress levels. This would reduce the uncertainty related to extrapolating existing test results and would allow evaluation of resistance model uncertainty at load magnitudes experienced by OWTs.

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