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Generic Framework for Reliability Assessment of Offshore Wind Turbine Jacket Support Structures under Stochastic and Time Dependent Variables

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Abstract: Offshore wind turbines (OWTs) are exposed to harsh marine environments with considerable uncertainties in the environmental loads and the soil properties, constituting their integrity assessment a challenging task and qualifying reliability assessment as the most suitable approach in order to systematically account for these uncertainties. In this work, a generic framework for the reliability assessment of OWT jacket support structures is developed based on a non-intrusive formulation. More specifically, a parametric FEA (finite element analysis) model of a typical OWT jacket support structures is developed incorporating load and soil-structure interaction, in order to map its response under varying input conditions. The results from a number of deterministic FEA simulations are post-processed through multivariate regression, deriving performance functions for relevant limit states. For this analysis five limit states are considered, i.e. buckling, deflection, fatigue, frequency and ultimate limit states. The reliability index under each limit state is then calculated using FORM (first order reliability method) to allow calculation of low probability values. The proposed framework has been applied to the NREL 5MW OWT OC4 jacket to assess the reliability of critical components of the structure. The results of the reliability assessment indicate that, for the given stochastic conditions, the structural components of the jacket support structure are found to be within acceptable reliability levels. The proposed framework, which can be applied in various complex engineering systems, has demonstrated to be capable of effectively assessing the reliability of OWT jacket structures and can be further applied to optimize jacket structures on the basis of reliability.

Keywords: Offshore wind turbine; Jacket support structure; Structural reliability assessment; Finite element analysis; Soil-structure interaction; Non-intrusive formulations.

1. Introduction

Wind Europe's central scenario expects 320 GW of wind energy capacity to be installed in the EU in 2030, 66 GW of which coming from offshore wind farms (EWEA, 2015). This interest to move offshore is mainly motivated by the higher wind shear, unrestricted space, and lower social impact in the marine environment (Lozano-Minguez et al., 2011). In fact, it is estimated that an additional 50% of electricity can be generated for the same wind turbine in an offshore environment. Key barriers towards further deployment of offshore wind farms are their high construction cost, especially foundation and electrical connection, and limitations in operation and maintenance which constitute a high percentage of the life cycle costs (Ioannou et al., 2018).

Despite the adverse environmental conditions, offshore wind installations have continuously been on the rise, especially in Europe. In the UK there have been installations in the North Sea, Irish and Baltic seas (Kallehave et al., 2015), with plans for expansion in the next decade. The successful deployment of OWTs largely depends on the accurate estimation of the effects of stochastic loads acting on the asset and the accurate prediction of the components' integrity throughout their service life. An OWT generally comprises of a wind turbine installed on top of a structure which is resting on a foundation that is embedded in the soil transferring loads. Typical stochastic parameters in the design are the soil conditions, which vary with location, wind and wave loads (and their directions) and material properties.

There are various types of support structures that can accommodate offshore wind turbines, while selection of the most appropriate configuration depends on a number of criteria including the water depth, the estimated environmental loads, the cost of production and installation, complexity of the design etc (Kolios et al., 2010, 2016; Mytilinou et al., 2018). The monopile support structure is currently employed in most opeartional projects in Europe due to its simple but robust design (Gentils et al., 2017), it becomes however uneconomical for offshore projects in deeper waters, yielding a requirement for more complex structures such as the jacket configuration which becomes more suitable.

OWT’s are large scale complex structures that are subject to varying environmental and operational loads. Based on recent design practices, they are generally designed for MRP (minimum return period) loads of 50 – 100 years with recommended load and material factors expected to maintain target reliability levels (Wei et al., 2014). The target reliability index for OWT support structures is typically 3.71, corresponding to a probability of failure of 10^{-4} (Det Norske Veritas, 2014). Ensuring that this target reliability levels are maintained throughout the service life of the asset in the presence of uncertainty is a pertinent condition towards optimising their design and performance.

The stability of OWT jacket support structures highly depends on the foundation layout. The surrounding soil and the soil-structure interaction is critical in determining the response of the jacket structure. A number of studies on the soil structure interaction of OWT support structures have been carried out. (Shi et al., 2015) carried out a study on the soil structure interaction of a jacket type OWT by developing a flexible foundation, p-y model of the pile groups and fixed foundation. (Zadeh et al., 2015) investigated the nonlinear response of a fixed offshore platform under the combined wave, wind and current loading for one year and one hundred year return periods. (Madhuri and Muni Reddy, 2019) studied the deflection and natural period due to the soil structure interaction. In all these studies, the soil behaviour was modelled using the p-y method (Det Norske Veritas, 2014), which tends to underestimate the deflection and the modal frequency. OWT support structures are generally expected not only to resist vertical load, but also to ensure that failure does not occur due to large moments. (Jung et al., 2015) compared different foundation modelling methods, investigating the effects of the modelling approach on the structural responses of the OWT.

Structural models to determine the response of OWT support structures can be roughly categorized into two groups i.e. the 1D (one dimensional) beam model and the 3D (three dimensional) FEA (finite element analysis) model. The 1D beam model generally represents the support structure into a sequence of elastic beam elements. This method has been widely used in structural modelling of OWT support structures due to its computational efficiency and acceptable accuracy for modelling global structural behaviour (Bossanyi, 2009). Though efficient, the beam model has the limitation of accurately representing the local structural responses such as local stress concentrations (Petrini et al., 2010). The 3D FEA model, which generally constructs OWT support structures using shell elements, is capable of accurately estimating the structural responses and examining the detailed stress distributions across the support structure (Wang et al., 2016). Therefore, the 3D FEA model is used further in this study for modelling the support structure, ensuring accurate prediction of structural responses subjected to complex loads.

Failure modes related to support structures include a number of time dependent phenomena which are predominant for their design such as the impact of corrosion and fatigue damage due to the marine environment which result to the degradation of the material (i.e. steel) which ultimately affects its resistance (Figueira et al., 2017). Due to the amplitude of fatigue loads in combination with a large number of load cycles as a result of the combined actions of wind, wave and operational loads, fatigue performance of welded connections is a design-driving criterion for OWT support structures (Dong et al., 2012). Corrosion is capable of reducing the material thickness thereby making it susceptible to fatigue crack initiation and buckling, which may result in failure of the structure (Adedipe et al., 2016, 2015). Several approaches such as the S-N curve and the fracture mechanics methods can be employed for fatigue analysis, and multiple NDT (non-destructive testing) methods are suitable to evaluate levels of corrosion with time.

Adopting a reliability approach for the integrity assessment of technical systems or components, the main concern is the evaluation of the probability of failure corresponding to a particular reference period and mode of failure (Faber, 2012). Reliability methods can be roughly categorised into four different levels, based on the way that uncertainty is taken into account in the analysis (International Standardisation Organisation, 2012; Kolios, 2010). In this paper level 3 methods are employed, which account for the first and second order reliability analysis methods. The selection of the method employed for estimation of reliability, also depends on the shape of the limit state and the degree of non-linearity they present. When the limit state is a linear function the FORM (first order reliability method) can be used, while for non-linear limit states SORM (second order reliability method) is more appropriate. FORM and SORM are analytical methods allowing the calculation of low probabilities of failure, however they involve approximations through Taylor's expansions and hence can skew the results in case of complex limit states. Monte Carlo Simulations can also be employed in complex systems and when reasonably high values of probability are expected to be calculated, however this approach can be very computationally intensive for complex systems and low probabilities of failure (Hanak et al., 2016, 2015).

This paper therefore focuses on the development of a generic framework for structural reliability assessment to accurately evaluate the integrity of an OWT jacket support structure under stochastic inputs and for a number of limit states. The methodology developed follows a non-intrusive approach, employing several discrete steps that can allow high fidelity tools to be integrated in the analysis, contrary to a closed form, fully integrated process which would be applicable only for a specific problem. After validation of the framework through a number of case studies, it is then applied to the NREL 5MW OWT OC4 jacket to assess the reliability of the support structure for a set of stochastic input variables.

The paper is structured as follows. Section 2 discusses the fundamentals of structural reliability and the development of the framework, presenting limit states relevant to offshore structures and reliability analysis methods. Section 3 presents the calculation of environmental loads and corresponding load cases, while section 4 documents the development and validation of a parametric model to map the response of the support structure. Section 5 presents and discusses the results of the reliability analysis followed by conclusions in Section 6.

2. Development of a structural reliability assessment framework

2.1. A non-intrusive reliability analysis algorithm

The integrity of an asset in the presence of uncertainty is best evaluated through a structural reliability approach. An effective way to achieve this is through a non-intrusive formulation, following a number of steps as illustrated in Figure 1. A similar approach has been applied by the authors for the evaluation of the performance of a wave energy converter, mainly focusing on global limit states (Athanasios Kolios et al., 2018), as well as calculation of reliability of offshore monopile support structures (Wang and Kolios, 2017). The benefit of this approach is that it is generic enough to accommodate a number of problems and high-fidelity tools can be employed for the individual steps increasing accuracy of the analysis (A. Kolios et al., 2018). The algorithm should be adapted to every different failure mode to be formulated in the form of limit states which distinguish safe and failure operational regions.

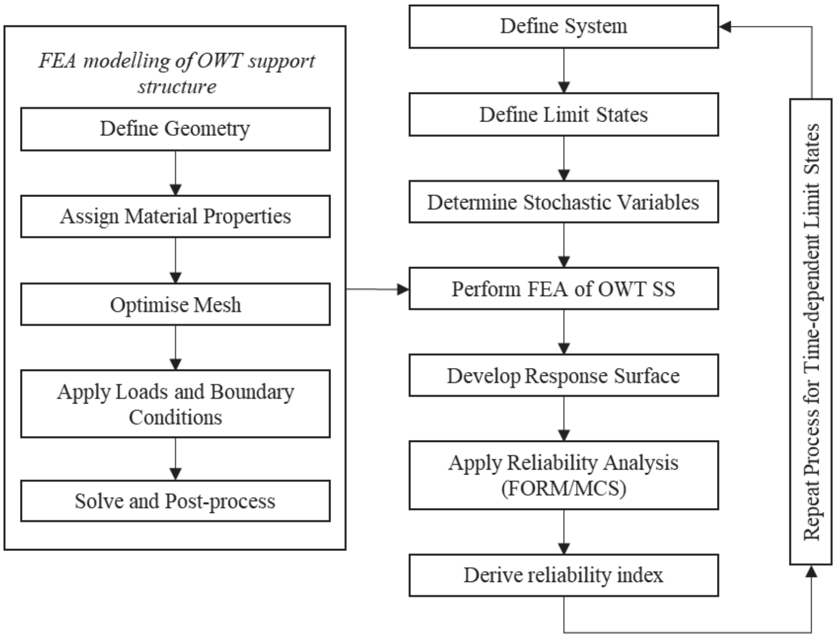

Figure 1: Non-intrusive formulation of a reliability analysis algorithm

For a non-intrusive reliability analysis the following steps are suggested:

i. Define the system: Here the details of the structure are determined, including the layout, material properties, type of analysis etc. This will stand as the basis of the parametric model that will be developed, allowing to effectively evaluate the response of the structure under varying input conditions.
ii. Define applicable limit states: Failure modes relevant to the system of reference need to be modelled in performance functions in the form of resistance (allowable) minus load (available), obtaining positive values for the cases that the structure/component stands in the safe region and negative values when it fails.
iii. Determine stochastic variables: Among the design variables, those with highest degree of uncertainty should be modelled statistically for further consideration in the analysis. Historical data can be employed, and fitting of various distributions should be tested to qualify the most appropriate statistical properties alleviating statistical uncertainty.
iv. Perform a number of simulations: A number of simulations needs to be executed, mapping the response domain for the combination of inputs, building

a design matrix to allow for the derivation of analytical expressions through regression.

v. Develop response surface: Employing appropriate approximation methods (response surface or surrogate modelling) an analytical expression can be developed allowing for solution of the probability integral. The complexity of the limit state will qualify the most appropriate approximation method.

vi. Apply reliability analysis methods: Once the response surface has been developed, analytical (FORM/SORM) or stochastic methods (MCS) can be employed to calculate reliability.

vii. Derive reliability index for each limit state: Depending on the method employed, FORM/SORM derives directly values for reliability index (β) while MCS requires a transformation from the direct calculation of probability of failure.

For time-dependent failure modes, such as fatigue and corrosion, the steps above should be followed in an iterative process quantifying reliability for different time periods.

2.2. Limit states definition

Recent structural design standards follow a limit state design approach, aiming to derive designs with adequate safety margins in order to take account of uncertainties that could adversely affect the safety of the structure. The structure is required to be checked for all types of limit states to ensure adequate safety margins between the maximum likely loads and minimum resistance of the structure for each case (Bai and Jin, 2016).

DNV-OS-J101 design standard (Det Norske Veritas, 2014), which is the most widely applied standard for the design of offshore wind turbines, suggests three basic design limit state considerations in the design of OWT support structures, i.e. (1) ULS (ultimate limit state), which is the load and resistance capacity of the structure (such as buckling and yielding stress); (2) FLS (fatigue limit state), which accounts for failures due to cyclic loading as a result of environmental and operational loads); and (3) SLS (serviceability limit state) which accounts for design tolerance criteria (such as deflection and vibrations). In this study, five design limit states are considered, i.e. buckling, deflection, fatigue, frequency and ultimate loading considering two more limit states relevant to slender structures.

2.2.1. Buckling limit state

OWT jacket support structures are generally regarded as thin-wall structures, and therefore they are susceptible to buckling failure. The performance function of the buckling limit state can be expressed as:

$$g_b(x) = L_m - L_{m,min} \tag{1}$$

where subscript b denotes the buckling limit state, L_m is the buckling load multiplier, which is given by the ratio of critical buckling to the load on the jacket support structure and $L_{m,min}$ is the allowable minimum load multiplier. The above equation implies that if the buckling load multiplier L_m is less that the allowable minimum load multiplier the structure fails.

According to the DNV-OS-J101 (Det Norske Veritas, 2014) the minimum allowable load multiplier is given as 1.4, and this is therefore used in this study.

2.2.2. Deflection limit state

During normal operating and extreme loading of the support structure, it may be deflected considerably in the direction of the load. Although, such deflections are expected, excessive deflections can significantly influence the serviceability of the OWT support structures. Thus, deflection limit state is also a critical factor for consideration in the reliability assessment. The performance function of deflection limit state design can be expressed as:

$$g_d(x) = d_{allow} - d_{max} \tag{2}$$

where subscript d denotes the deflection limit state, d_{allow} and d_{max} are the allowable and maximum deflections respectively. The expression above implies that for a reliable design the maximum deflection of the support structure must not exceed the allowable deflection.

According to DNV-OS-J101 (Det Norske Veritas, 2014) the allowable deflection can be obtained based on the following empirical equation:

$$d_{allow} = \frac{L}{200} \tag{3}$$

where L represents the length of the support structure.

2.2.3. Fatigue limit state

OWT support structures are exposed to significant cyclic loading throughout their service life, making fatigue limit state consideration particularly important to this study. Based on the S-N curve method of fatigue analysis, the number of loading cycles to failure can be expressed as:

$$\log N = A - m \log \Delta S \tag{4}$$

where A represents the intercept, m is the slope of the S-N curve in the log-log plot and ΔS is the stress range. The values of the intercept (A) and the slope (m) are taken as 12.75 and 3 respectively according to DNV-OS-J101 (Det Norske Veritas, 2014).

The performance function of the fatigue limit state can be expressed as:

$$g_f = \ log(N) - \ log\ (N_t) \tag{5}$$

where subscript f denotes the fatigue limit state, N is the number of loading cycles to failure and N_t is the design life cycles, i.e. number of loading cycles expected during the service life. This can be estimated based on the rated speed of the rotor and the availability of the turbine on the selected location. Therefore, considering a design life number of cycles for 20 years, with a rated speed of 12.1 rpm and an availability of (98.5%), the design life cycle N_t can be calculated as:

$$N_t = availability \times rotor\ speed \times [20\ (yrs) \times 365 \left(\frac{day}{yr}\right) \times 24 \left(\frac{day}{hour}\right) \times 60 \left(\frac{min}{hour}\right) \tag{6}$$

The design life cycle N_t derived in the equation above is then used in the S-N curve to derive the fatigue design stress range $\sigma_{f,design}$, while the maximum fatigue (operational) stress range $\sigma_{f,max}$ of the structure is obtained from the FEA simulations result.

2.2.4. Frequency (modal) limit state

OWT support structures are prone to vibrations during their service life that can result to resonance. In conscious prevention of such occurrence the first natural frequency of the jacket support structure needs to be separated from the induced frequency f_{1P} of the rotor and the blade-passing frequency f_{3P}. The safe natural frequency is any frequency range between the rotor f_{1P} and f_{3P} frequencies. GL standards

(Germanischer Lloyd, 2010) suggest that the first natural frequency should be separated from rotor induced frequencies with a tolerance of ±5%. This can be expressed as;

$$f_{1p+5\%} \leq f_{1st} \leq f_{3p-5\%} \quad (7)$$

The rated and the cut-in rotor speeds of the NREL 5MW OWT are 12.1 rpm and 6.9 rpm respectively. Therefore, the limit state design expression for resonance can be expressed as:

$$0.212 \leq f_{1st} \leq 0.328\text{Hz} \quad (8)$$

2.2.5. Ultimate limit state

This limit state accounts for the ability of the support structure to resist plastic deformation. For an OWT jacket support structure, the equivalent stress is generally determined using the von-Mises stress theory. The performance function of the ultimate limit state based on the von-Mises criterion is given by:

$$g_u(x) = \sigma_{allow} - \sigma_{max} \quad (9)$$

where subscript u denotes ultimate limit state, σ_{allow} is the allowable stress, and σ_{max} is the maximum von-Mises stress. The allowable stress σ_{allow} is, given by;

$$\sigma_{allow} = \frac{\sigma_y}{\gamma_m} \quad (10)$$

where σ_y represents the yield strength of the material, with a value of 355MPa for steel S355; γ_m is the safety factor for the material, with value of 1.1 suggested by DNV-OS-J101 (Det Norske Veritas, 2014). Therefore, the allowable stress in this study is 323 MPa.

2.3. Selection of stochastic variables

The performance of an offshore structure depends mainly on the environmental loads, such as wave load L, wind thrust F and tilting moment M; and the structure's resistance which is a function of the material properties, such as young's modulus E and the mass of the RNA (W).

Upon selection of stochastic variables, assignment of appropriate statistical distributions to the selected variables should take place in order to allow for the systematic consideration of uncertainty through reliability analysis. Although the stochastic data are characterized in this application by normal distributions, the framework can accommodate variables of any statistical distribution through appropriate transformations. For illustration purposes, the mean value is taken as the base design value, while the standard deviation is correlated to the mean value through a coefficient of variation, i.e. the standard deviation for the static structural loads is taken as 10% of the mean, and 20% for the time dependent analysis. Table 1 presents the stochastic design parameters.

Table 1: Design Variables and their characteristics

	Static Structural Analysis		**Fatigue Analysis**		
Stochastic variables	*Mean*	*Standard Deviation*	*Mean*	*Standard Deviation*	*Distribution*
$\boldsymbol{F}$ (KN)	781	78.1	197	39.4	Normal
$\boldsymbol{M}$ (KN)	38,567	3.8567	3,687	737.4	Normal
$\boldsymbol{L}$ (KN)	121.2	121.2	121.2	24.2	Normal
$\boldsymbol{E}$ (GPa)	210	21			Normal
$\boldsymbol{W}$ (Kg)	350	35			Normal

The design variables presented in Table 1, are also used to determine the input parameters for the deterministic FEA simulations that will be executed to map the response of the NREL 5MW OWT jacket support structure through the parametric FEA model. Using the ANSYS 'design of experiment function', which converts the input parameters to sets of stochastic variables based on the defined statistical distribution, a series of deterministic FEA simulations is performed. The results are exported to a MATLAB code developed in this study for the next steps of the analysis.

2.4. Stochastic response surface method

For the reliability analysis of an offshore jacket support structure, which is a complex structural system, an analytical expression that is able to express the relationship of the actual loading and the exact response of structural members becomes difficult. Therefore, for cases that involve complex failure mechanisms, the stochastic response surface method can be employed due to its capacity to accurately estimate the response of a component of a system as a function of global design variables, allowing for the calculation of the probability that certain thresholds have been exceeded in the presence of uncertainty. The expressions derived from this method can be input in the performance function which is then used in reliability analysis algorithms such as FORM/SORM and MCS.

Least-Square method (LSM) (Choi et al., 2007) regression method was employed in this study to perform a multivariate regression analysis, where the curve of best fit is obtained by minimizing the absolute distance between the fitted value and the observed values by providing a fitting model. For linear regression, it can generally be expressed as follows:

$$y(x) = a_0 + a_1.x_1 + a_2.x_2 + a_3.x_3 + \cdots + a_n.x_n + \epsilon \tag{11}$$

where y is the independent variable, x is the dependent variables, $[a_0\ a_1\ ...\ a_n]$ are the coefficients of regression, and ϵ is the error term.

The equation above, can be re-written in the following matrix form:

$$Y = XA + E \tag{12}$$

where Y is a matrix of the dependent variables, X is a matrix of the independent variables, A is a matrix with the regression coefficient and E is the error term.

The regression coefficients A can be derived based on the LSM as;

$$A = (X^T\ X)^{-1}\ X^T\ Y \tag{13}$$

To obtain a more accurate approximation, a quadratic multivariate regression is employed in this study. A 3-variable multivariate polynomial regression can be expressed as follows:

$$y(x) = a_0 + a_1.x_1 + a_2.x_2 + a_3.x_3 + a_4.x_1^2 + a_5.x_2^2 + a_6.x_3^2 + \cdots + \epsilon \tag{14}$$

2.5. FORM (First order reliability method)

With the formulation of the performance function, which is a combination of the result obtained from the regression analysis and the limit state expression, the FORM is then used to calculate the reliability index via an iterative process. The overall principle of this method is based on the theory that random variables are usually defined by their first and second moments. The reliability index is estimated based on the approximation of the performance function following the conversion of the random variables in terms of their moments.

The relationship between the probability of failure and the reliability index is given as:

$$P_f = \phi(-\beta) \tag{15}$$

where β represents the reliability index, and ϕ the cumulative distribution function of a normal standard variable.

FORM has been widely used in reliability assessment due to its computational efficiency and ease of implementation. The method has limitations in analysing non-linear limit state functions and this limitation can be overcome by using SORM methods. In this study, the FORM, considering the Hasofer and Lind index is used in the reliability assessment (Hasofer and Lind, 1974). The flowchart of the FORM process for this study is shown in Figure 2.

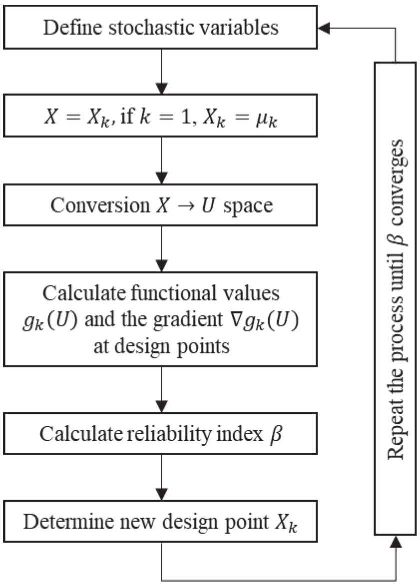

Figure 2: FORM process flowchart

3. Loads for offshore structures

3.1. Environmental loads

OWT jacket support structures are generally exposed to complex and variable loads. Therefore, in conducting any structural analysis adequate consideration of the loads is essential. The loads relevant to OWT jacket support structures can be roughly categorized into three groups, i.e. dead load, live load and environmental loads which are site-specific. The design loads to be taken into account during the design phase of the support structures are generally suggested in relevant design standards such as DNV-OS-J101 (Det Norske Veritas, 2014) and IEC 61400-3 (IEC, 2009). In this study, numerical computation of loads was based on DNV-RP-C205 (Det Norske Veritas (DNV), 2010). The schematic diagram in Figure 3, illustrates the various loads acting on the jacket support structure.

3.1.1. Inertia load

Inertia load is mainly due to the mass of the RNA (rotor-nacelle assembly) and the self-weight of the support structure. This load can significantly influence the buckling and modal frequencies limit states, and therefore, it was considered in this study as a critical contributor to the resultant eigen buckling and modal frequencies analysis of the jacket support structure.

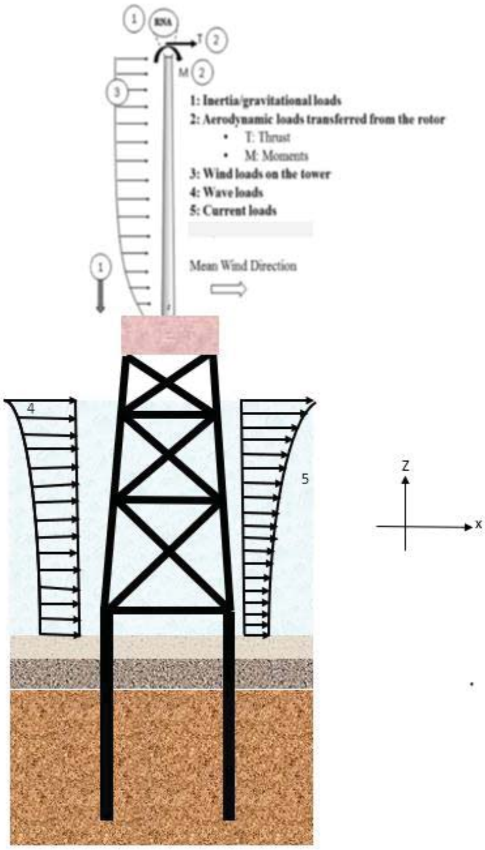

Figure 3: Schematic of loading of offshore jacket structure

3.1.2. Aerodynamic loads

3.1.2.1. Wind loads

Wind loads are among the most important load sources to be considered in the design of a wind turbine support structure. Wind loads on a structure are due to the interaction between parts of the structure above sea level and the wind in a given field, which causes a drag from the air particles motion. The magnitude of the drag is dependent on the met-ocean data such as the mean wind velocity $\bar{V}(z)$. The wind speed for any elevation above the mean sea level is given by;

$$\bar{V}(z) = \bar{V}_r \left(\frac{z}{z_r}\right)^{\propto} \tag{16}$$

where $\bar{V}_r$ is the wind speed at the reference, i.e. at the height of the top of the jacket, since the hub was not considered in this study. z_r and α are reference height and roughness coefficients respectively. The wind force acting on a structure is the summation of wind force acting on individual members. Therefore, formulation of drag force on an object within a flow can be applied to obtain the wind force on the members, and is given by:

$$F_{tower}(z) = \frac{1}{2}\rho_a C_{D,T} D(z)\bar{V}_r^2(z) \quad (17)$$

where ρ_a is the air density, $C_{D,T}$ is the drag coefficient of the tower and $D(z)$ is the outer diameter of the tower at height z.

3.1.2.2. Aerodynamic loads transferred from the rotor

Although this study does not consider the structures above the transition piece, the effect of the loads on the rotor that is transferred to the top of the support structure cannot be neglected. Realistically, aerodynamic loads acting on the rotor are usually transferred to the top of the jacket and are generally decomposed through the load matrix defined in the turbine's axis. Typical design load values for both fatigue and extreme loads used for this study were extracted from the WindPACT (Wind Partnership for Advanced Technologies) report on Turbine Rotor Design Study (Lanier and Way, 2005).

3.1.3. Hydrodynamic loads

3.1.3.1. Wave Load

Proper estimation of the wave load is critical, to achieve a realistic model, as waves can induce a significant force on an offshore structure. The choice of wave theory to be applied to a model is dependent on the site characteristics such as the water depth, wave height and wave period. The decision of the choice of wave theory is dependent on the ratio of the height to diameter of the structural member. When the diameter of the structure, D, is less than one fifth of the wave length, λ, Morrison's equation can be applied for the wave estimation (Det Norske Veritas, 2014).

$$D \leq 0.2\lambda \quad (18)$$

Morrison's equation propounds that the wave load is a combination of the drag and inertia forces, which can be expressed as:

$$F_{wave}(z) = \frac{1}{4}\rho_w \pi D^2 C_M u(z,t) + \frac{1}{2}\rho_w D C_D u(z,t) \,.\, |u(z,t)| \quad (19)$$

where ρ_w is the density of water with a typical value of 1000 kg/m^3, C_M and C_D are the coefficient of inertia and drag of the piles respectively, and their corresponding values are 1.6 and 1.0 respectively, according to (DNV GL AS, 2016).

3.1.3.2. Current load

Current accounts for the movement of water, and such movement around a support structure can induce a drag acting on it. The current velocity at MSL (mean sea level) can be estimated using an exponential profile, given as;

$$u_c(z) = u_{c,\ MSL}\left(\frac{d+z}{d}\right)^{\frac{1}{7}} \tag{20}$$

where $u_{c,\ MSL}$ is the current velocity at MSL, d is the depth of water and z the reference depth. For simplicity, the wave and current are generally assumed to align to each other, and therefore the current velocity can be added to the wave particle velocity in the drag term of the Morrison's equation.

3.1.3.3. Hydrostatic load

The submerged sections of the jacket support structure are bound to experience hydrostatic pressure, which is a permanent normal load and varies with the depth of the water. The hydrostatic force is given by;

$$F_h = \rho_w g h \tag{21}$$

where F_h is the hydrostatic force, ρ_w is the density of water, g is the gravitational constant and h is the depth of the water.

3.2. Load cases

IEC 61400-3 (IEC, 2009) defines 32 DLCs (design load cases), covering various operational modes of the turbine such as start-up, normal operation, shut down and 50-years extreme conditions. These DLCs can be roughly categorized into two major groups namely ultimate and fatigue DLCs. Basically, the typical load cases applied in structural design of OWT is the fatigue load under normal sea conditions and the ultimate load under 50-year extreme condition. In this study, both ultimate and fatigue DLCs are considered.

3.2.1. Fatigue load case

The manner of environmental loading and the rotor operations during the service life of an OWT induces significant source of cyclic loading, making the support structure of the turbine susceptible to fatigue failure (Muskulus M, 2014). A commonly used fatigue DLC is the NTM (normal turbulence model) or NSS (normal sea state), where the site is assumed to have no current and the wave height and the cross zero period are obtained via a probability density function of the site. DLCs 1.2 and 1.3, as prescribed in (Det Norske Veritas, 2014) and (IEC, 2009), are generally regarded as the governing fatigue DLCs for OWT support structures, and therefore they are considered in this study as fatigue load cases.

3.2.2. Ultimate load case

For the extreme environmental conditions experienced by the OWT the 50-year return period is generally considered as a critical ultimate load case. It has been demonstrated in previous studies that the NREL 5MW OWT is predominantly governed by the impact of the aerodynamics (wind) load rather than the hydrodynamic (wave and current) loading (Baniotopoulos et al., 2011). Therefore, the critical load case for ULS is mostly considered to correspond to the parked turbine, under the 50-year EWM (extreme wind model) with a 50-years RWH (reduced wave height) and ECM (extreme current model). The loading characteristics as described above correspond to the IEC61400-3 (IEC, 2009) DLC 6.1b and 2.1 GL regulation (GL, 1995) respectively. Load safety factor for gravitational load and other loads (such as wind, wave, and current loads) are given as 1.1 and 1.35 respectively (IEC, 2005). The design load cases used for this study are summarized in Tables 2(a) and 2(b).

Table 2 (a): Summary of aerodynamic loads (Lanier and Way, 2005)

Load case	Thrust force (KN)	Tilting moment (KN-m)
Fatigue load case	781	38,567
Ultimate load case	197	3,687

Table 2 (b): Summary of design load cases (DLC's)

Load cases	Wind condition	Wave conditions	Load safety factor
Fatigue load case (operating) DLC 1.2	NTM: V_{ave}	NSS: H_{ave}, T_{ave} No current	1.0
Ultimate load case (parked) DLC 6.1b/2.1	EWM: V_{g50}	RWH: 1.32 x H_{s50} , T_{s50} ECM: $V_{c,ex}$	Normal N 1.1/1.35

4. Parametric Finite Element Analysis model

4.1. Parameters definition

This section presents the development of a parametric FEA model based on the 5MW NREL (Jonkman J, Butterfield S, Musial W, 2009) wind turbine OC4 jacket structure, taking into account environmental and operational loading and soil-structure interaction. The first step in parametric FEA modelling is to define all geometrical parameters of the model such as diameters, structural member thicknesses and other geometric data.

4.2. Geometry generation

The jacket support structure model used in this study is the OC4 jacket support structure (Vorpahl et al., 2013) which is designed for a reference site with water depth of 50 m (Fischer T, de Vries W, 2010). The four-legged jacket support structure has four levels of X-bracing, a corresponding mud brace, and four central piles with a penetration of 45 m (Vorpahl et al., 2013). The structure is made up of an interconnecting circular tube frame and are joined together via 64 welded connections. The height of the jacket from the top of the TP (transition piece) to the mudline is 70.15 m and the hub height with respect to the MSL (mean sea level) is 90.55 m. The piles are grouted to the jacket legs, while the transition piece between the tower and the jacket is a rigid block which is penetrated by the top part of the jacket legs.

Table 3: Properties of Jacket members (Vorpahl et al., 2013)

Property set	Component description	Outer Diameter (m)	Thickness (mm)
1	X- and mud braces	0.8	20
2	Leg at lowest level	1.2	50
3	Leg level 2 to 4	1.2	35
4	Leg crossing the TP	1.2	40
5	Piles	2.082	60

The geometry model considered in the FEA modelling comprises the hub, transition piece, tower (jacket), tower leg, grout, piles and soil. The tower is further discretized into three segments in order to assign varying thickness profiles. The jacket structure was generated from bottom to top, using design point, while the braces are generated by lines. The soil and the transition piece where created as 3-D solids. The tower leg, grout and piles where modelled as shell elements, connected through the contact function.

4.3. Definition of material properties

The primary parts of the jacket support structure are mainly made up of steel materials. The steel material used for the design of the 5MW NREL OWT on OC4 jacket is S355NL steel plate, which has been widely used for OWT structures due to its high weldability (DNV GL AS, 2016; Igwemezie et al., 2018). This class of steel follows isotropic elastic behaviour, and its physical properties are given in Table 4.

Table 4: Properties of the S355NL structural steel (Vorpahl et al., 2013)

Density	Young's Modulus ES	Poison Ratio VS	Yield strength
8500 Kgm-3	2.1E11 Pa	0.3	355 MPa

The transition piece is a steel-concrete configuration. Grout (of different type and composition) is also used for pilling. The steel material data was obtained from the reference OC4 project (Vorpahl et al., 2013). The grout material data were obtained from the Ducorit data sheet (Ducorit, 2013), representative for most OWTs. The properties of both the transition piece and grout are summarized in Table 5.

Table 5: Properties for grout and transition piece (Ducorit, 2013; Vorpahl et al., 2013)

	Young's Modulus	Poisson's ratio	Density	Compressive Strength	Tensile strength
Transition piece	70 GPa	0.18	2300 Kg/m3	200 MPa	10 MPa
Grout	70 GPa	0.19	2740 Kg/m3	200 MPa	10 MPa

The soil model consists of three layers of sand i.e. loose, medium and dense sand, of which material properties are listed in Table 6. The soil material can be well described by the Drucker-Prager model (Drucker DC, 1952), which is mainly dependent on pressure and has been widely used in soil modelling. According to the Drucker-Prager model, the yield strength of the soil, $\sigma_{y,s}$, can be expressed as functions of the internal friction angle φ and the cohesive value of c using the equation below:

$$\sigma_{y,s} = \frac{6c\cos(\varphi)}{\sqrt{3}\left(3 - sin(\varphi)\right)} \tag{22}$$

The frictional coefficient between the pile and the soil, C_f, can be expressed as (Jung et al., 2015):

$$C_f = tan\left(\frac{2}{3}\varphi\right) \tag{23}$$

The properties of the soil used for this study were adapted from (Jung et al., 2015; R. Obrzud, 2010). The depth of third layer of the soil which is the dense region was assumed to be 36 m in order to achieve realistic soil conditions.

Table 6: Properties of the layers of sandy soil (Jung et al., 2015; R. Obrzud, 2010)

Type of sand	Unit weight (KN/m3)	Young modulus (MPa)	Angle of friction (deg.)	Cohesion (KPa)	Yield stress (KPa)	Friction coeff (-)
Loose	10	30	33	50	59.2	0.40
Medium	10	50	35	50	58.5	0.43
Dense	10	80	38.5	50	57.0	0.48

4.4. Element type definition and mesh generation

The tower is a thin-wall structure and thus can be effectively and efficiently modelled using shell elements. The shell element used for this model is the shell281, which is characterized by eight nodes and six degrees of freedom at each node. More details of the shell element can be found in ANSYS documentation. This type of element configuration is most suitable for linear, large rotation and/or large strain non-linear applications. The soil was modelled using a linear order solid element (SOLID 185) while the grout was modelled with 2nd order solid elements (SOLID 186) which enables the development and propagation of bending stresses. The grout, transition piece and soil were modelled using solid elements.

Mesh convergence studies were conducted to establish the most appropriate mesh sizes. To well control the mesh seven meshing parameters are defined for the seven parts i.e. (1) Soil body, (2) grout body, (3) pile body, (4) jacket leg, (5) jacket body, (6) transition piece and (7) tower. The element sizes were refined, and the percentage difference in the maximum von-Mises stress as well as the total number of nodes were recorded. In this case a horizontal force applied to the top of the jacket structure, and the result for mesh refinement carried out are presented in Table 7.

As can be seen, the maximum von-Mises stress obtained is converged in refinement 2, as it has a relative low percentage difference of (0.3%) when compared to further refinement. Therefore, the meshing parameters used in Refinement 2 are deemed appropriate for this study. The FEA model of the structure based on the created mesh is depicted in Fig.4.

Table 7: Mesh convergence study results

ID	Element Sizes of the respective bodies (M)							Total number of element	Maximum VMises stress (MPa)	% Diff
	Soil	Grout	Pile	Jacket Legs	Jacket Body	Transition piece	Tower			
Meshing 1	3.5	1.4	1.4	0.5	0.7	0.7	0.7	38023	3.4537	36
Refinement 1	5	2	2	0.5	1	1	1	5282	1.8836	0.35
Refinement 2	4.5	1.8	1.8	0.5	0.9	0.9	0.9	6501	1.8894	0.31
Refinement 3	4	1.6	1.6	0.5	0.8	0.8	0.8	7735	1.8961	

Figure 4: Isometric View of 3-D Model

4.5. Apply boundary conditions

The main loads acting on an OWT's support structure are mainly the aerodynamic and hydrodynamic loads. The aerodynamic loads are applied on the top of the jacket as force and moment. The aerodynamic loads used in this study were obtained from the fatigue and extreme load data for 5MW NREL WindPACT design load cases. The wave load is applied as pressure, which enables the automatic update of the loads in case of modification of the diameter of the steel members.

A fixed body-body contact was created between the jacket legs beneath the mudline brace and the jacket legs embedded in the pile and grout configuration. The pile and the soil contact were based on augmented Lagrangian formulations, enabling an appropriate soil-solid interaction. Contact between other faces were based on bonded formulations.

4.6. Solve and post process results

Sequel to the appropriate definition of all design parameters, geometry, materials, mesh, element properties and boundary conditions, several analyses can be conducted such as static, modal and time-dependent analysis. The results obtained, such as deformation and stress distributions for the jacket and soil structures, are then plotted using the ANSYS post-processing functions and are introduced in a purpose developed Matlab routine for the reliability analysis through approximation and FORM.

5. Results and discussion

In the previous sections, a framework for the reliability assessment of complex support structures has been developed. Results from the implementation of the framework from application to the NREL 5MW OWT jacket support structure are presented here for the multiple limit states introduced earlier and accounting for a number of stochastic input variables. Case studies are performed to validate the main components of this framework, i.e. the FEA model and the FORM.

In this study the results presented account for the most critical component of the structural system, in a detailed analysis however the reliability indices for each component should be evaluated separately. Then, following one of the system analysis techniques, such as the push-over analysis, failure paths should be identified and the reliability of the system could be calculated with systems in series and parallel calculations.

5.1. Validation

5.1.1. FEA model validation

Two base cases which have been reported in literature (Jonkman J, Butterfield S, Musial W, 2009), i.e. the deflection analysis and the modal frequency of the NREL 5 MW OWT OC4 jacket support structures, are considered as benchmarking studies, comparing previously published results to values obtained from this analysis in order to validate the parametric model developed.

5.1.1.1. Deflection static analysis

This scenario assesses the total deflection of the OWT support structure in static analysis. In the reference literature four case studies were performed for the deflection analysis, which consist mainly of an application of a thrust of 2MN and 4MN, with and without the weight of the RNA. The displacements of the RNA and tower base are measured with respect to the location of the RNA and tower base centre under unloaded conditions respectively.

As can be seen in Table 8, good agreement is achieved, with a maximum relative difference (+6.24%) observed for the displacement at the tower base. This result confirms the validity of the present FEA model.

Table 8: Deformation result of static analysis of 5MW NREL on OC4 jacket structure.

Load case	**Displacement at RNA**			**Displacement at tower base**		
Thrust /mass	**Ref. (Jonkman J, Butterfield S, Musial W, 2009)**	**Present**	**%Diff.**	**Ref. (Jonkman J, Butterfield S, Musial W, 2009)**	**Present**	**%Diff.**
2MN / RNA	1.2089	1.2073	-0.13	0.1375	0.14368	+4.49
4MN / RNA	2.4178	2.3013	-4.8	0.2749	0.29206	+6.24
2MN / 0	1.2089	1.2073	-0.13	0.1375	0.14368	+4.49
4MN / 0	2.4178	2.3013	-4.8	0.2749	0.29206	+6.24

5.1.1.2. Modal analysis

This case study also assessed the natural frequencies of NREL 5MW OWT OC4 jacket support structure. The modal frequencies calculated from the present FEA model are compared against those reported in (Jonkman J, Butterfield S, Musial W, 2009), and the comparison results are presented in Table 9. Reasonable agreement is achieved, with maximum relative difference (12.38%) observed for the 2nd fore-aft mode. This further confirms the validity of the developed parametric FEA model.

Table 9: Modal analysis results, comparing the mode frequencies of structure and the reference values

Mode Frequencies (Hz)	**Ref. (Jonkman et al., 2013)**	**Present**	**%Diff.**
1st Fore-aft	0.31896	0.32973	+3.37
1st Side-to-side	0.31896	0.32973	+3.37
2nd Fore-aft	1.1936	1.0446	+12.38
2nd Side-to-side	1.1936	1.0478	+12.12

5.1.2. FORM Validation

To validate the FORM, a simple hypothetical truss structure as shown in Fig.5 was used for the analysis, and a comparison of the reliability index obtained by Direct

Simulation (DS), through Monte Carlo Simulation, and results obtained through combination of the response surface methods (RSM) and FORM was reported (Kolios et al., 2018). The simple 3-D reference jacket model consist of 40 interconnected beam members in three levels of symmetric geometry in series of 12 lateral and 4 vertical loads acting at the top of the structure. Four basic stochastic parameters are taking into account, these variables are presented in table 10 and Fig. 5.

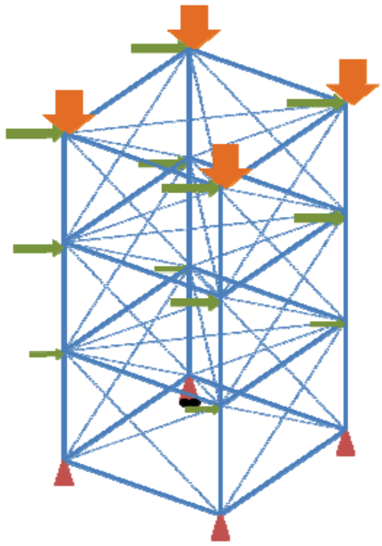

Figure 5: Reference Hypothetical Structure

Table 10: Stochastic loads consideration

Parameter	Characterization
Loads	$F * N\ (1, 0.2)$
Elasticity	$N\ (21 * 10^{10}, 1 * 10^{10})$
Area	$A * N\ (1, 0.01)$
Allowable stresses	$N\ (100{,}000, 10{,}000)$

The results obtained from the Direct simulation as well as the FORM procedure, executed only on members of the reference structure that their probability of failure is other than zero, is presented in Fig. 6. From the reported results the RSM presents a higher reliability index compared to the DS method. However, the difference in the estimated reliability index values from both methods is consistently under sufficiently low. Therefore, an agreement between the methods can be concluded due to the marginal differences in their reliability estimation and considering that RSM takes only a few seconds compared to MCS that requires significantly longer time to complete analysis.

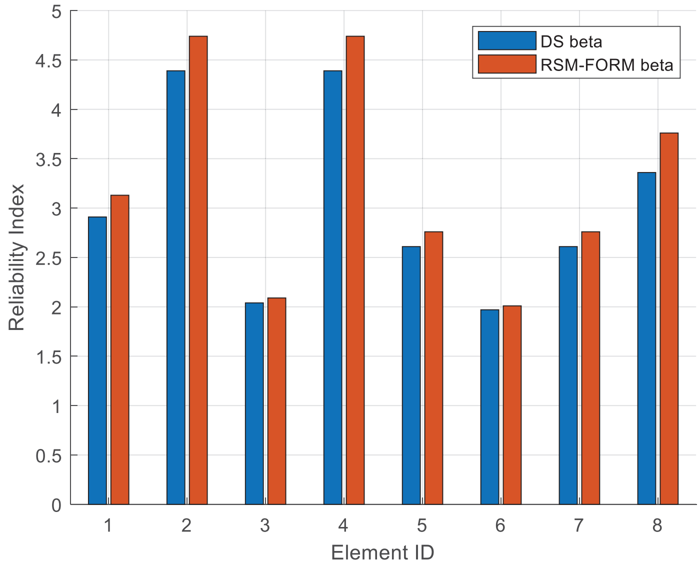

Figure 6: Stochastic Loads Consideration for Validation of the FORM

5.2. Soil-Structure-Interaction Analysis

This study also analysed the effect of the soil on the structure during loading. Jacket structures are normally connected to the pile legs by grouting with concrete. In this study, all the results present so far, where based on the FEA simulation results obtained from the model that includes the soil. However, the soil was supressed, and the analysis repeated. The results obtained from the study are present in Fig. 7.

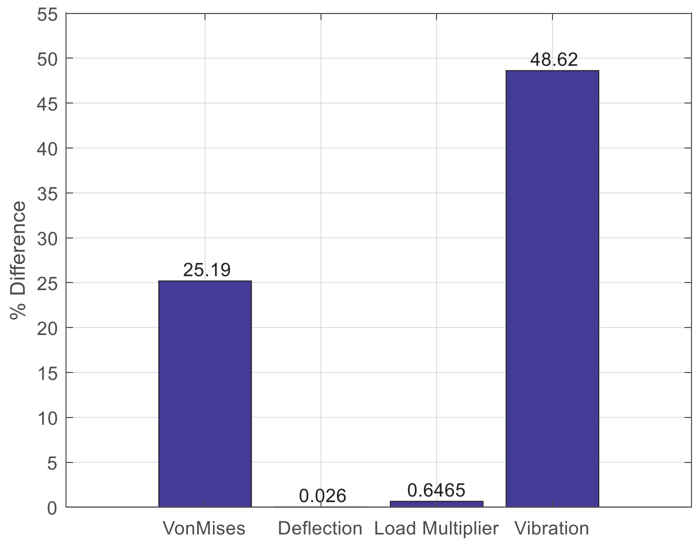

Figure 7: Soil-structure Interaction Sensitivity analysis

The results in Fig. 7, clearly show that incorporating the soil model to the jacket model, affects the response of the model. The results of the modal frequency tend to be mostly impacted by the addition of the soil model to the jacket model. The response for the buckling and deflection analysis shows a lower effect to the inclusion of the soil model.

The impact of the soil interaction with the model does not significantly affect the load multiplier and the total deflection, although the FEA results for deflection at the mudline sections of the jacket model show a higher deflection reading, when the soil is incorporated.

5.3. Application of reliability-based framework

5.3.1. Ultimate load case

The reliability assessment results obtained from the ultimate load cases, which mainly depend on the buckling, deflection and ultimate stress analysis of the OWT jacket support structure, are presented in Fig. 8. It should be noted that the framework calculates reliability indices for each component of the structure, hence different values are obtained for each component. For illustration purposes only the values of the minimum reliability components are reported in the subsequent sections of this paper. As evident the multi-attribute reliability assessment exercise performed on the structure shows that the present model, as designed and for the modelling of the stochastic variables considered, satisfies the recommended reliability assessment criteria, as the reliability index for various limit state are within design thresholds, as they all clearly exceed the target reliability set at 3.71.

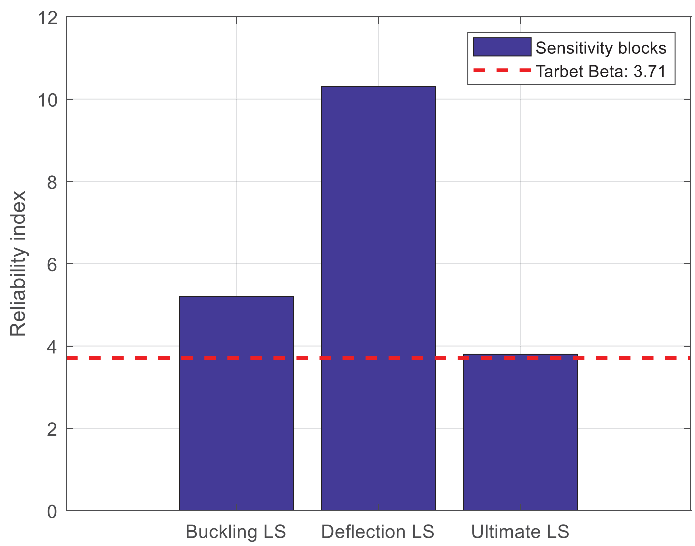

Figure 8: Reliability Index of factored multi-criteria limit state

In addition to the ultimate load cases analysis, the vibration analysis was also performed based on the parametric FEA model, to establish the safety of the structure with regards to resonance. As stated earlier the safe natural frequency range for the structure is in the region of the rotor f_{1P} and f_{3P}. With the reported 1st modal frequency given as 0.2394Hz, the model is found to be safe since the reported natural frequency falls within the safe region.

5.3.2. Fatigue reliability assessment

Sequel to the completion of the FEA model validation process, the model developed is applied to assess the fatigue reliability of the NREL 5MW OWT OC4 jacket support structure. The fatigue reliability assessment study performed is based on the well-known fatigue limit state method as described earlier, which follows the S-N approach to determine the fatigue life of the structure. According to DNV RP C203 (2005), the parameters of standard S-N curve such as the intercept (A) and slope (m) for studying the fatigue life of a steel structure in water for N $\leq 10^7$ is given as 12.75 and 3, respectively. The result obtained from the fatigue life analysis is presented in Fig.9, showing a comparison of the reliability index deterioration with the target reliability index.

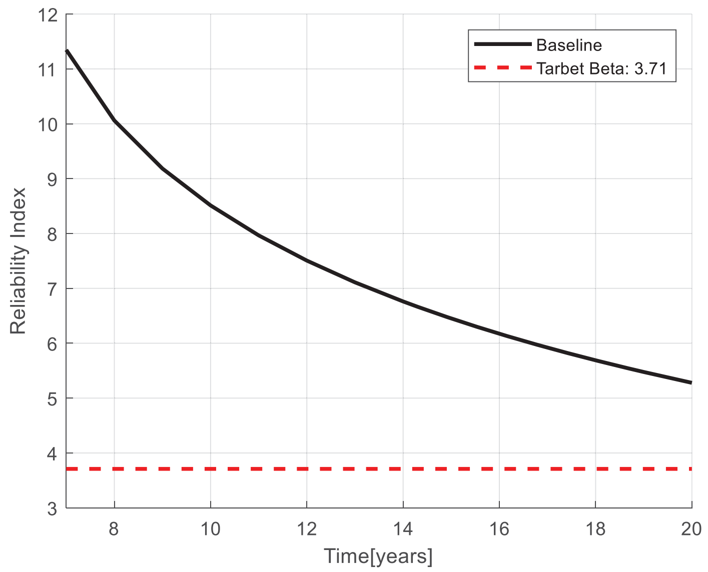

Figure 9: Fatigue reliability assessment

From the fatigue reliability index curve, it can be observed that the structure maintains a reliability index exceeding the defined threshold of target reliability as specified by the standards for the nominal 20 years of operation. In an opposite case, there would be a requirement for design intervention or change in the operational loading envelop that the asset is expected to experience throughout its service life.

Further, a sensitivity analysis has taken place and is presented here, varying gradually the mean values and standard deviation of each stochastic parameter, one at a time by 20%, and evaluating the reliability variation for comparison with the defined threshold. The result of the sensitivity analysis is reported in Fig.10. The results obtained from the sensitivity analysis, clearly show that the mean values of the tilting moment (M) and the wind thrust (F) are the most influential parameters in the fatigue reliability analysis, with the tilting moment qualifying as the most sensitive parameter.

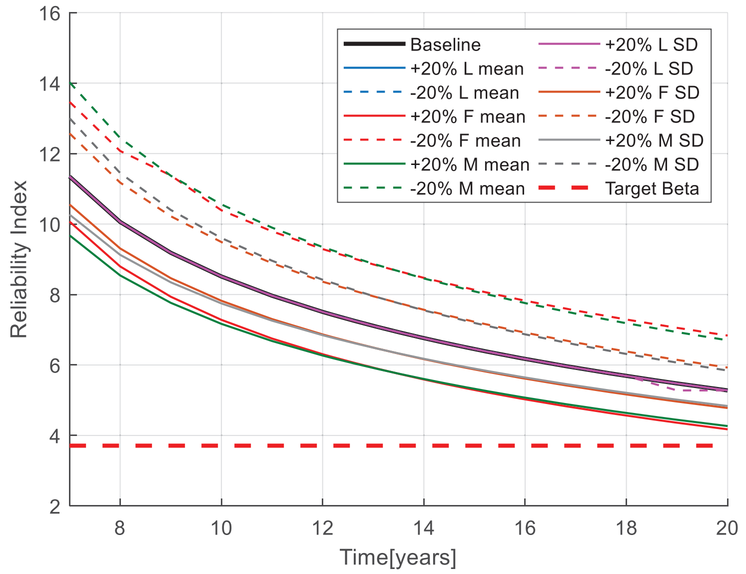

Figure 10: Sensitivity analysis of statistical parameters

6. Conclusions

In this study, a reliability assessment framework for OWT (offshore wind turbine) jacket support structures has been developed. The framework starts with defining a set of limit states which include the buckling, fatigue, ultimate stress, vibration and deflection design criteria. A parametric FEA model for a typical OWT jacket support structure is developed, taking account of stochastic variables and SSI (soil structure interaction). After a number of FEA simulation are performed, results are post-processed through response surface models, deriving performance functions expressed in terms of global stochastic parameters. FORM is then employed to calculate the reliability index, evaluating the reliability of the structure. The proposed framework has been applied to the NREL 5MW OWT OC4 jacket support structure to assess its reliability.

The following conclusions can be drawn from the present study:

- Good agreement is achieved when comparing the results from the present FEA model against those reported in literature, which confirms the validity of the present FEA model.
- The model considered in this present work, when subjected to a multi-variate reliability assessment and for a set of stochastic variables, shows that the results obtained fall within the recommended design limits for OWT support structure design for all limit states examined.
- The results from the sensitivity analysis performed to analyse the effect of the soil in the model clearly show that the addition of the soil in the model impacts considerably the response results obtained. The vibration consideration presents the highest impact in this instance.
- A sensitivity analysis performed on the statistical parameters of the stochastic variables considered, and more specifically for the fatigue reliability assessment, indicates that the most sensitive parameters were the mean value of the tilting moment (M) and wind thrust (F).
- The fatigue reliability analysis performed shows that the reliability index of the model was 5.2 at year 20 of this analysis, which is above 3.71 and thus the structure can be said to be safe, and potentially useable even after its original design life, although this would require a thorough structural assessment of the structure.

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