**SUPERGEN Wind Hub Deliverable**

# WP6.1: Risk-based structural design methodology

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# SUPERGEN Wind Hub Deliverable

# WP6.1: Risk-based structural design methodology

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**Abstract:**

This report documents findings of work package 6.1: Risk-based structural design methodology of the Supergen wind hub. This work explores the potential of wind energy structural systems to employ risk-based methods towards more efficient and optimised designs, through transition from deterministic to probabilistic design philosophies that can reduce life-cycle costs through a systematic consideration of design uncertainties, avoiding unnecessary conservatism that leads to underutilisation of the capacity of structures. This document aims to present related concepts, gradually developing a framework for structural reliability assessment of complex structural systems, such as offshore wind turbine foundations. This is based on a non-intrusive sequence of steps which combine stochastic modelling of environmental loads and structural capacity variables, followed by high fidelity structural analysis simulations against appropriate limit states (ie fatigue and ultimate) which can map the response within the design space and then calculate reliability index of components of the structural system through first order reliability methods.

The report starts with an introduction on the fundamentals of risk and reliability and a review of existing design methods, standards and target safety levels. Following specific aspects of reliability analysis of offshore structures is presented, with extra focus to fatigue reliability assessment which is the prevailing damage mechanism in offshore wind turbine support structures. The concept of reliability-based calibration of safety factors is also presented as the primer of a fully risk-based design, followed by a discussion on non-intrusive methods for structural reliability and more particularly approximation methods that can link external loads to their local effects acting upon each member of the structural system. Finally a series of three case studies will illustrate applicability of the concepts presented, together with a technical assessment of the impact of risk-based concepts to the resultant design of support structures.

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# 1. Introduction

## 1.1. Scope of the report

This research aims to discuss concepts of risk-based design, specifically for the context of offshore wind support structures. Similar concepts have been applied in the past in structural systems of different classes, such as those relevant to the offshore oil & gas or marine application. Reference to structural systems is appropriate for the reference application, as it can be considered that each of these systems comprises of a number of structural components, i.e. cans or tubulars and connections, which can be modelled as a system, also integrating concepts such as active redundancy in their design.

The distinctive difference between offshore wind energy structural systems and other system is on the marginal profit that they operate, the fact that a great number of structures need to be fabricated contrary to the uniqueness of oil & gas structures and the fact that technological development of the sector is slower that its commercial development. To this end, it becomes a pertinent to seek ways to optimise design and operation of assets focusing both on the CAPEX as well as OPEX part through adopting advanced concepts such as risk-based design, integrity monitoring, condition-based maintenance etc.

The nature of offshore and marine structures accounts for a number of uncertainties that should be accounted in the design systematically. Typical examples are the environmental loads, i.e. wind and wave loads, and the variables associated with structural capacity, i.e. material properties and geotechnical conditions, which are all better represented through statistical means rather than deterministic values. From the limit state design approach where a probabilistic basis can be illustrated, this work discusses the development of a fully probabilistic, risk-based approach for the efficient design of offshore support structures.

The three key elements of the report are the formulation of fatigue reliability analysis limit states, presentation of the reliability-based calibration of safety factors approach and the development of a non-intrusive structural reliability analysis framework applicable to offshore wind turbine support structures. Applicability of these concepts will be illustrated through a series of three case studies will illustrate applicability of the concepts presented, together with a technical assessment of the impact of risk-based concepts to the resultant design of support structures.

It is expected that this report and presented methods, will be useful to researchers and practitioners working on the optimisation of the design of offshore wind support structures.

## 1.2. Basics of risk and reliability

Current evolution in engineering practices has allowed more critical and complicated structures to be designed with greater confidence than in the past. In addition, increase in number and complexity of assets indicates that design optimization is required to derive competitive structures, compromising performance characteristics for more cost effective designs. Structural Reliability has been established as a key performance indicator that ensures derivation of preferable structures which comply with minimum safety requirements. The requirements set above, indicate the demand of a more systematic assessment of the uncertainties of the basic design and resistance variables; the functional and environmental loads, geometrical and model parameters, as well as material properties.

Cases with limited levels of randomness can be treated deterministically, applying a magnification factor on the loading or a reduction on the capacity modelling, to account cumulatively for the effect of uncertainties. This rather simplistic approach in the design process produces most of the times oversized designs without providing accurate information on the service life performance of the structure, or ensuring adequate levels of structural safety. In contrast, when the level of uncertainty is high, a stochastic approach of the design variables seems essential. Following this approach, statistical representation of the design parameters will provide the response of the structural member or system in a stochastic way, allowing a better understanding of its service life performance.

ISO 31000 [1], defines risk as the effect of uncertainty in objectives, while in ISO 2394 [2], the most complete definition of reliability, which has been adopted with minor alterations from most current design standards, summarizes that “reliability is the ability of a structure to comply with given requirements under specific conditions during the intended life for which it was designed”. In this definition, the important elements of design requirements, service life period, and design conditions are included. A more practical, direct approach defines reliability of a structure as the opposite of its probability to fail. Reliability is the indicator that will evaluate performance compromising the two requirements of design; structural integrity and economy also complying with the ALARP principle. In [3], a fundamental requirement for design states that “a structure should be designed and executed in such a way that it will, during its intended life time with appropriate degrees of reliability and in an economic way sustain all actions and influences likely to occur during execution and use and remain fit for the use for which it is required”. Structures can be designed to have nearly zero probability to fail. Absolute no failure can never be achieved because absolutely every forthcoming event cannot be realistically predicted or mitigating against all potential unlikely events. Therefore, failures are accepted up to a level that all parties involved in the design and operation of the structure will agree.

Structural reliability works on the prediction of the probability of exceedance of the structural restrictions imposed by the design requirements at any stage of its service life. The probability of occurrence of such an event is directly correlated to its reliability, and once this is derived, design alterations can be identified, in order to either improve structural reliability, or optimize already adequate designs. A classification between probability concepts, distinguishes frequentistic to Bayesian Probability. The first refers to the statistical interpretation of the outcomes of stochastic experiments and its approach to probability can be adequately predicted when the number of experimental iterations is increased respectively. Techniques of structural reliability, following computational resources and numerical methods evolution, can be applied in wider multidisciplinary design environments, considering the joint effect of multiple uncertain variables. Practice can verify that structures that have been designed deterministically, neglecting analytical modelling of uncertainty in variables, can have a greater probability of failure compared to less expensive structures of similar service that have been designed following a stochastic approach.

As mentioned above, risk and uncertainties are linked, hence understanding the latter is important towards determining the number and type of variables to be considered stochastically. A clear classification of uncertainty in structural design [4] distinguishes physical (intrinsic or inherent), measurement, statistical and model uncertainty. Based on the same source, different levels of the development of reliability methods can be distinguished:

- Level I methods are deterministic reliability methods that only use one characteristic value to describe each uncertain variable (standard deterministic design methods, i.e. allowable stress method).
- Level II methods use two values for the representation of each uncertain variable and a supplementary measure of the correlation between the variables.
- Level III methods introduce the joint probability distribution of the sum of the uncertain variables, calculating directly the probability of failure for a limit state function.
- Level IV reliability methods, are the most advanced, introducing the element of target cost to the principles of engineering in order to derive a technically feasible and at the same time economically optimized solution.

Current practice in the development of design methods and standards are mainly classified as Level III methods, while methods introduced in this report can be considered as Level IV methods.

## 1.3. Reliability analysis of offshore structures

Offshore structural reliability analysis becomes of particular importance recently considering numerous changes within the offshore industry. Introduction of the Load Resistance Factor Design format in standards has significantly contributed to a more systematic design of offshore structures (compared to the global safety factor approach). Further, the introduction of a performance-based design approach which stands as a requirement as well as a target restriction in the structural design, allows more flexibility in the procedure of the design of offshore structures. This has resulted in the establishment of basic guidelines for a thorough reliability based design.

The above changes combined with the increasing need for a better understanding of the performance of structures throughout their service life in aspects of inspection, maintenance and reliability have created a wider acceptance framework for reliability assessment methods. More accurate modelling techniques and tools together with a higher available computational capacity, allow for analytical assessment of the reliability evaluation of structures in different stages and under different loading and capacity conditions.

Structural reliability analysis can provide significant benefit to the potential safety and cost; however the level of confidence of a reliability assessment strongly depends on the uncertainty consideration, accuracy of modelling and simplifying assumptions made. Structural problems, and even more extensively in offshore environments, are in most cases non-deterministic, with limited information and knowledge in both the conceptual and the design phase. Therefore, risk quantification yields for stochastic (random) variables to be considered.

During the last decades, momentous developments have occurred in the methodology as well as the tools for calculation of structural reliability. At a component level, methods such as first order reliability method (FORM) and second order reliability method (SORM) have been widely used, proposing modifications to account for complicated formulations of limit state functions and transformation of complicated statistical distributions to a normalized u-space. Further, simulation techniques, such as the widely known Monte Carlo Simulation, have been introduced overcoming limitations of the deterministic techniques. At a system level, approximation methods are sufficiently developed to build models that can simulate the response of the structural system efficiently, allowing for a global assessment of the reliability of an asset.

## 1.4. Design methods and standards for the design of offshore Structures

Development of our knowledge of structures and their behaviour, and the tools that are available, have advanced design procedures and methodologies applied on the design of structures. A classification of design methods can illustrate the following four categories which stand as a basis for the development of relevant design standards.

- Permissible stresses: This is the first widely accepted approach to systematic design, also noted as "allowable (working) stress method". It is in line with the linear elastic theory. The condition that the design should satisfy is:

  $$\sigma_{max} < \sigma_{per} \text{ or } \sigma_{max} < \frac{\sigma_{crit}}{k}$$

  Where the coefficient $k$, also noted SF by safety factor, is the only explicit measure considered to account for all types of uncertainties.

- Global Safety Factor: The method of global safety factors is based on the relation between the mean values of the structural resistance $R$ and the load effects $E$. The ratio of the two specifies the quantity of the global safety factor.

  $$s = \frac{R}{E} > s_0$$

  The value of $s_0$ should be defined and is the target that the designer should aim to meet.

- Partial Safety factor: The method of Partial Safety Factors is the widely used method to the establishment of design methodologies. It is also called 'Limit State Method' because it is applied in parallel with the concepts of limit states design for different design conditions. The method is advanced considering that it gives potential for mathematical optimization in several aspects. It can be summarized as follows:

  $$E_d(F_d, f_d, a_d, \theta_d) < R_d(F_d, f_d, a_d, \theta_d)$$

  Where: $E_d$ and $R_d$ represent the design values of actions effects and resistance respectively, $F_d = \psi \cdot \gamma_F \cdot F_k$ design values of variables describing the actions, $f_d = \frac{f_k}{\gamma_m}$ describes the material properties, $a_d$ describes the geometrical uncertainties, and $\theta_d$ the model uncertainties. The design values derive from the corresponding characteristic values of the variables $(F_k, f_k, a_k, \theta_k)$, applying the required partial factors $\gamma$, reduction factors $\psi$ and any other specified factor, which are the control values of the reliability of the design.

- Probabilistic Methods: These methods are the most advanced that have been proposed. Their basic requirement is that during the service life of a structure the probability of failure does not exceed an acceptable design value. This can be expressed as:

$$P_f \leq P_d \text{ or } \beta > \beta_d$$

The above two expressions are equivalent. The design values that should be fulfilled can be determined by the specifications of the structure. In addition to the Partial Safety Factor method, concepts of probabilistic analysis can be used to optimize the values of the partial safety factors.

The design methods as presented above, starting from the permissible loads method and heading to the fully or partially probabilistic methods become more complicated, demanding greater engineering and mathematical skills. However towards the same direction, the level of conservativeness and therefore the over sizing of the structures is reduced leading to more efficient structures with a better understanding of their service life performance.

A list of standards are available for the design of offshore structures:

- API RP-2A: Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms WSD/LRFD
- ISO 19902/A1:2015: Petroleum and Natural Gas Industries – Fixed Steel Offshore Structures
- BS EN 1993-1-1:2005+A1:2014 EUROCODEs 3: Design of Steel Structures
- NORSOK N-004 Standard for Design of Steel Structures
- IEC 61400-3:2009 Wind Turbines – Part 3: Design Requirements for Offshore Wind Turbines
- ANSI/AISC 360-10: Specification for Structural Steel Buildings
- DNV Offshore Standards
- GL Guideline for the Certification of Offshore Wind Turbines
- DNV GL OTG: Offshore Technical Guidelines
- DNVGL-ST-0126: Support Structures for wind turbines
- ABS Guides for Building and Classing Onshore/Offshore Wind Turbine Installations

## 1.5. Limit state design

The general design requirement is to provide structures with adequate safety margins in order to account for all types of uncertainties affecting its integrity (load and capacity variability, modelling idealizations etc). A simplistic definition of limit state design indicates that the demand (load) of a structural system should under no conditions exceed its capacity (resistance). Considering a case of multiple loading, the safe region criterion should be expressed as:

$$D_d = \gamma_o \sum_i D_{ki}(F_{ki}, \gamma_{ki}) < C_d = C_k/\gamma_M$$

In the above expression, index $k$ represents the characteristic value of a load or resistance variable while index $d$ the design values that incorporate the required magnification or reduction to account for consideration of uncertainties. Load variables are magnified with load factors $\gamma_{ki}$ in order to account for unforeseen events, while the capacity is diminished with the material factor $\gamma_M$ in order to account for capacity uncertainties (material properties, quality of construction, corrosion etc). A further partial safety factor $\gamma_o$ is added to consider the seriousness of the examined limit state to the integrity of the structure.

The characteristic (nominal) value of a variable is defined by its statistical properties. For a capacity variable, it can be based on the lower bound or 95% exceedance value, while for a load variable, the characteristic value on an upper bound or a 5% exceedance value. Derivation of partial safety factors is based on either previous experience or through a rigorous procedure that provide acceptable levels of safety and performance. In the previous methodology of allowable (or working) stress design the basic concept was to make sure that the response of the structure due to loads acting on it will remain below specific levels throughout the service life of the structure. The limit state design approach systematically examines the response of the structure under various conditions it might have to withstand, as a combination of loads and capacity.

For offshore and marine structures, several limit states are proposed by regulatory bodies and classification societies while [5] presents some common limit states for the four main types of limit states that should be considered; Serviceability limit state (SLS), Ultimate limit state (ULS), Fatigue limit state (FLS), and Accidental limit state (ALS).

## 1.6. Target reliability levels

Structural design aims to develop structures that are able to perform adequately, compromising cost and safety. The consequence of failure is an important parameter that should be assessed in order to specify the potential of injury or life loss, economic losses (direct and indirect), environmental pollution etc. Quantification of consequences is a demanding task influencing risk-based design approaches. Previous experience for a class of structures is an essential guide for the determination of target reliability levels for similar structures.

On the other hand, innovative structures cannot follow provisions of existing standards that refer to different structures since loading behaviour and consequences of failure strongly depend on their service, even if more conservative loading factors are adopted. This practice would drive the cost of the design without ensuring sufficient levels of reliability. One typical example of this problem, refers to the design of offshore wind turbines with jacket type foundation; although the general layout of the structure is similar to that of an offshore oil and gas platform, the loads added due to the rotor, the fact that the structures are unmanned and the large scale of production, constitute standards that refer to the latter application unable to ensure sufficient levels of reliability. For such cases, a robust reliability based design should be adopted that would allow from basis derivation of load factors applicable to each case [6].

Design according to standards can achieve minimum levels of target reliability. Some standards clearly state the target reliability they aim for, such as the EUROCODEs, while others rely on producing sufficient structures when the provisions of the code are followed as close as possible. Obtaining a better understanding of structures, may be update the reliability performance of standards. Table 1, summarises indicative values of target reliabilities found in various references, while in Table 2, an interesting classification against classes and mode of failure is presented.

*Table 1: Indicative values of target reliability*

| Standard | Target reliability value | Reference |
|---|---|---|
| ABS (American Bureau of Shipping) 'Rules for ship structures' | 3.15-3.65 | [7] |
| API recommended practices for Offshore Jacket structures (LRFD edition) | 3.35 | [8] |
| AISC | 3.5 | [9], [10] |
| Eurocode | 3.71 | [3] |

*Table 2: Values of acceptable annual probabilities of failure (PF) according to DNV*

| Class of failure | Consequence of failure | |
|---|---|---|
| | Less serious | Serious |
| I - Redundant structure | $P_F=10^{-3}$ | $P_F=10^{-4}$ |
| | ($\beta_t$=3.09) | ($\beta_t$=3.71) |
| II - Significant warning before the occurrence of failure in a non-redundant structure | $P_F=10^{-4}$ | $P_F=10^{-5}$ |
| | ($\beta_t$=3.71) | ($\beta_t$=4.26) |
| III - No warning before the occurrence of failure in a non-redundant structure | $P_F=10^{-5}$ | $P_F=10^{-6}$ |
| | ($\beta_t$=4.26) | ($\beta_t$=4.75) |

## 2. Reliability analysis of offshore structures

### 2.1. Basic formulation of the problem

The behaviour of a structure can be determined by the values of loads (actions) or load effects $L$ acting on it and its load bearing capacity (resistance) $R$. The following correlation between the two variables can form the acceptance criterion of the structure for a specific failure mode – limit state:

$$R - L > 0 \text{ or } \frac{R}{L} > 1$$

The safety margin, of the structure can be expressed as:

$$Z = R - L$$

In practice, both resistance as well as loading effects, involve a number of variables or material properties, subject to several sources of uncertainty. In the critical case where the resistance and load values are equal, limit state equations can be formed as: $Z(X) = 0$. In the case when $Z(X) \geq 0$ the structure operates in the safe region, while when $Z(X) < 0$, it is considered in the failure region. For each limit state, the probability of failure can be expressed as:

$$P_f = P\{Z(X) < 0\}$$

Considering probabilistic models for the assessment of the variables $X = [X_1, X_{2,\ldots,} X_n]$ and simplifying that they are described by time independent joint probability density function $\varphi_x(x)$, the expression of the probability of failure can be described with the integral:

$$P_f = \int_{Z(X)<0} \varphi_x(x)dx$$

The expression above can be extended to become applicable to some cases of time-dependent quantities which can be transformed into time independent ones [11]. For cases where this is not feasible, the process of calculating the probability of failure becomes more complicated and in practice should be assisted by different numerical methods.

Instead of using the term of “probability of failure”, the equivalent term of reliability index ‘$\beta$’ is usually referred to in the design standards and relevant documentation. This is the negative value of the standardized normal variable, corresponding to the probability of failure:

$$\beta = -\Phi_U^{-1}(P_f)$$

Where, $\Phi_U^{-1}(P_f)$, is the inverse standardized normal distribution function. The benefit of using this notation is that $\beta$ can provide results for several types of statistical distributions based on deterministic methods that will be presented later on in this report. Figure 1 [12], illustrates the correlation between the reliability index and the probability of failure.

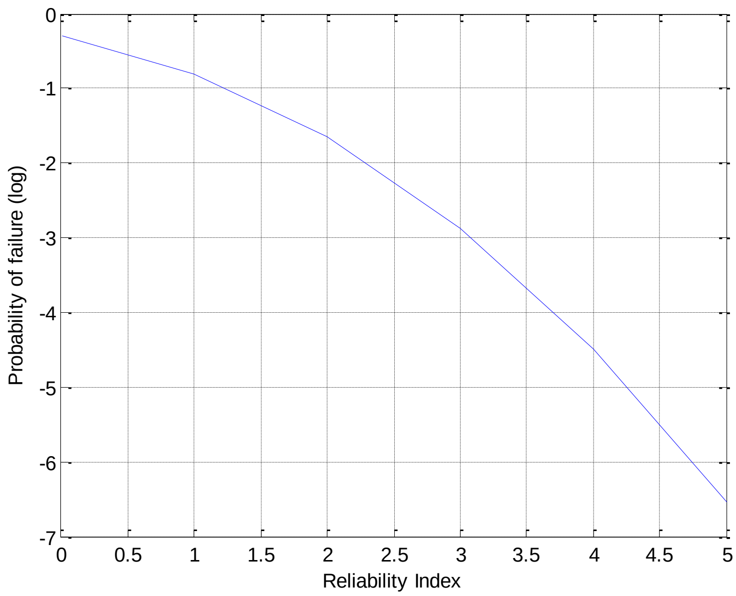


*Figure 1: Relationship between β and Probability of failure*

A quantitative definition of risk derives it as the product between the probability of occurrence of an adverse event and its consequences [13]. The first is influenced by the reliability of the structure while the latter of its function and specifications. This implies that different values of risk can exist for different combinations of these parameters. Unmanned offshore structures for example can experience failures that halt the operation/production but without fatalities and therefore the measured consequences can be considered lower; the same risk can be achieved with designs of increased probability of failure but lower consequences in the case of potential failure. Taking this into account, the calculated reliability levels can be interpreted.

An interesting parameter of the reliability index which represents the relative positions of the Load and Resistance distributions is its performance throughout the operational life-cycle. The load effects may increase with time due to crack growth etc, moving the corresponding curve to the right while the resistance might decrease due to deterioration of fracture toughness or other age related mechanisms. This fact leads to a decrease in the relative difference between mean values, which reflects to a decrease in the reliability index and therefore an increase in the probability of failure, as depicted in Figure 2. The importance of this characteristic is significant for the design process and the engineer should incorporate this time dependent component into the design model in order to avoid unwanted residual uncertainties in the calculations.

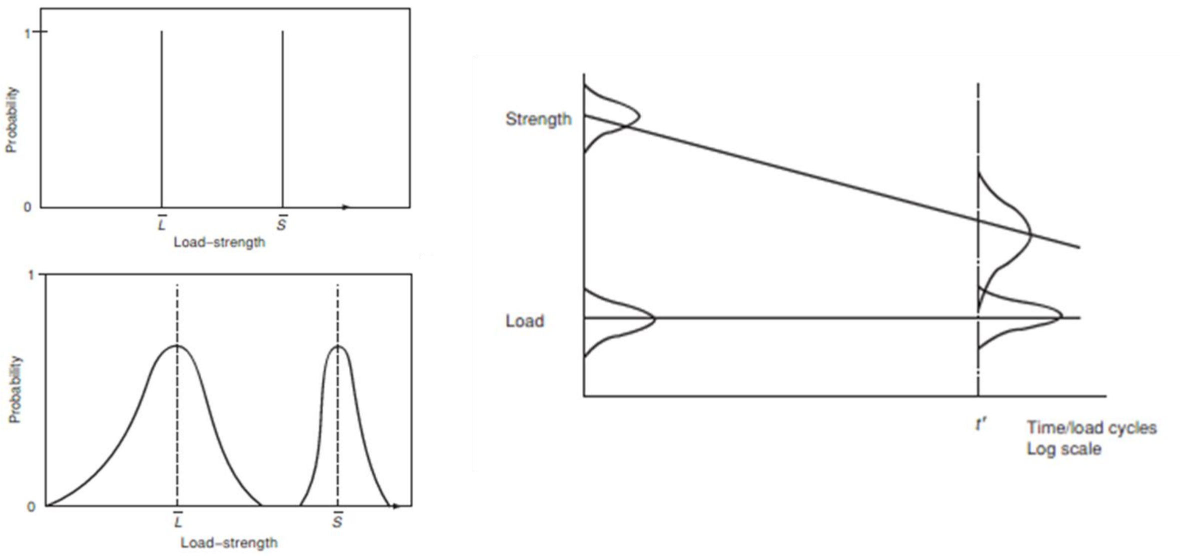


*Figure 2: Time-dependent reliability degradation*

## 2.2. Response analysis

A fundamental decision to be made before the analysis is the identification of the type that is required. Several analytical methods are available and may be categorized as static or dynamic, linear or non-linear, deterministic or stochastic. Combinations of the above categories can identify the analysis method to be employed, depending on the properties of the structure under consideration. The type of response of a structure may require different types of analysis.

Deterministic process is a process for which it is possible to describe the exact magnitude of the load at any given time. A deterministic analysis involves an initial consideration of the statistical data for environmental loading. For extreme response analysis, for example, a suitable event could be defined as the wave which is expected to cause the most severe response. This requires that the structural model is exposed to a unidirectional, periodic wave. The loading is calculated in the time domain at given points in time during a wave cycle. Contrary to deterministic processes, a stochastic process is described by the use of probabilities. Therefore, a stochastic load or response may not be fully described by exact magnitude at a given time, but rather by the probability (statistical distribution) by which it will exceed some specified value.

As mentioned earlier, engineering structural systems, often comprise of a number of components, hence should be analysed from a systems reliability point of view. A structural system with multiple failure paths can be represented by a series of parallel sub-systems, with each subsystem representing a failure mode. Starting at a component reliability level, the combined structural reliability of the system can be then calculated. In the case of complex structures such as offshore platforms, there is a number of potential failure paths and structural components which makes this approach not practical. The increased available computational resources for a system reliability analysis of an offshore platform has given motivation for the development of a number of “search algorithms”, in order to identify the most dominant failure paths and calculate the combined system probability of failure [14].

Identification of the most dominant failure path can be also performed by the so called ‘pushover analysis’. This analysis will identify deterministically the most critical members but will not take into account the effect of possible residual strength after failure which may redistribute loads and result in different sequences of failure and different combination of members. A simplified system reliability approach that can be applied as a preliminary design tool for configuration of new platforms, has been based on a series system where the components in series and within each component parallel elements so for each component to fail, failure of all parallel elements should happen [15]. The use of simplified analytical procedures to estimate reference storm lateral loadings and the ultimate capacities of platforms are comparably well in agreement to those derived from more complex analysis.

Reliability of a structural member strongly depends on whether it behaves in a brittle or a ductile way. This does not mainly refer to the material of the structure but in the behaviour of the potential members to fail. The response of the component's post-failure behaviour is one of the key factors that determine the effective redundancy of a structure. The two extreme types of failure are the perfect brittle and the prefect ductile failure performance. The first type becomes completely ineffective after failure, eliminating completely its load-bearing capacity. If a failure element maintains its load-bearing capacity after failure it is categorized as ductile. Real materials, in most of cases lay between the two extreme categories. One model which can be incorporated in the probabilistic analysis is the bi-linear, two state model. In the non-failure condition, the component is linear elastic, while in the failed condition the component still behaves linearly but with a modified stiffness matrix. With this type of models, various component behaviours, ranging from brittle to prefect plastic, can be described. In the semi-brittle model, the member force increases elastically to the member capacity or resistance [16].

### 2.3. Structural reliability assessment methods

Extensive use of Finite Elements in various fields of engineering has evolved using stochastic field theory in structural engineering. The critical issue towards this scope is the discrete representation of stochastic variables and the corresponding interpretation of stochastic responses. Stochastic expansion is an efficient tool for reliability analysis [17]. The purpose of stochastic expansions is to consider uncertainties through a series of polynomials in order to investigate the reliability of a system. Combination of stochastic expansions together with principles of the Finite Element Analysis Methods provide a useful tool, the Spectral Stochastic Finite Element Method (SSFEM), towards an analytical assessment of the reliability of structural systems [18].

Stochastic Expansions could be classified in two categories: the non-intrusive and intrusive formulation procedures. An intrusive formulation is the one in which the representation of uncertainty is expressed explicitly within the analysis of the system, ie directly modifying the stiffness matrix of a finite element analysis procedure. On the other hand, non-intrusive formulations, represent uncertainties in a non-explicit way, treating the analysis code as a "black box" without requiring access to the analysis code. This method is called Stochastic Response Surface Method. The framework that will be developed as part of this work is based on non-intrusive stochastic expansions, allowing for utilisation of high fidelity tools that can accurately predict the response of the structural system.

### 2.4. Stochastic modelling of design variables

Selection of the number and statistical properties of the variables that will be modelled stochastically is among the key decisions towards an effective structural reliability assessment. Environmental loads such as wind and wave magnitude and direction as well as resistance properties such as material properties and geotechnical conditions, are often treated stochastically, while a number of references exist suggesting appropriate statistical distributions for each variable with related coefficients [4] (Figure 3). Figure 4 presents indicatively a Rayleigh distribution often employed for wave heights.

| Variable name | | Distribution type |
|---|---|---|
| Wind | Short-term instantaneous gust speed | Normal |
| | Long-term n-minute average speed | Weibull |
| | Extreme speed, yearly | Gumbel |
| Waves | Short-term instantaneous surface elevation (deep water) | Normal |
| | Short-term heights | Rayleigh |
| | Wave period | Longuet-Higgins |
| | Long-term significant wave height | Weibull |
| | Long-term mean zero upcrossing or peak period | Log-normal |
| | Joint significant height/ mean zero upcrossing or peak period | 3-parameter Weibull (height)/ Log-normal period conditioned on height |
| | Extreme height, yearly | Gumbel |
| Current | Long-term speed | Weibull |
| | Extreme, yealy | Gumbel |
| Forces | Hydro-dynamic coefficients | Log-normal |
| Fatigue | Scale parameter of SN-curve | Log-normal |
| | Fatigue threshold | Log-normal |
| Fracture mechanics | Scale parameter of da/dN-curve | Log-normal |
| | Initial crack size | Exponential |
| | P.O.D.-curve | Log-normal or Weibull |
| Properties | Yield strength (steel) | Log-normal |
| | Young's modulus | Normal |
| | Initial deformation of panel | Normal |
| Ship data | Still water bending moment | Normal |
| | Joint still water moment/draught | Joint normal |
| | Ship speed | Log-normal |
| | Model uncertainty of linear calculations | Normal |

*Figure 3: Standard variables and corresponding distribution types*

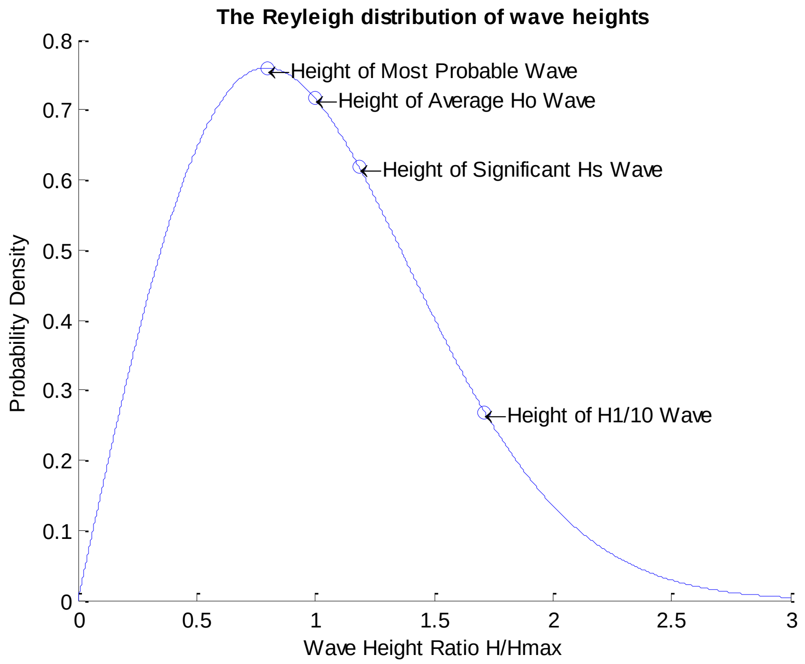


*Figure 4: The Reyleigh distribution of wave heights*

## 3. Fatigue reliability analysis

### 3.1. SN curve approach

The S-N curve method is based on fatigue test data (i.e. S-N data) and on the assumption that fatigue damage accumulation is a linear phenomenon which is independent of previously applied cycles. The fatigue behaviour of various types of structures is generally examined in constant-cycle fatigue tests and the results are normally presented in terms of the nominal applied stresses and the number of cycles of loading that produces failure. The resulting S-N curves are generally presented as straight lines on a log-log plot. The basic equation of S-N curve is given by:

$$N = \frac{A}{S^m}$$

where $N$ is the number of cycles to fatigue initiation (failure), $A$ is the intercept, and $m$ is the slope of the S-N curve in the log-log plot. The above equation can also be expressed in a linear form as:

$$\log_{10} N = A - m \log_{10} S$$

Given S-N data (i.e. a set of $N$ and $S$) obtained from fatigue tests, a statistical analysis method [19], [20] is generally used for analysing the S-N data to produce S-N curves and associated parameters (i.e. intercept $A$ and slope $m$).

Offshore wind turbine support structures generally consist of several steel cans, with adjacent cans generally connected together through welded joints. The fatigue strength of welded joints is dependent on plate thickness to some extent. This effect is due to the local geometry of the weld toe in relation to thickness of the adjoining plates, and it is also dependent on the stress gradient over the thickness. According to DNV-RPC203 [21], the thickness effect can be accounted for by a modification on stress ranges such that the design S-N curve for thickness larger than the reference thickness reads:

$$\mathrm{logN} = \mathrm{A} - \mathrm{mlog}\left(S\left(\frac{t}{t_{ref}}\right)^k\right)$$

where $t$ is the thickness through which a crack will most likely grow, and $t = t_{ref}$ is used for thickness less than $t_{ref}$; $t_{ref}$ is the reference thickness equal 25mm for welded connections other than tubular joints, $t_{ref} = 32mm$ for tubular joint, and $t_{ref} = 25mm$ for bolts; $k$ is the thickness exponent on fatigue strength.

The parameters involved in the S-N curve, i.e. intercept $A$, slope $m$, and thickness exponent $k$, are dependent on the material types, environmental conditions, whether having cathodic protection, etc. Taking DNV C1 and D curves as example, the S-N curves in air, in seawater with cathodic protection and in seawater for free corrosion are presented in Table 3.

*Table 3: DNV C1 and D S-N curves*

| S-N curve | Environment | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | In air | | | | | In seawater with cathodic protection | | | | | In seawater for free corrosion | |
| | $N \le 10^7$ | | $N > 10^7$ | | $k$ | $N \le 10^7$ | | $N > 10^7$ | | $k$ | $A$ For all cycles $m = 3.0$ | $k$ |
| | $m_1$ | $A_1$ | $m_2$ | $A_2$ | | $m_1$ | $A_1$ | $m_2$ | $A_2$ | | | |
| C1 | 3.0 | 12.449 | 5.0 | 16.081 | 0.15 | 3.0 | 12.049 | 5.0 | 16.081 | 0.15 | 11.972 | 0.15 |
| D | 3.0 | 12.164 | 5.0 | 15.606 | 0.20 | 3.0 | 11.764 | 5.0 | 15.606 | 0.20 | 11.687 | 0.20 |

### 3.2. Fracture mechanics approach

The fracture mechanics method is based on crack growth data of an initial defect of known (or assumed) geometry and size. For welded joints, it is assumed that an appropriate initial defect exists, which is just under the threshold of detection. The fatigue life can then be calculated using the fracture mechanics method to obtain the number of cycles needed to develop the crack to a certain unstable growth. The fracture mechanics method is more detailed and it involves evaluating crack growth and calculating the number of load cycles that are required for small initial defects to grow into cracks large enough to trigger fracture. The growth rate of the crack is proportional to the stress range, and it can be expressed in terms of a stress intensity factor $K$, which accounts for the current crack size, magnitude of stress, weld and joint details. The basic equation governing the crack growth is defined as:

$$\frac{da}{dN} = C\Delta K^m$$

where $a$ is the crack size; $N$ is the number of fatigue cycles; $\Delta K$ is the range of stress intensity factor; $C$ and $m$ are empirically derived crack propagation parameters.

The range of the stress intensity factor $\Delta K$ is given by [22]:

$$\Delta K = SY(a)\sqrt{\pi a}$$

Where $Y(a)$ is a function of crack geometry.

Failure is assumed to occur when the crack size $a$ reaches some critical crack size $a_{cr}$. Although most fatigue tests are performed with constant-amplitude stress ranges, the equation above is generally applied to variable stress range models that ignore sequence effects [23]. By rearranging the variables, the number of cycles $N$ can be expressed as:

$$N = \frac{1}{CS^m}\int_{a_0}^{a_{cr}} Y^m(a)\left(\sqrt{\pi a}\right)^m$$

The crack propagation parameters $m$ and $C$ in the equation above depend on material types, environments and whether having corrosion protection, etc. The typical values of $m$ and $C$ for offshore welded steels are listed in Table 4, taken from DNV-OS-J101 [24].

*Table 4: Crack propagation parameters*

| Condition | $m$ | $C$ |
|---|---|---|
| Welds in air and in seawater with adequate corrosion protection | 3.1 | 1.1e$^{-13}$ |
| Welds subjected to seawater without corrosion protection | 3.5 | 3.4e$^{-14}$ |

## 3.3. Formulation of fatigue limit state functions

The performance function for fatigue reliability analysis trough out the structures service life, is given by either one of the following two expressions:

$$g_1 = N - N_t \text{ or } g_2 = \log(N) - \log(N_t)$$

where $g_1$ and $g_2$ are performance functions; $N$ is the number of loading cycles to crack initiation; $N_t$ is the number of loading cycles expected during the design life of the structure. Using Miner's rule for cumulative fatigue damage with an effective stress formulation for a variety of loading, the equation above can be rewritten as:

$$g_1 = \frac{\Delta A}{\varepsilon' S_e^m} - N_t$$

where $A$ and $m$ are the intercept and the slope of the S-N curve, respectively; $\varepsilon'$ is the uncertainty in the S-N relationship; Δ is the Miner's rule damage at failure; and $S_e$ is the effective stress range for variable amplitude loading for the structure.

The effective stress range $S_e$ can be obtained by:

$$S_e = \sqrt[m]{k_s^m E(S^m)} = \sqrt[m]{k_s^m \sum_{i=1}^{n_b} f_i S_i^m}$$

where $k_s$ is the fatigue stress damage factor; $n_b$ is the number of stress blocks in a stress (loading) histogram; $f_i$ is the fraction of cycles in the $i$-th stress block; $S_i$ is the stress in the $i$-th block.

Combining the equations above, in a base-10 logarithmic form in terms of the fatigue variables yields:

$$g_1 = \log(A) - \log(\Delta) - m\log(S_e) + \varepsilon - \log(N_t)$$

where $\varepsilon = -\log \varepsilon'$.

In the fatigue damage ratio formulation, the performance function for fatigue reliability analysis is given by either one of the following two equations:

$$g_1 = \Delta - D \text{ or } g_2 = \log(\Delta) - \log(D)$$

where $g_1$ and $g_2$ are performance functions; $\Delta$ is random variable denoting fatigue damage at failure; $D$ is the fatigue damage expected during the design life of a structure. By using Miner's rule for cumulative fatigue damage with an effective stress formulation for variable amplitude loading, the equation above can be rewritten as:

$$g_1 = \Delta - \sum_{i=1}^{n_b} \frac{n_i}{N_i}$$

where $\Delta$ is the fatigue damage ratio limit having a mean value of one; $n_b$ is the number of stress range levels in a stress range histogram; $n_i$ is the number of actual load cycles at the $i$-th stress range level; and $N_i$ is the number of load cycles to failure at the $i$-th stress range level.

The equation above can also be expressed as [25]:

$$g_1 = \Delta_L - \sum_{i=1}^{n_b} \frac{n_i}{A k_S^m S_i^m}$$

Assuming a lognormal distribution of fatigue failure lives, the probability of failure can be then associated with a number of standard deviations from the mean S-N curve. Using two-standard deviations $(2\sigma)$ from a mean regression line that represents the S-N strength of fatigue detail:

$$g_1 = \Delta_L - \sum_{i=1}^{n_b} \frac{n_i}{10^{(log(A)-2\sigma)} k_S^m S_i^m}$$

where $A$, $m$ and $\sigma$ are generally obtained from line regression analysis of S-N data in a log-log space.

## 4. Reliability-based calibration of safety factors

### 4.1. General

In the previous sections of this report, evolution of design methods and standards has been documented and consideration of uncertainty in design has been discussed. Earlier standards relied mostly in experience for the derivation of mainly global safety factors while in the more recent limit-state design format of standards, uncertainty is taken into account more systematically through utilisation of partial safety factors which assign certain weights to different design variables. In presence of data available for the design and operation of a structural system, these partial safety factors can be further calibrated, removing the inherent generalisation of these safety factors. This section presents two numerical approaches that can be easily implemented on the design of offshore wind support structures for calibration of partial safety factors.

The target safety level for structural design of wind turbine support structures and foundations to the normal safety class is generally a nominal annual probability of failure of $10^{-4}$. This target safety is the level aimed at for structures, whose failure is ductile, and which have some reserve capacity. Additionally, this target safety is for unmanned structures. For wind turbines where personnel are planned to be present during severe loading conditions, design to high safety class with a nominal annual probability of failure of $10^{-5}$ is expected. Structural components and details should be shaped such that the structure as much as possible will behave in the presumed ductile manner. Connections should be designed with smooth transitions and proper alignment of elements. Stress concentrations should be avoided as much as possible.

### 4.2. Calibration of partial safety factors by calibrating design values

In the reliability based calibration of design values, these values need to be defined for all basic variables. The design is considered to be safe if the limit states are not reached when the design values are introduced into the analysis model. This can be expressed as:

$$R_d \geq E_d$$

where the subscript $d$ denotes design values, $E_d$ is the design load effect, $R_d$ is the corresponding resistance. The equation above represents a practical way to ensure that the reliability index $\beta$ is equal or larger than the target value.

$R_d$ and $E_d$ are respectively given by:

$$R_d = R\{X_{d1,}X_{d2,} \dots, b_{d1,}b_{d2,} \dots, \theta_{d1,}\theta_{d2,\dots}\}$$

$$E_d = E\{F_{d1,}F_{d2,} \dots, a_{d1,}a_{d2,} \dots, \theta_{d1,}\theta_{d2,\dots}\}$$

where $R$ is the resistance, $E$ is the load effect, $X_{dj}$ is the design strength of material $i$, $F_{dj}$ is the design value for load $j$, $b$ is a geometrical property, $\theta$ is a model uncertainty.

Design values should be based on the values of the basic variable (e.g. the FORM design point), which can be defined as the point on the failure surface ($g = 0$) closest to the average point in the space of normalised variables.

The design values of resistance $R_d$ and load effects $E_d$ should be defined such that the following equations are satisfied:

$$P(R > R_d) = \Phi(-a_R\beta_t)$$

$$P(E > E_d) = \Phi(a_E\beta_t)$$

where $\beta$ is the target reliability index; $a_R$ and $a_E$ are the values of FORM sensitivity factors.

The design values (such as $X_{d1,}$ and $F_{d1,}$) can be derived by solving the system of equations above. The relevant partial safety factor can be then obtained through dividing the design value of a variable by its representative or characteristic value.

### 4.3. Calibration of partial safety factors with partial factor format

An alternative way of reliability based calibration of partial safety factors starts with some arbitrary partial factor format and requires that the partial factors are chosen in such way that reliability of the structure is as close as possible to some selected target value.

Assume the partial factor format can be written as:

$$g\left(\frac{f_{k1}}{\gamma_{m1}}, \frac{f_{k2}}{\gamma_{m2}}, \dots, \gamma_{f1}F_{k1}, \gamma_{f2}F_{k2}, \dots\right) \geq 0$$

where $f_{ki}$ is the characteristic strength of material $i$, $\gamma_{mi}$ is the partial factor for material $i$, $F_{kj}$ is the representative value for load $j$, $\gamma_{f1}$ is the partial factor for load $j$.

The next step is to define a representative set of $n$ test elements, covering types of actions, structural dimensions, materials and limit states.

For a given set of partial factors $(\gamma_{m1}, \gamma_{m2}, \dots, \gamma_{f1}, \gamma_{f2,\dots})$, the set of representative structural element can be designed. Each element will then possess a certain level of reliability which will deviate more or less from the target value. With the help of the reliability index $\beta$, the aggregate deviation $D_a$ can be expressed as:

$$D_a = \sum_{k=1}^{n} \left[\beta_k\left(\gamma_{mi}, \gamma_{fi}\right) - \beta_t\right]$$

where $\beta_t$ is the target value of $\beta$, $\beta_k$ is the reliability index for element $k$ as a result of a design using a set of partial factors $\left(\gamma_{m1}, \gamma_{m2}, \dots, \gamma_{f1}, \gamma_{f2,\dots}\right)$,.

Obviously, the best set of partial factors can be obtained by minimising the aggregated deviation $D_a$ given in the equation above. If not all elements are considered to be equally important, weight factors may be applied.

### 4.4. Limitations of Structural Standards

Although in general use of standards results in design of structures with acceptable reliability, limitations arise for their application on novel and special structures, due to the fact that they primarily refer to specific structures and are presented in a high level that generally provides limited detail information and guidance on the background of the methodology they follow [26]. In this aspect the concept of reliability based design method can provide adequate support for the design of novel structures. Adopting the target reliability requirements from relevant standards, partial safety factors can be calculated independently, avoiding unwanted conservativeness imposed. Further, in areas of high uncertainty, design details are approached in such a way that the consequences of failure can be reduced (eg. structural redundancy, etc). The former can be realized by combination of different standards where appropriate, resulting in solutions that provide a reliable design that meets the specifications set.

During fabrication and service of the structure, safety elements can be introduced such as quality control, alignment control, visual inspection, instrumented monitoring and proof loading. Those practices provide information about the structure, additional to those available at the design stage, reducing the overall uncertainty. Once the manufacturing process is completed, a structural integrity monitoring system can compare real data to ones initially calculated, verifying the conditions of the structure. Data obtained, can provide, throughout its service life, all the necessary information having good confidence levels for life-cycle fitness-for-service assessments including cases following unforeseen events such as local collision or component failure. Therefore, current reliability can be calculated, identifying the actual condition of the structure and indicating the actions that should be taken for any required intervention as well as the ability of the structure to work above the initially considered service life. Finally, the database that has been created, can provide substantial information for relevant optimized future structures and systems.

## 5. Non-intrusive methods for structural reliability

### 5.1. Response surface analysis

In complex engineering systems, such as an offshore wind support monopile or jacket structure, a mathematical relationship between the actual loading acting on the whole structure (eg. wave or wind loads) and the actions that each member is subjected to (eg. axial force and bending moments) is difficult to be explicitly expressed. For such cases of complicated failure processes, simulation techniques can deal with the complexity of the problem; however, they are often inefficient for the calculation of small values of probability of failure, since a great number of iterations is required until sufficient results are derived. For such cases, where simulation techniques are computationally intensive, the stochastic response surface method (SRFM) can provide an accurate estimation of structural reliability, regardless of the complexity of the system under consideration [27], [28]. The concept of this method is the approximation of the actual limit state function, which in some cases can be unknown, using simple and explicit mathematical functions of the random (stochastic) variables affecting the response of the structural member or system. Those functions can be simple polynomials (eg. second or of higher order) with coefficients that can be calculated by fitting the response surface function to a number of sample points from calculation of the response of the member. In this explicit expression of the limit state function First and Second Order Reliability Methods can be applied for the estimation of the reliability index and therefore the probability of failure. Further, although the number of variables is the same in the response surface function and the initial limit state function, simulation techniques are more computationally efficient since this expression is less complicated than matrix manipulation.

Limitations of the Stochastic Response Surface Method arise in cases where the initial limit state includes non linearities or in cases where very low probabilities of failure should be accurately calculated which are caused due to the improper representation of the response surface based on arbitrary sample points that might be relatively far from the MPP [29], [30]. In order to overcome these restrictions, several methods have been proposed based on the adaptation of the response surface function to the location close to the design point [31]–[35] The accuracy of a highly non-linear limit state depends on the initial selection of sampling points.

Often the order of polynomials that is selected for the approximation of the response surface function is 2 (quadratic terms) since it demands few minimum sample points - (2n+1) - for the approximation of the coefficients of the function. The disadvantage of this practice is that it pre-assumes the shape of the target response surface which could lead to inaccurate approximation.

For the case where more than one independent or dependent variables are present, the fundamental equation can be solved providing adequate sets of $(y, x_i)$. The general problem can be described as:

$$y(x) = \sum_i a_i \cdot p_i(x_1, x_2, \ldots, x_n) + e$$

Considering monomials, this can also be described as:

$$y(x) = \sum_i a_i \cdot {x_1}^{\alpha_i} \cdot {x_2}^{\beta_i} \ldots {x_n}^{\omega_i} + e$$

Where: $a_i$ are the regression coefficients and $\alpha_i$,$\beta_i$ ,…, $\omega_i$ are the power coefficients for the independent variables.

For a case where the maximum monomial degree is 2, with 2 independent variables, the expression can be rewritten as:

$$y(x) = a_o + a_1 \cdot x_1 + a_2 \cdot x_2 + a_3 \cdot {x_1}^2 + a_4 \cdot {x_2}^2 + a_5 \cdot x_1 \cdot x_2 + e$$

Considering $Y$ to be a $(n \times q)$ data matrix containing the dependent variables, $X$ to be a $(n \times p)$ data matrix containing the independent variables, $A$ a $(p \times q)$ data matrix with the regression coefficients and $E$ is $(n \times q)$ matrix with the error terms. It forms the above equation in a matrix form:

$$Y = \tilde{X} \cdot A + E$$

Where $\tilde{X}$ denotes a matrix formed from X, containing the different powered values of X.

The above dimensions of the participating matrices imply that in order for the system to have a solution, $(p \times q)$ sets of data should be available. An important observation that can ensure accuracy in the regression coefficients results is the level of how well conditioned the matrix $X^T \cdot X$ is.

Forming the above equation in a matrix form:

$$Y = X \cdot \alpha + e$$

Where,

$$Y = \begin{bmatrix} y_1 \\ y_2 \\ \vdots \\ y_n \end{bmatrix}, X = \begin{bmatrix} 1 & f_1(x_1) & f_2(x_1) & \ldots & f_m(x_1) \\ 1 & f_1(x_2) & f_2(x_2) & \ldots & f_m(x_2) \\ \vdots & \vdots & \vdots & \vdots & \vdots \\ 1 & f_1(x_n) & f_2(x_n) & \ldots & f_m(x_n) \end{bmatrix}, a = \begin{bmatrix} a_1 \\ a_2 \\ \vdots \\ a_n \end{bmatrix}, e = \begin{bmatrix} e_1 \\ e_2 \\ \vdots \\ e_n \end{bmatrix}$$

The least squared method, expressed in a matrix form, is expressed as follows in order to derive the regression coefficients vector $a$:

$$a = (X^T \cdot X)^{-1} \cdot X^T \cdot Y$$

Having calculated the regression coefficients, the values of the dependent variables for the sampled dependent ones and the error for each of them is:

$$\bar{Y} = X \cdot a \text{ and } e = Y - \bar{Y}$$

The total sum of squares SST, regression sum of squares SSR and error sum of squares SSE are calculated as:

$$SST = Y^T \cdot Y$$

$$SSR = \overline{Y^T} \cdot \bar{Y} = a^T \cdot X^T \cdot Y$$

$$SSE = SST - SSR$$

In order to evaluate the level of accuracy of the modelled equation, a coefficient of determination ($R^2$) can be calculated as follows. The practical meaning of this equation, implies that when the regression sum of errors equals zero, and therefore $R^2 = 1$ the modelled function satisfies all of the sets of $(y, x_i)$ and therefore absolute regression has been achieved.

$$R^2 = 1 - \frac{SSE}{SST}$$

## 5.2. Other methods

The general procedure to develop an approximation model usually involves the following steps [36]:

- Identification of the design space: definition of the allowable values for design variables;
- Design of the experiment (DOE): identification of input design variable sets, $x_s$, to produce sampled system responses;
- Sample design space: implementation of direct simulations to produce the set of sampled responses, $y_m(x_s)$;
- Approximation model development: construction of the estimator function, $y_b$, interpolating sampled responses;
- Exploration of the design space: prediction of the responses at un-sampled locations of the design space.

Beside this common procedure, different modelling techniques are available. Generally, it is expected that more complex algorithms lead to higher accuracy results. Even if this assumption is verified in most of the cases, it should not be taken as a rule, since the effectiveness of the approximation is strongly dependent on the case study. Therefore, the accuracy of the approximation is consistently influenced by the method employed.

More refined techniques such as the radial basis function (RBF) [37] and Kriging [38] have proven their effectiveness towards higher complexity problems, especially in the case of non-linearities [39], [40] The performance of these methods has been improved by adding adaptive sampling to maximise accuracy prediction, but at the price of extended computational effort due to higher complexity algorithms. These techniques are addressed as dynamic RBF or Kriging.

Surrogate models realised using the above mentioned techniques are particularly suitable for uncertainty quantification. For example, while Kriging directly provides a prediction resulting from a stochastic optimisation process [38], MPR and RBF can be simply implemented in MCS. This feature enhances their applicability for problems in which a deterministic approach is not desired, such as probabilistic performance assessment.

Interesting relevant references with respect to radial basis function and ordinary kriging are the following [41]–[44].

## 5.3. A framework for structural reliability analysis based on non-intrusive stochastic expansions

This report recommends a 6 step approach to reliability assessment which can stand as an implementation framework for offshore and marine energy structures:

1. System definition: The system parameters should be defined and the target reliability levels should be set.
2. Define limit sates: The applicable limit states for the structural system of reference should be chosen to account all potential failure mechanisms that may occur during the assets service life (i.e. fatigue, ultimate strength, buckling, corrosion etc)
3. Determine stochastic variables: Design variables that are governed by uncertainty should be chosen here, with appropriate statistical distributions assigned to them according to design standards and best practice.
4. Execute simulations: For the defined variables a series of design cases should be listed in a design matrix, and a number of simulations should run, in order to map the response of the structural system

5. Develop response surface: For the set of inputs and outputs that have been gathered in the previous step, an approximation model will be fitted here to allow further analysis to take place.
6. Perform reliability analysis: Having defined the set of stochastic variables and the implicit or explicit expression of the approximation model, calculation of the probability of failure or reliability index can be performed through analytic (FORM) or stochastic (MCS) methods.

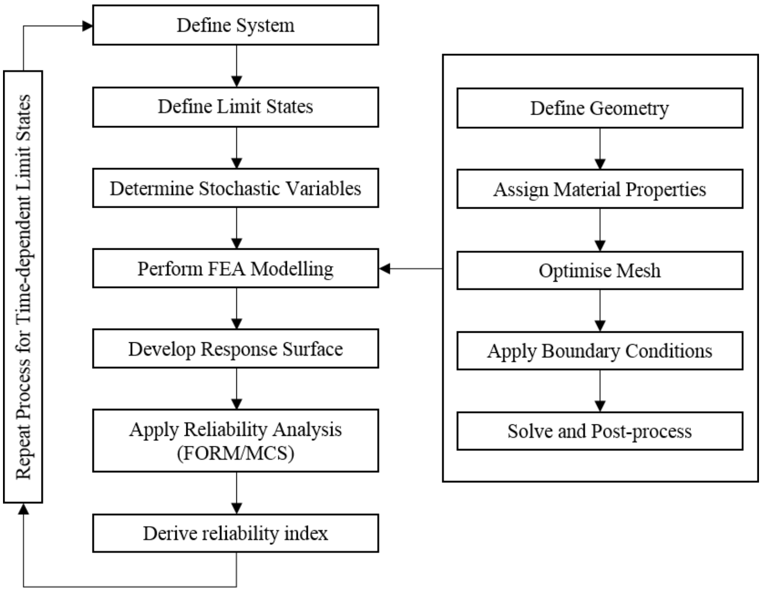


*Figure 5: Structural reliability analysis framework*

Typical parametric FEA models for offshore wind monopiles have been reported in literature [45], [46]. Having obtained the performance function as the expression of safety margin (allowable minus actual/resistance minus load), the FORM is used to estimate the reliability index through an iterative process. The principle behind this method is based on the fact that the random variables are defined by their first, second moments, and so on. The random variables are transformed in terms of their moments, and the reliability index can be assessed by the approximation of the limit state function.

The probability of failure is computed as:

$$P_f = \Phi(-\beta)$$

where $\beta$ is the reliability index, also defined as the minimum distance between the MPP (most probable failure point) and the origin, and $\Phi$ is the cumulative distribution function of a normal standard variable. Non-normal distributions can be accommodated through employing the Hasofer-Lind-Rackwitz-Fiessler method, which employs a normalised tail approximation to also account for non-normal distributions. The flowchart of the FORM analysis implemented in the study is presented in Figure 6.

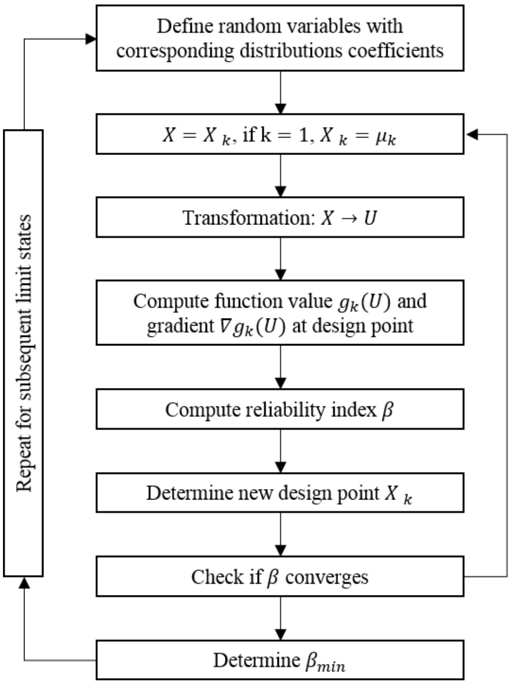


*Figure 6: FORM analysis algorithm*

# 6. Case studies

## 6.1. Case study 1: Reliability assessment of a typical jacket type structure

The first case study that is presented here, concerns that of a typical jacket type structure, as the concept of structural system becomes more evident here. Based on experience from offshore oil and gas platforms, the jacket-type support structure is a commonly selected configuration of offshore structures for medium and high water depths. Current interest for such locations considering massive deployment of wind turbines, sets a reference depth between 30-50 m.

For the application of the reliability assessment procedure that has been developed in the previous chapters, a typical structure will be examined for the reference depth of 50 m. The design depth in conjunction with the operational loads, will determine the general layout of the structure, in aspects of number of required legs, general layout and consideration of loading conditions. The depth of interest allows a four-legged configuration located in the corners of a square cross section at each elevation.

The structure is assumed to be constructed of tubular steel members of common 355 MPa steel with Young modulus of 200 GPa. Each of the legs is supported with a pile driven through the legs and extended to the seabed. The four legs are battered to achieve better stability against toppling, with a common bat angle with ratio 1:10. It has 5 elevations of horizontal and 4 of inclined full X bracing. The base elevation, which is positioned on the seabed, has dimensions of 25.0 m x 25.0 m and the structure extends above the water surface by 12 m, resulting to a total height of 62 m.

On top of the jacket support structure, an additional load will be considered to account for any operational loads acting on top of the support structure eg. the loads due to machinery on the top of an oil and gas platform, including the drag force imposed by the complicated geometry of the top side, or the aerodynamic loads induced by the operation of a wind turbine including the drag of the turbine tower. For this scope, an additional load will be applied on the top of the support structure proportional to the square of wind speed. The technique of ultra-stiff elements was employed in order to transfer the point loads to the legs, avoiding extensive stresses and deflection to the members of the top elevation. The structure has been designed with the commercial software DNV SESAM, specialized for the design of offshore steel structures. This software allows efficient modelling of environmental loads, including wave loads and piling, providing the static and dynamic response of structural members in the form of, bending moments, axial forces and principal stresses. The basis for the design of this structure was selected to be API RP 2A WSD, since this is the most conservative existing design standard, as it will be discussed in the next chapter. Design according to the provisions of this standard, has ensured that both buckling and ultimate strength capacity of members have been achieved. Table 5, presents the design load input parameters for dimensioning of the structure. For the numbering of members the following common notation has been followed: for legs, the string A0B characterizes the member, for horizontal braces, A1B, while for vertical braces A5B, where A represents the elevation and B the orientation of the member. Figure 57, illustrates the structure of reference.

*Table 5: Design load input parameters for dimensioning of the structure*

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| Significant Wave Height | 11.47 | m |
| Design Wave Height | 21.46 | m |
| Associated Wave Period | 13.3 | sec |
| Drag Coefficient | 1.05 | |
| Morison Coefficient | 1.2 | |
| Wind Speed | 25 | m/sec |
| Current Profile | MWL: 1<br>-25 m: 0.5 | m/sec |

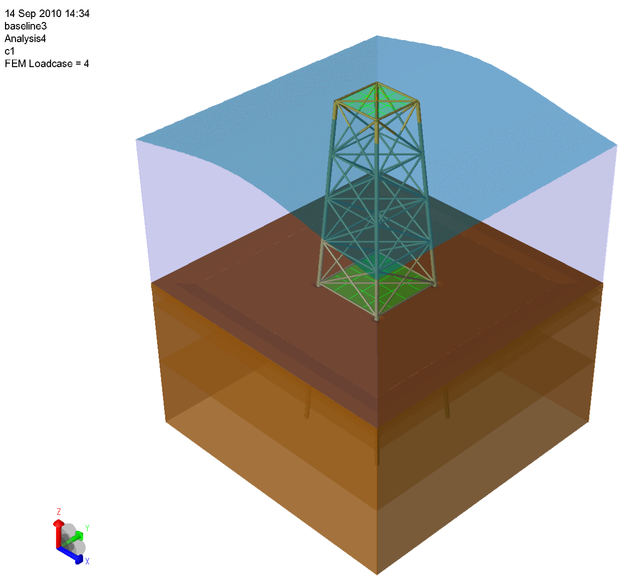


*Figure 7: FEA model of a jacket structure developed in DNV GeniE software*

This case study will focus on ultimate limit state in order to show applicability of the reliability framework, and accounts for four stochastic variables that will be considered in this analysis. Incorporation of more variables is feasible, however as it will be derived from the analysis, fewer variables should be modelled stochastically that have a greater effect on the structural response. The actual/allowable stress relationship will be derived based on the difference of the von Mises stress calculated from the simulations from the Yield Strength of the material. For the derivation of the reliability indices of each member, several simulations have been executed in FEA software and the results are imported in the MATLAB code that has been developed for the data regression and later the calculation of the FORM and SORM reliability index. The four variables that are considered stochastically are summarized in Table 6. Table 7 summarises the minimum reliability index of members incorporating different directions of loading.

*Table 6: Properties of stochastic variables*

| | ***Variable*** | ***Distribution Type*** | ***Coefficients*** | ***Units*** |
|---|---|---|---|---|
| $x1$ | Wave height | Reyleigh | $A = 8.08$ | m |
| $x2$ | Wind Force | Normal | $(400,40)$ | kN |
| $x3$ | Current | Normal | $(0.8,0.15)$ | m/sec |
| $x4$ | Yield | LogNormal | $(2.55,1.398)$ | MPa |

*Table 7: Minimum Reliability index of members, incorporating 8 different directions*

| ***Member ID*** | $min(\boldsymbol{\beta})$ | ***Member ID*** | $min(\boldsymbol{\beta})$ | ***Member ID*** | $min(\boldsymbol{\beta})$ | ***Member ID*** | $min(\boldsymbol{\beta})$ |
|---|---|---|---|---|---|---|---|
| **b116** | 35.01 | **b416** | 6.02 | **b104** | 4.50 | **b252** | 5.25 |
| **b115** | 35.01 | **b415** | 6.02 | **b103** | 4.50 | **b251** | 5.25 |
| **b114** | 54.83 | **b414** | 5.76 | **b102** | 4.50 | **b358** | 5.12 |
| **b113** | 54.83 | **b413** | 5.76 | **b101** | 4.50 | **b357** | 5.12 |
| **b112** | 54.83 | **b412** | 5.76 | **b158** | 5.54 | **b356** | 5.12 |
| **b111** | 54.83 | **b411** | 5.76 | **b157** | 5.54 | **b355** | 5.12 |
| **b216** | 16.38 | **b404** | 10.20 | **b156** | 5.54 | **b354** | 5.12 |
| **b215** | 16.38 | **b403** | 10.20 | **b155** | 5.54 | **b353** | 5.12 |
| **b214** | 9.33 | **b402** | 10.20 | **b154** | 5.54 | **b352** | 5.12 |
| **b213** | 9.33 | **b401** | 10.20 | **b153** | 5.54 | **b351** | 5.12 |
| **b212** | 9.33 | **b304** | 7.23 | **b152** | 5.54 | **b458** | 3.72 |
| **b211** | 9.33 | **b303** | 7.23 | **b151** | 5.54 | **b457** | 3.72 |
| **b316** | 9.05 | **b302** | 7.23 | **b258** | 5.25 | **b456** | 3.72 |
| **b315** | 9.05 | **b301** | 7.23 | **b257** | 5.25 | **b455** | 3.72 |
| **b314** | 6.94 | **b204** | 5.72 | **b256** | 5.25 | **b454** | 3.72 |
| **b313** | 6.94 | **b203** | 5.72 | **b255** | 5.25 | **b453** | 3.72 |
| **b312** | 6.94 | **b202** | 5.72 | **b254** | 5.25 | **b452** | 3.72 |
| **b311** | 6.94 | **b201** | 5.72 | **b253** | 5.25 | **b451** | 3.72 |

A series of parameters can be varied in the analysis that can influence the reliability of the asset. Such parameters can account for the wave models adopted, effect of the selection of statistical distributions, surface roughness coefficients etc.

Once of the important variables to be taken into account is that of corrosion modelling as corrosion is one of the most important phenomena related to capacity deterioration of the structure. Different models for this time-dependent phenomenon have been proposed and express this problem as a time-independent one through a relative decrease in the thickness of members. In this section, different models have been considered in the reliability assessment and the corresponding reliability deterioration throughout the structure's service life is presented. In Figure 8, the cumulative relative degradation in members' thickness is expressed based on the models examined. For the models where thickness decrease is expressed through statistical distributions (mean value and standard deviation) the thickness value that corresponds to probability of occurrence of 95% is considered. From the results that have been collected, Figure 9 presents graphs that illustrate the degradation of the reliability index according to each method for 2 critical methods of the jacket structure, while Figure 10 (i-iv) the reliability index according to each method for 4 members of one leg, as they represent values of reliability index of different range.

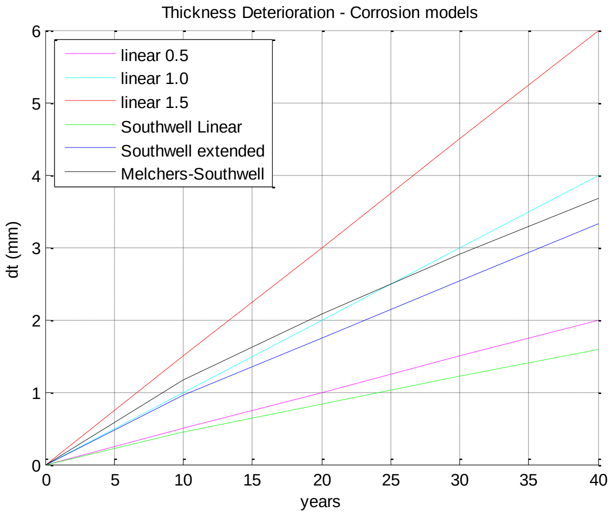


*Figure 8: Thickness deterioration as a function of time*

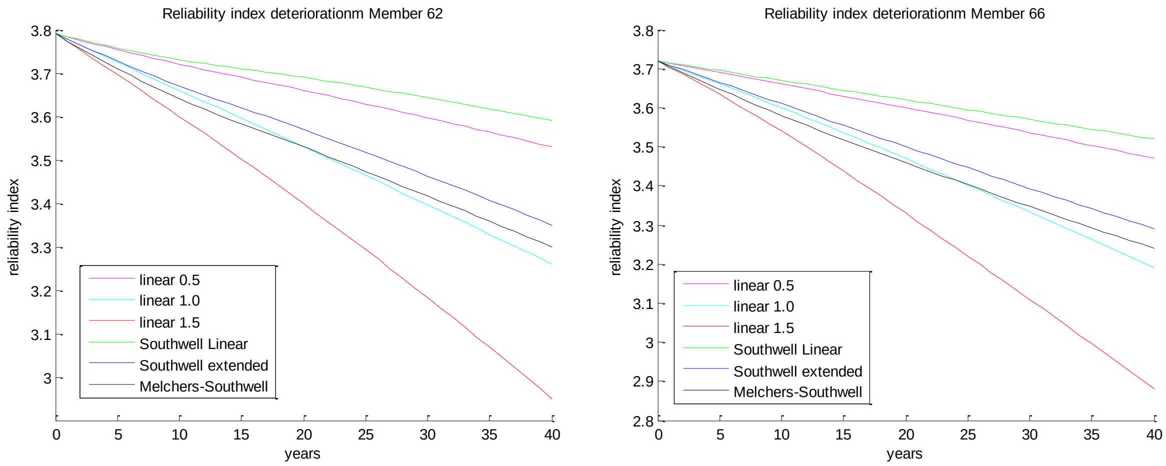


*Figure 9: Reliability index deterioration of critical members (b454, b458)*

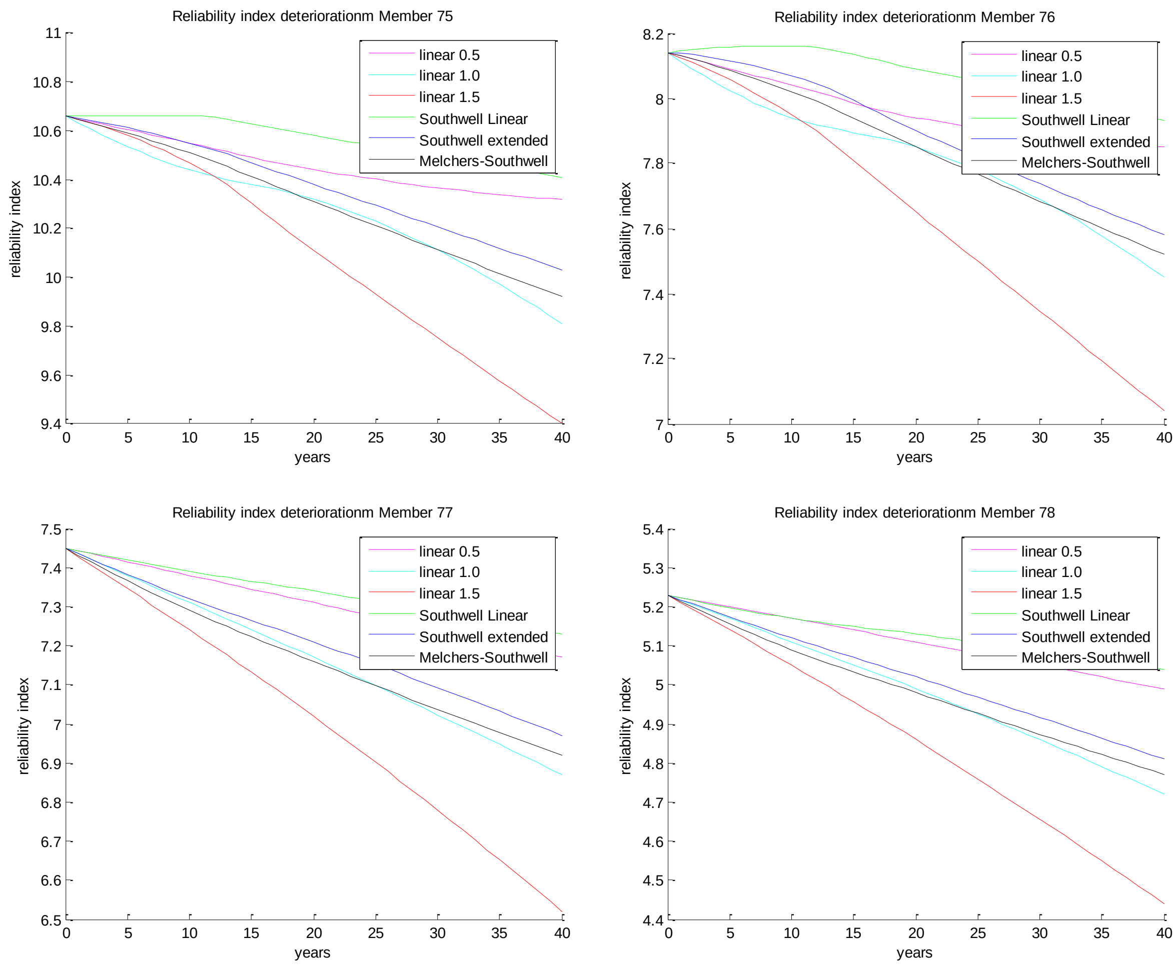


*Figure 10: Reliability index deterioration of members (b403-b103)*

## 6.2. Case study 2: Reliability-based structural optimisation of typical monopile

### 6.2.1. Structural optimisation model based on coupled FEA and GA

Before proceeding further, a structural optimisation model of offshore wind turbine monopiles based on coupled FEA and GA will be presented, together with the constraints/limit states taken into account.

a. Objective function

The reduction in offshore wind turbine monopiles is beneficial to reduce the material cost of the monopile, achieving successful and economic operation of an offshore wind turbine. Therefore, the minimum monopile mass chosen as the objective function $F_{obj}$, expressed as:

$$F_{obj} = min(m)$$

b. Design variables

The monopile embedded into the soil is designed to have a uniform thickness in order to facilitate the installation. Thus, six design variables are defined, which can be expressed in the following form:

$$X = [x_1\ x_2\ x_3\ x_4 x_5 x_6]$$

where $x_1$ is the thickness of Segments 1 to 7; $x_2$, $x_3$, $x_4$, $x_5$and $x_6$ are the thickness of Segments 8, 9 10, 11 and 12, respectively.

c. Constraints

The structural optimisation model of offshore wind turbine monopiles takes account of multiple constraints, such as the deflection, ultimate stress, fatigue and design variable constraints.

*Deflection constraint*

In order to ensure the overall structural stability and to avoid the uncertainties introduced by large deflection, the maximum monopile deflection $\boldsymbol{d_{max}}$ should not exceed the allowable deflection $\boldsymbol{d_{allow}}$. This constraint is given by the following inequality:

$$\boldsymbol{d_{max} \leq d_{allow}}$$

*Ultimate stress constraint*

The von-Mises stress $\boldsymbol{\sigma}$ generated by the loads should not exceed the allowable stress $\boldsymbol{\sigma_{allow}}$. This can be expressed in the following inequality form:

$$\boldsymbol{\sigma \leq \sigma_{allow}}$$

The allowable stress $\boldsymbol{\sigma_{allow}}$ can be expressed as:

$$\boldsymbol{\sigma_{allow}} = \frac{\boldsymbol{\sigma_y}}{\boldsymbol{\gamma_m}}$$

where $\boldsymbol{\sigma_y}$ is the yield strength; $\boldsymbol{\gamma_m}$ is the material safety factor, which can be obtained from design standards or calibrated on the basis of reliability.

*Fatigue constraint*

Fatigue is particularly important in structures subject to significant cyclic loads. In terms of fatigue constraint, the number of loading cycles to crack initiation should be not less than the number of loading cycles expected during the design life of the offshore wind turbine monopiles. This can be expressed as:

$$\boldsymbol{N} \geq \boldsymbol{N_d}$$

where $\boldsymbol{N}$ is the number of loading cycles to crack initiation, $\boldsymbol{N_d}$ is the number of loading cycles expected during the design life of offshore wind turbine monopiles.

The equivalent stress range $\Delta\boldsymbol{S}$ can be determined from FEA modelling of wind turbine monopiles subject to fatigue loads. Having obtained the equivalent stress range $\Delta\boldsymbol{S}$, the number of loading cycles to crack initiation, $\boldsymbol{N}$ , can then be determined from S-N curve, expressed as:

$$\mathbf{log}\,\boldsymbol{N} = \boldsymbol{A} - \boldsymbol{m}\,\mathbf{log}\,\Delta\boldsymbol{S}$$

where $\boldsymbol{A}$ is the intercept, $\boldsymbol{m}$ is the slope of the S-N curve in the log-log plot.

*Design variable constraints*

The thicknesses of the monopile generally increase from the monopile top to the mud line. This is ensured by following constraint:

$$\boldsymbol{x_i} - \boldsymbol{x_{i+1}} \geq \mathbf{0}, \boldsymbol{i} = \mathbf{1}, \mathbf{2}, \mathbf{3}, \mathbf{4}, \mathbf{5}$$

Additionally, each design variable is constrained to vary within a range defined by upper and lower bound. This constraint can be expressed as:

$$\boldsymbol{x_i^L} \leq \boldsymbol{x_i} \;\; \leq \boldsymbol{x_i^U} \;\; \boldsymbol{i} = \mathbf{1}, \mathbf{2}, \mathbf{3}, \mathbf{4}, \mathbf{5}$$

where $\boldsymbol{x_i^L}$ and $\boldsymbol{x_i^U}$ are the lower bound and upper bound of the $\boldsymbol{i}$-th design variable, respectively.

The lower and upper bounds of the design variables $\boldsymbol{x_1}$ represents thickness of segments 1 to 7 and $\boldsymbol{x_2}$ to $\boldsymbol{x_6}$ thickness of segments 8 to 12.

d. Development of parametric FEA model

A parametric FEA model of offshore wind turbine monopiles is established using ANSYS, which is a widely used FE software. The parametric FEA model enables the design parameters of wind turbine monopiles to be easily modified to create various monopile models. The flowchart of the parametric model of wind turbine monopiles is presented in Figure 11.

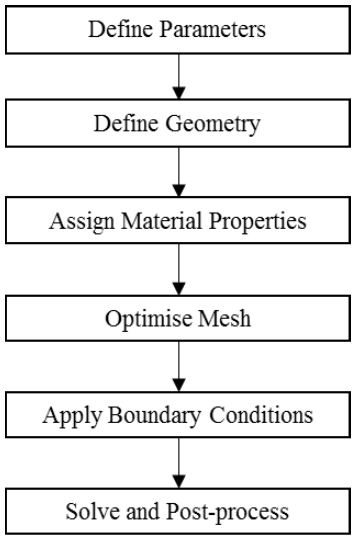


*Figure 11: Flowchart of the parametric FEA model for offshore wind turbine monopiles*

e. Genetic algorithm

GA is a search heuristic that mimics the process of natural selection. In GA, a population of individuals (also called candidate solutions) to an optimisation problem is evolving toward better solutions. Each individual has a set of attributes (such as its genotype and chromosomes) which can be altered and mutated. The evolution generally starts with a population of random individuals, and it is an iterative process. The population in each iteration is called a generation, in which the fitness of every individual is evaluated. The fitness is generally the value of the objective function in the optimisation problem being solved. The individuals with higher fitness are stochastically chosen from the current population, and the genome of each individual is modified (such as recombined and mutated) to form a new generation, which is then used in the next iteration. Commonly, the GA terminates when either the current population reaches a satisfactory fitness level or the number of generations reaches the maximum value.

GA generally requires a genetic representation of the solution domain and a fitness function to evaluate the solution domain. Each individual can be represented by an array of bits (0 or

1) or other types. Having defined the genetic representation and the fitness function, GA proceeds to initialise a population of candidate solutions and then to improve the population through repeatedly using mutation and crossover operators.

GA searches for optimal solutions through an iterative procedure, which is summarised below.

1. Define objectives, variables and constraints: The optimisation objectives, design variables and constraints are defined at the first step of GA.
2. Initialise population: Initial population (candidate solutions) is randomly generated in this step.
3. Generate a new population: In this step, a new population is generated through mutation and crossover.
4. Design point update: In this step, GA updates the design points in the new population.
5. Convergence validation: The optimisation converges when having reached the convergence criteria. If the convergence criteria have not yet been reached, the optimisation is not converged and the evolutionary process proceeds to the next step.
6. Stopping criteria validation: If the iteration number exceeds the maximum number of iterations, the optimisation process is then terminated without having reached convergence. Otherwise, it returns to Step 3 to generate a new population.

The above Steps 3 to 6 are repeated until the optimisation has converged or the stopping criterion has been met.

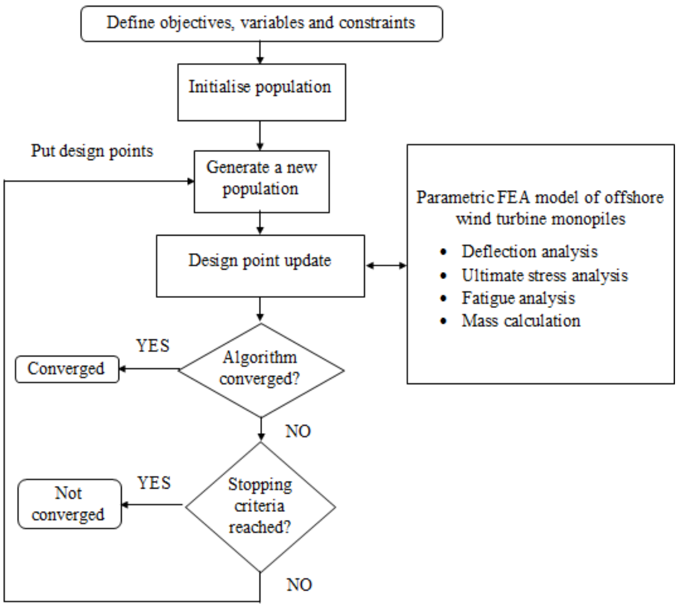


*Figure 12: Flowchart of the optimisation model*

### 6.2.2. Baseline Case

The case study will first specify a baseline monopile with an outer diameter of 5.5m and an overall length of 36m, consisting of twelve 3m-length segments (labelled 1-12 from bottom to top). 18m of the monopile are embedded into the soil, and the remaining 18m cover the distance from seabed level up to the sea water level. Soil-solid interaction is considered in this study, and the soil is modelled as a cylindrical body with a depth of 35m and a diameter of 60m. The distribution of thicknesses across the uniform diameter monopile is illustrated in Figure 13.

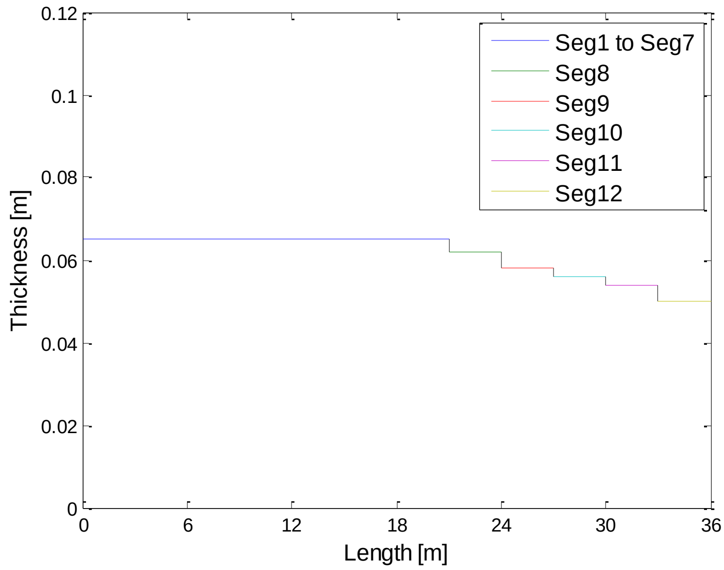


*Figure 13: Thickness distribution of the monopile*

The design of the baseline monopile considers both ultimate and fatigue load cases. For the ultimate load case, the extreme sea condition (i.e. 50-year extreme wind condition combined with extreme significant wave height) is taken as the critical ultimate load case. For the fatigue load case, both wind and wave fatigue loads for the normal operation of offshore wind turbine monopiles are considered. Table 8 lists the ultimate loads under extreme sea condition, and Table 9 presents the fatigue loads. The aerodynamic loads in Table 9 are taken from [47] for WindPACT 5MW wind turbine, which is a reference wind turbine designed by NREL (National Renewable Energy Laboratory), used in several benchmarking studies.

*Table 8: Ultimate loads under extreme sea condition*

| Item | Aerodynamic loads | Wave loads | Current loads |
|---|---|---|---|
| $F_x$ (kN) | 1,057 | 609.46 | 287.80 |
| $M_y$ (kN-m) | 135,000 | - | - |

Table 9: Fatigue loads

| Item | Aerodynamic loads | Wave loads |
|---|---|---|
| $F_{x,f}$ (kN) | 197 | 187.60 |
| $M_{y,f}$ (kN-m) | 29,874 | - |

A safety factor of 1.35 taken from DNV standard is used for aerodynamic, wave and current loads in the ultimate load case. For the fatigue load case, the safety factor is 1.0 according to DNV standard. A structural optimisation model based on coupled FEA (finite element analysis) and GA (genetic algorithm) is used to determine the thickness distributions of monopiles.

Fatigue reliability assessment is performed for the designed monopile. The stochastic load variables used in the fatigue reliability assessment are listed in Table 10. The COV in this table refers to the coefficient of variation, which is defined as the standard deviation divided by the mean for normal distributions. The selection of normal distribution in this stage is a simplification in order to facilitate calculations for the automated optimisation process. Further, documents such as the Probabilistic model code report of the Joint Committee on Structural Safety can suggest indicative values for COVs [48]. The calculated fatigue reliability index over 20-year design life is presented in Figure 14. As can be seen, fatigue reliability index reduces with time, reaching the lowest value of 4.54 at the end of 20-year design life.

Table 10: Stochastic load variables

| Item | Distribution Type | Mean | COV |
|---|---|---|---|
| Wave force | Normal | 187.60 kN | 0.1 |
| Wind force | Normal | 197.00 kN | 0.1 |
| Wind bending moment | Normal | 29,874.00 kN-m | 0.1 |

An initial application of the optimisation algorithm aims to optimise the monopile mass taking a target reliability index for the monopile at the end of 20-year design life is generally taken as 3.71, corresponding to a probability of failure of 10-4. Considering from figure above that the monopile has sustained a reliability index exceeding 4.5, a potential for further optimisation based on reliability-based design can be obtained. Therefore, the monopile can be further optimised to meet the target reliability index of 3.71. The thickness distributions of the initial and optimised monopiles are presented in Figure 15. These results, which are based on hypothetical statistical distributions (and hence should not be generalised) indicate that reliability-based design provides the potential to achieve a more cost-effective design when compared to the conventional partial safety factor based design. Further, a comparison of reliability deterioration of the initial and optimised designs is depicted in Figure 16. As can be seen from Figure 6, the reliability index of optimised design at the end of 20-year design life is 3.75, which is close to the target reliability index of 3.71.

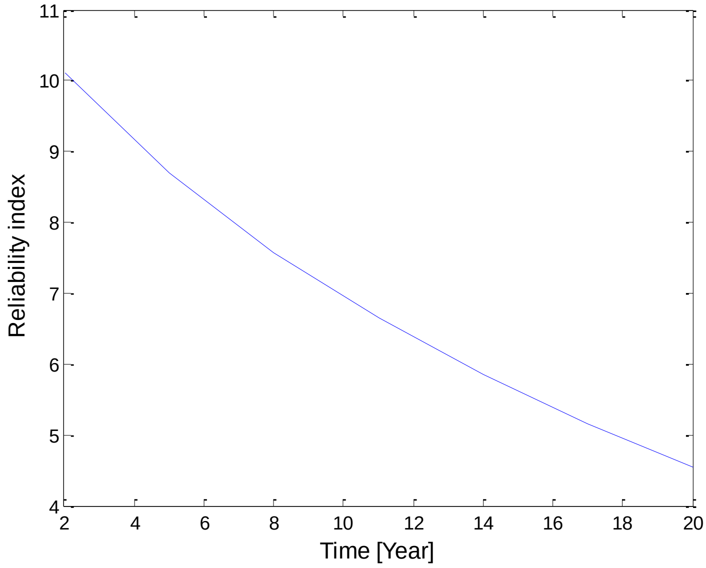


*Figure 14: Fatigue reliability index over 20-year design life*

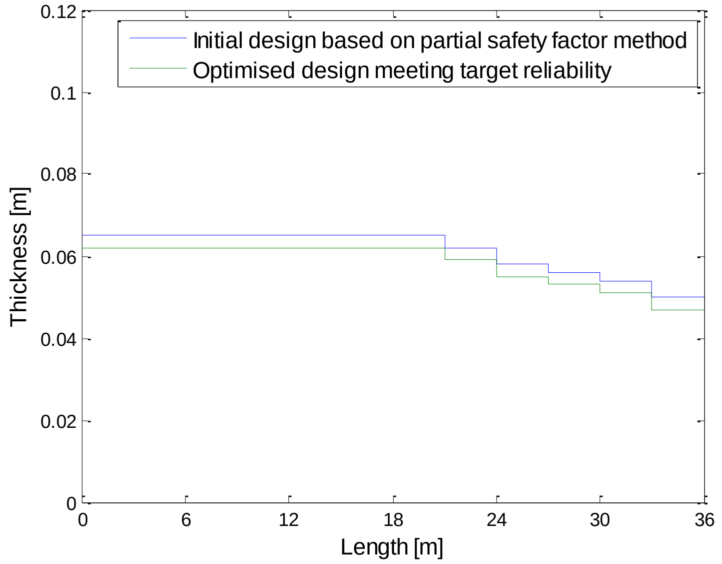


*Figure 15: Thickness distributions of initial and optimised design of monopile*

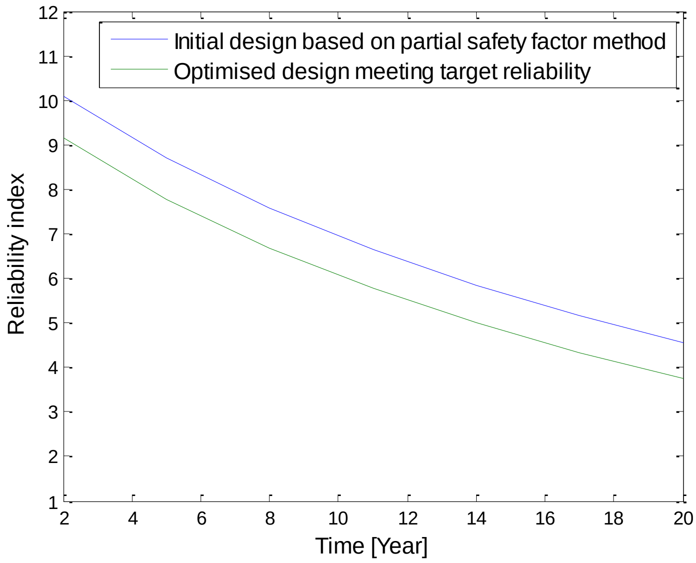


*Figure 16: Reliability index of original and optimised design*

Two of the factors that should be studied towards further understanding the impact of reliability levels for the design optimisation of an asset, are the target reliability levels and set design life period of the asset. The optimisation algorithm has run for both cases and results are documented accordingly. In Figure 17 and Figure 18, the optimised thickness distributions and reliability profiles are presented for target reliability index of 3.09 and in Figure 19 and Figure 20 the same figures for a reduced/extended service life period of 15/20/25 years.

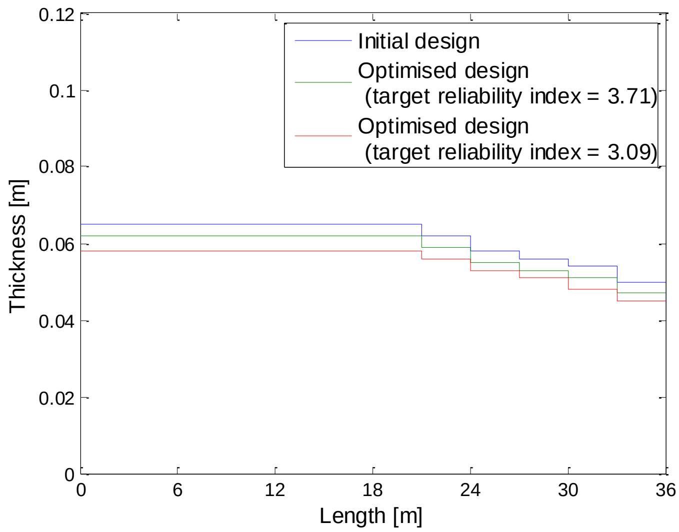


*Figure 17: Thickness distributions of initial and optimised design of monopile*

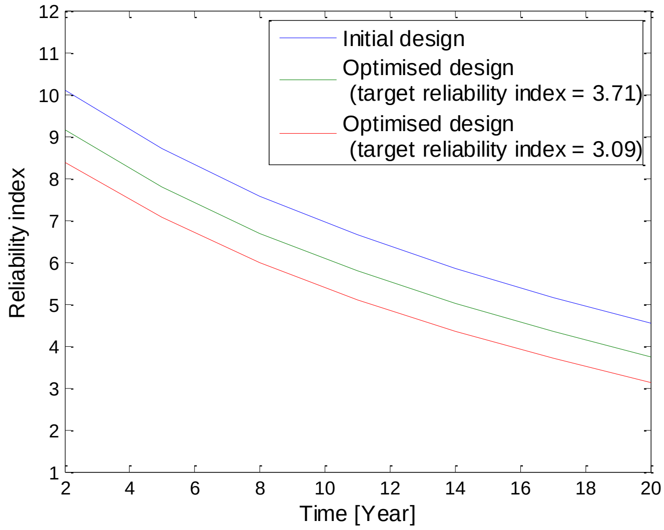


*Figure 18: Reliability index of original and optimised design*

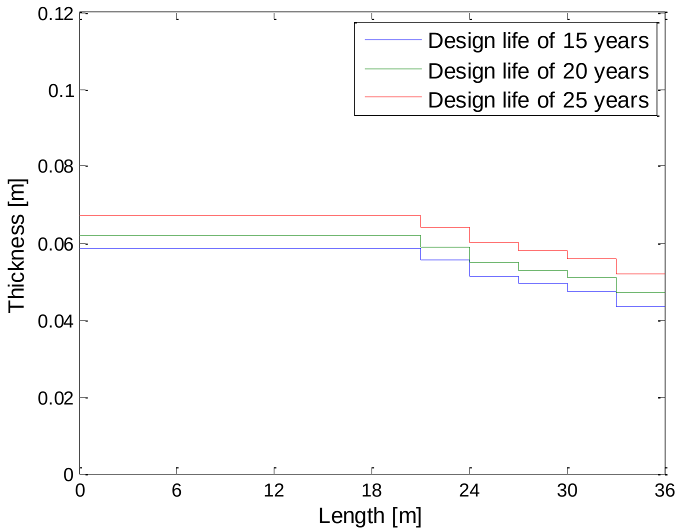


*Figure 19: Thickness distributions of monopiles with different design life*

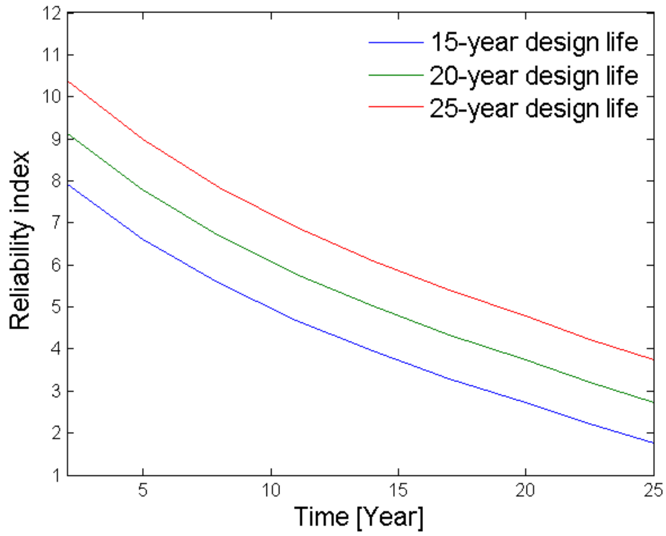


*Figure 20: Reliability index of monopiles with different design life*

## 6.3. Case study 3: Reliability-based safety factor calibration

Here we aim to compare the baseline design derived through application of safety factors as prescribed from the DNV standard against an optimised design with calibrated safety factors on the basis of target reliability. According to the DNV standard, the load safety factors for fatigue loads are 1, and therefore there is no need to calibrate safety factors for fatigue loads. For the purpose of this case study, only the ultimate load case is considered. The stochastic variables used in this study are presented in Table 11. The mean values are used as the characteristic values in the partial safety factor design method. The COV values correspond to the coefficient of variation and it reflects the uncertainties of the stochastic variables. The higher value of COV implies higher uncertainties of the variable. In this study, two cases (i.e. Cases A and B) are considered, and Case A has higher value of COV than Case B.

*Table 11: Stochastic variables*

| **Variable** | **Distribution Type** | **Mean** | **COV** | |
|---|---|---|---|---|
| | | | *Case A* | *Case B* |
| Yield strength of steel | Normal | 325.00 MPa | 0.05 | 0.025 |
| Wave force | Normal | 609.46 kN | 0.1 | 0.05 |
| Current force | Normal | 287.80 kN | 0.1 | 0.05 |
| Wind force | Normal | 1,057.00 kN | 0.1 | 0.05 |
| Wind bending moment | Normal | 135,000.00 kN-m | 0.1 | 0.05 |

The safety factors are calibrated to meet the target reliability index of 3.71, corresponding to a probability of failure of 10-4 (as per DNV-OS-J101 standard). The calibrated safety factors are presented in

Table *12* together for comparison with the safety factors given by the DNV standard. The obtained thickness distributions of the monopiles with DNV safety factors and calibrated safety factors (Cases A and B) are presented in Figure 21.

*Table 12: Safety factors*

| Item | Values given in DNV Standard | Calibrated values | | Note |
|---|---|---|---|---|
| | | Case A | Case B | |
| $\gamma_m$ | 1.35 | 1.17 | 1.08 | Partial safety factor for yield strength of steel |
| $\gamma_{f1}$ | 1.35 | 1.10 | 1.05 | Partial safety factor for wave load |
| $\gamma_{f2}$ | 1.35 | 1.10 | 1.05 | Partial safety factor for current load |
| $\gamma_{f3}$ | 1.35 | 1.26 | 1.13 | Partial safety factor for wind load |

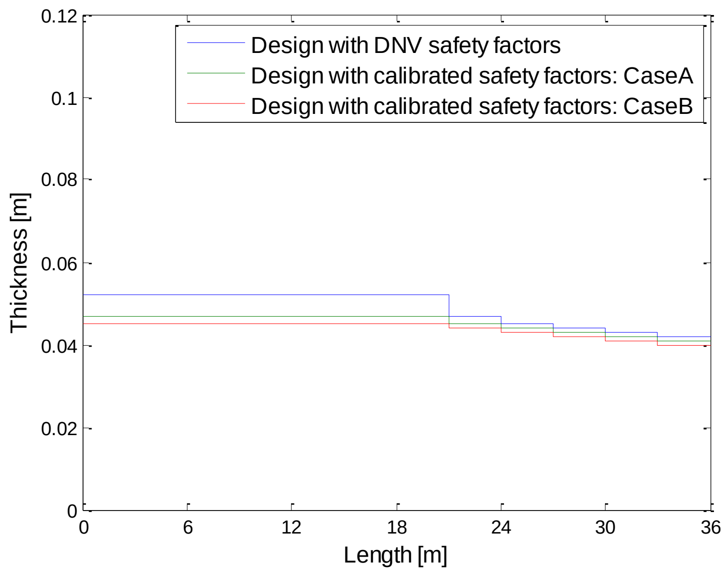


*Figure 21: Thickness distributions of monopile*

### 6.4. Case study 4: Structural reliability optimisation of a typical floating structure

Following the similar concept as above, the structural reliability assessment framework has been applied to the case of a floating structure, setting priorities based on reliability performance for future structural optimisation decisions. The basic geometry of the floating structure is presented in Figure 22.

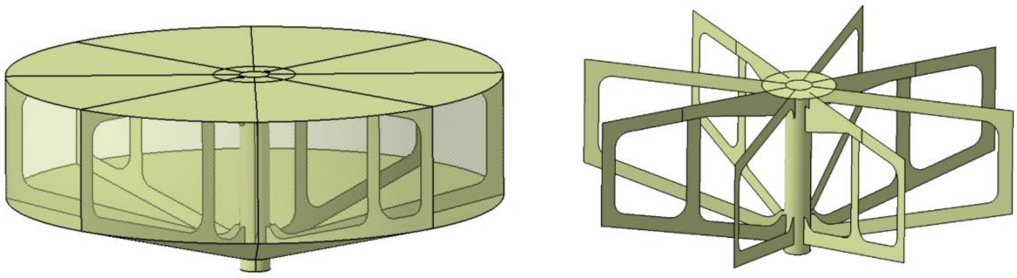

*Figure 22: Geometry: a cone-shaped floater, b internal reinforcements of the floater*

A typical S355 material has been chosen for the analysis with Young's modulus of 200 GPa, Poisson's ratio 0.3 and density 7850 kg/m3. A mesh sensitivity study illustrated an element size of 10 mm as providing adequate results for the analysis. The reliability assessment of this asset has been based on the fatigue limit state as presented earlier on in this report. The stochastic loads that were taken into account in this analysis are summarised in Table 13.

*Table 13: Stochastic parameters*

| Variable | Distribution | Mean $\mu$ | Standard deviation $\sigma$ |
|---|---|---|---|
| $E$ [GPa] | Normal | 200 | 30 |
| $D$ [kN/(m/s)] | Normal | 320 | 48 |
| $K$ [kN/m] | Normal | 100 | 15 |

The wave load is considered differently, based on its dependence on wave height and wave period. To find the peak of the force acting on the system, a truncated Rayleigh distribution is applied to the wave height allowing the system to consider that, in the case of storm sea conditions, the model can be taken out of the sea to avoid further damage. Once the maximum and minimum wave heights are defined, considering an increment and decrement of 45% from the base load, the Rayleigh distribution is implemented. This distribution is then fitted into a normal-equivalent distribution, to then follow relevant FORM procedures. The latter transformations allow for the extension of the traditional FORM to account for various types of statistical distributions, which can better fit certain variables. In the presence of observed data, distribution fitting algorithms (such as Akaike Information Criterion, Bayesian Information Criterion and Kolmogorov-Smirnov) are often employed to determine the shape coefficients of the most appropriate statistical distributions.

Having defined the stochastic variables, the FEA model is then used to perform a series of deterministic FEA simulations of the WEC floater, with the help of the Design of Experiments module and PDS (Probabilistic Design System) in ANSYS, in order to map the response domain and later on derive an appropriate response surface model. This allows the input parameters to be designated as stochastic variables, having different types of distributions. A number of simulations have been performed in ANSYS, including different design samples related to the different stochastic parameters selected. Each parameter is changed by incrementing and decrementing it with +3σ and -3σ in order to cover a reasonable and probable range of values. The results are then imported into a MATLAB code that has been developed in this work for response surface modelling, as detailed below.

The calculated reliability index of the floater under fatigue load case is presented in Figure 23. It can be seen that the structure does not achieve the reliability target of the 20 years design life, and the reliability index drops below the target reliability index (3.71) after about 4.1 years. This indicates that the design specification considered for this analysis will experience fatigue failure before its target life.

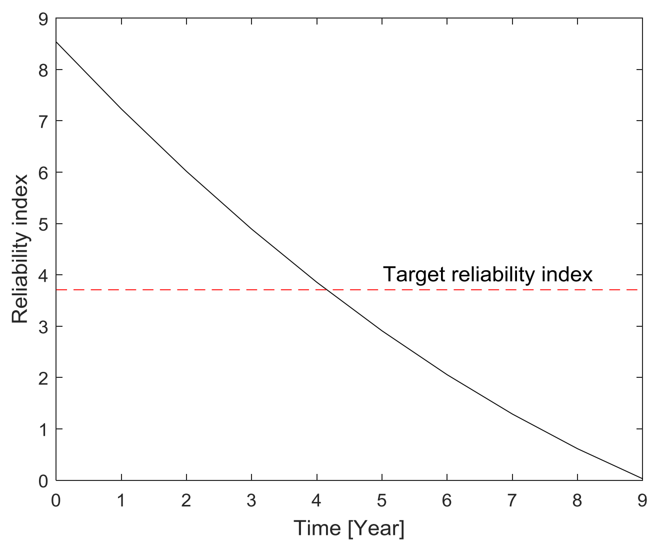


*Figure 23: Reliability index trend*

Additionally, performing all the simulations allows us to understand how the structure behaves in terms of maximum von Mises stress. A local sensitivity analysis can be performed in order to assess what the most critical and important parameters are that can radically change the response and reliability performance of the system. Figure 24 shows how sensitive to the four design variables the structure is.

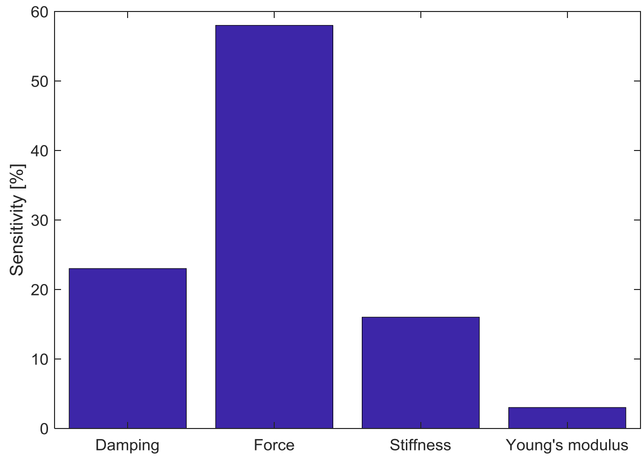


*Figure 24: Local sensitivity factors*

As expected, the most critical parameter is the wave force that accounts for 58% of the total sensitivity. Both stiffness and damping coefficients are around 20% and also expected is the fact that the Young's module is only 3%.

Starting from the results shown here, a further study is conducted to improve the behaviour of the structure from a fatigue life perspective. Moreover, considering that the crucial point of the system is Reinforcement A that goes to failure after about 4.1 years, some considerations are made starting from the thickness of this component.

It is clear that, in order to achieve the reliability target of 20 years, some changes have to be made in the design. The thickness of Reinforcement A, which is the first one that is expected to fail, is considered as a design variable in this case for refining the design.

A parametric analysis considering different thickness of Reinforcement A is performed, studying the fatigue life of the system with the S-N curve approach. The percentage of increment in the thickness starting from the base load and the design life is represented in Figure 25.

As can be seen in Figure 25, the design life of the model increases with the thickness of reinforcement. In this case, the target design life of 20 years is reached at a percentage increment of around 140% in the thickness of Reinforcement A. This indicates that the target design life of WEC floaters can be obtained through properly engineering the thickness of the floater structure. In this study, results of reliability analysis were used to inform improvements to the design in order to achieve target reliability levels. Robustness and efficiency of the method reported herein, allow for a further, more holistic, optimisation approach through appropriate constraints in order to minimise the objective function of mass structure while fulfilling the criterion of target reliability.

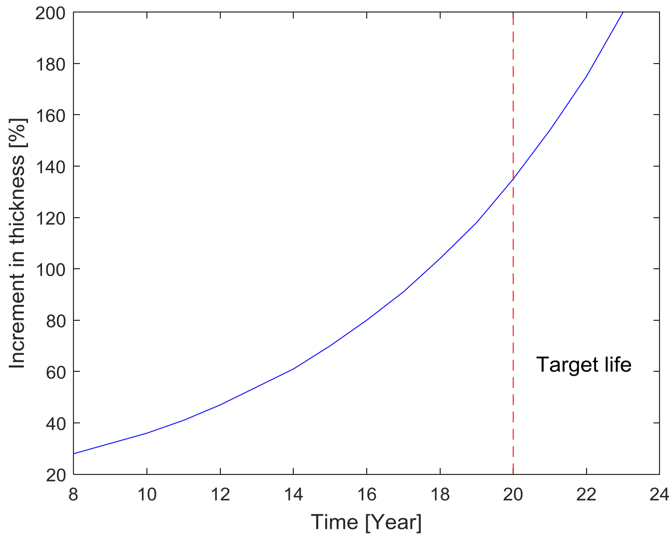


*Figure 25: Optimisation trend with increment in thickness*

## 7. Conclusions/Summary

This report documents fundamentals of risk-based design for the advanced design of next generation wind turbine support structures. Starting from fundamentals of risk and reliability, the evolution of design methods and standards are documented and the concept of target safety levels is set. Following, concepts of reliability analysis specifically applicable to offshore structures are presented showing the motivation to more advanced design methods and setting the specification of reliability-based concepts. Offshore structures are prompt to fatigue due to the cyclic loading they are subject to and hence relevant limit states are formulated analytically based on the S-N curve and fracture mechanics methods. A methodology is then documented for reliability-based calibration of partial safety factors which can link existing design methods with probabilistic concepts. Finally the response surface method is discussed, as it can allow a non-intrusive algorithm to be documented linking global loads to local load effects. ~Applicability of the concepts discussed and approaches suggested is illustrated through a series of 4 case studies.

The first case study concerns the reliability assessment of a typical jacket type structure against ULS, as a complex engineering system subject to a variety of stochastic loads and initially designed against conventional design standards. Following, a reliability-based structural optimisation algorithm is developed and applied to the case of a monopile with varying cross section, evaluating initially its reliability performance against fatigue and optimising it then for a set threshold of target reliability. A case study on the reliability-based safety factor calibration is also included in order to illustrate the benefits of including deployment specific information in the design of assets. Finally, the case of reliability assessment of a typical floating structure is presenting, in order to illustrate how reliability analysis can inform design optimisation.

It should be noted that this report does not aim to present the state-of-the-art in relevant literature, but rather develop a methods through gathering concepts from relevant applications towards designing support structures with better understanding of their service-life performance.

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