CRANFIELD UNIVERSITY

ORIBI RICHARD IVANHOE

GENERIC FRAMEWORK FOR THE RELIABILITY ASSESSMENT OF AGEING OFFSHORE WIND TURBINE JACKET SUPPORT STRUCTURES

SCHOOL OF WATER, ENERGY AND ENVIRONMENT (SWEE)
Department Of Energy And Power

Thesis Submitted for the Degree of Doctor of Philosophy
Academic Year: 2020 - 2021

Supervisor: Professor John Oakey
Associate Supervisor: Dr Ali Mehmanparast
December 2020

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ABSTRACT

Wind Europe annual report for 2019 shows that a significant amount of installed offshore wind turbines will reach their design service life in the next decade. Most of these structures will remain in service if granted permission for life extension. Life extension remains a complex decision given the limited experience in the offshore wind industry, and so structural integrity assessment of ageing structures is seen as a potential upcoming challenge. While the existing guidelines provide a general process for assessment, it is crucial to have robust offshore wind-specific guidelines by including relevant concepts and notation of the wider structural integrity assessment and models that can adequately account for the time-dependent degradation mechanism more accurately.

Offshore wind turbine (OWT) support structures are exposed to harsh marine environments with considerable uncertainties in the environmental loads and soil properties, making structural integrity assessment difficult. Hence, reliability assessment is seen as the most suitable approach to methodically account for these uncertainties. In this thesis, a generic framework for assessing the reliability of ageing OWT jacket support structures is developed based on a non-intrusive formulation. A parametric finite element analysis (FEA) model of a typical OWT jacket support structure was developed incorporating operational and environmental load and soil-structure interactions in order to map its response under varying input conditions appropriately.

The results from several FEA simulations have been analysed through multivariate regression, deriving performance functions and formulation of relevant limit states. for this assessment, five limit states were considered: deflection, buckling, vibration, ultimate and fatigue limit states. The reliability index under each limit state is then calculated using the first-order reliability method (FORM). The developed reliability assessment framework has been applied to the NREL 5MW OWT OC4 jacket design to determine the reliability of critical components of the structure. The results of this reliability assessment show that, for the given stochastic conditions, the structural components of the OWT jacket support structure are found to be within acceptable reliability levels as defined in DNV-OS-J101 Offshore wind turbine design standards. A robust inspection, maintenance and repair (IMR) plan, adequately executed, proved to boost the remaining life of the structure, thus, making life extension of ageing offshore structures more economical for owners of wind turbines and operators. The validity and applicability of the framework to the OWT industry were reviewed, and recommendations were made on the next steps required for the deployment of this framework in the offshore wind energy sector.

Keywords:

Life Extension, Finite Element Analysis, Soil-structure interaction, Non-intrusive formulations, IMR, FORM, Limit State Design

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Finally, to everyone that has supported me in any way throughout my journey in completing this PhD, my family, friends, colleagues etc., I may not have mentioned your name, but I am most grateful to you. I pray that God will reward you. Thank you

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1 INTRODUCTION

1.1 Background

Wind Europe reported a total of 205 GW of wind energy capacity, of which about 3.6 GW is offshore, which also accounts for 15% of the source of electricity in Europe (Wind Europe, 2020). With a central scenario projection of 320 GW of wind energy to be installed in Europe in 2030, 66 GW of the total energy is expected to come from offshore wind farms (EWEA, 2015). This interest to move offshore is mainly motivated by the higher wind shear, unrestricted space, and lower social impact in the marine environment (Lozano-Minguez et al., 2011). It is estimated that an additional 50% of electricity can be generated for the same wind turbine in an offshore environment. Key barriers towards further deployment of offshore wind farms are their high construction cost, especially foundation and electrical connection, and limitations in operation and maintenance, which constitute a high percentage of the life cycle costs (Ioannou et al., 2018).

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

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

Despite the vast technological advancement achieved in the offshore renewable industry in Europe, as established by the deployment of OWT's in shallow and deep waters environments (Lozano-Minguez et al., 2011), there are still challenges associated with the deployment of OWT. Most of which are higher capital expenditure (CAPEX) and operational expenditure (OPEX), as well as difficulty in accessing the facility (Ioannou et al., 2018).

Most of the installed OWT in operations offshore Europe are still well within their service life. Some others approaching their design life, which implies in few years, most of the structures in operation would have outlived their service year and are most likely to undergo life extension as witnessed in the oil and gas industry (Ersdal et al., 2014). 2016 Wind Europe annual report (2017) reported about 12% of the total installed wind capacity in Europe were older than 15 years, with a projection that by 2020, it would have risen to 28%. Therefore, it implies that more wind turbines would have reached the end of their service life, typically 20 years.

Consequently, the offshore wind industry must brace up for the challenges ahead, such as life extension of ageing assets, maintenance, structural integrity assessment or decommissioning. Life extension seems the most appealing and economic decision. Owners and operators of wind farms would have to make a decision based on critical evaluation of Economic, technical and legal aspects of the OWT and the support structure and the continental shelf it is located. The structure is expected to have sufficient remaining structural life for life extension studies and maintain a reliability index greater than the minimum target reliability index of 3.71 according to DNV-OS-J101 (DNV, 2015). Also, life extension could result in higher operation and maintenance costs as well as downtime. Also, the legislation in some sites prohibits the repowering of some existing wind farm sites.

The decision process still is complicated due to uncertainties, with not many references available (Lisa et al., 2018).

Ageing in offshore structures is a phenomenon by which the reliability of the structure or components deteriorates with time (Ersdal et al., 2019). It is also a reduction of the remaining life of the structure. Ageing in offshore structure can be categorized into four categories: functional ageing, knowledge-based ageing, technological ageing and organizational ageing (Hokstad et al., 2010; Hornlund et al., 2011; Ersdal et al., 2014). Functional ageing comprises mainly of material degradation due to fatigue and corrosion, dents etc. In contrast, knowledge-based ageing includes obsolete original detailed design documents due to new knowledge advancement, such as, the analytical method or new standards. Technological ageing may be due to old design regulations or standards that may not necessarily meet the current safety margins. Organizational ageing is the ageing of the organization's resource persons who may not be competent based on existing standards. Furthermore, apart from these categories of ageing offshore platforms are also susceptible to changes in the corrosive environment and the environmental loads over time (Ersdal et al. 2019), which also changes the original design loading profile and possibly the configuration of the installation.

A thorough understanding of these ageing categories is crucial in addressing the challenges of ageing, material degradation due to corrosion and fatigue. Corrosion is capable of reducing the material thickness, thereby making it susceptible to fatigue crack initiation and buckling, which may lead to failure of the structure (Adedipe et al., 2016, 2015). It is a major concern for offshore structures, as it changes the mechanical properties of the steel material with time (Garbatov et al., 2014). Uniform corrosion has been identified as the most common type of corrosion and can lead to the collapse of the structure due to deterioration of the stiffness of the structure (Popoola et al., 2013; Stacey et al., 2008). Localized corrosion, such as crevice and pitting, are usually restricted to small sections of the structural material and can lead to local stress concentrations (Adasooriya and Siriwardane 2014; Aeran et al., 2019).

On the other hand, fatigue is a major cause of failure in offshore structures. Depending on the amplitude of fatigue loads could lead to fatigue cracking which may impact the structural reliability of the platform (Stacey and Sharp 2007). Large fatigue amplitude in combination with a large number of load cycles as a result of the combined actions of wind, wave and operational loads, fatigue performance of welded connections is a design-driving criterion for OWT support structures (Dong et al., 2012).

Corrosion fatigue is a phenomenon when cyclic fatigue loading in a corrosive environment could significantly impact the fatigue strength by as much as 60%, depending on the type and rate of corrosion (Zhang and Yuan, 2014). Generally, a corrosion protection system (CPS) is usually provided to counteract corrosion as part of the offshore structure design. However, it has a typical life of 5 – 15 years (Dong et al., 2012), and it is ineffective in the splash zones owing to intermittent actions of current and wave (Det Norske Veritas, 2014). Besides, maintenance CPS in fixed offshore platforms fixed offshore structures can undergo localized corrosion even before the CPS becomes ineffective (Aeran et al., 2019; Qin and Cui, 2003).

These ageing mechanisms, if not monitored, can result in colossal damage to the asset, environment and reputation of the organizations involved. Although there are several ongoing research on ageing offshore platforms, with learnings taking from the oil and gas industry as the offshore renewable energy industry is still in its developmental stages. So there has been no record of any incident. However, based on the incidences recorded in the offshore oil and gas industry, it is expedient for the offshore renewable energy sector to begin to consider the possibility of incidence, especially for ageing structures, as some structures are attaining their design life. The imminent deployment of more structures in offshore, ageing and life extension should be factored in the whole life cycle of any offshore project.

In assessing corrosion fatigue of an ageing OWT structure, some uncertainties are involved. These uncertainties can be due to the stochastic nature of the design parameters such as wave and wind, scatter in selecting the right S-N curve for corroded detail, employing empirical relationships in the determination of stress factors, material dispersion. Uncertainties in conjunction with poor Inspection Maintenance and Repair (IMR) reports could further increase the probability of failure (Moan, 2005). Therefore, the need for detailed design review using appropriate standardized guidelines and a thorough knowledge of the complex ageing mechanism (Aeran et al., 2019).

There is presently a significant number of research work on the mechanism of life extension of offshore structures and some other ingenuities focused on developing standards and guidelines for the assessment of life extension of aged offshore structures. Emphasis has also been put on the development of risk-based inspection guidelines for the planning and execution of structural inspection DNVGL-RP-0001 (2015). Health Safety Executives (HSE) has launched several critical programmes on the United Kingdom Continental Shelf (UKCS), as well as a detailed investigation of all ageing offshore structures (HSE KP, 2012; Stacey, 2011).

Most of the early studies on the assessment guidelines and framework for ageing structures kicked off in the mid-1990s with the initial insertion of sections in API RP 2A (API, 2000), which was predominantly focused on oil and gas structures and the acceptance criteria and inputs are those of US waters. Afterwards, in 1996 similar sections were added to ISO 2394 (1996), which was followed by the release of ISO 13822 (2001) and ISO 19900 (2002) in 2001 and 2002, respectively. A more recent update of international standards was in 2007, which included an offshore structure assessment section (ISO 19902, 2007), which only had minimal information on quantitative analysis (Aeran et al., 2019).

The UK HSE in 2009 then proposed a structural integrity management framework for jacket structures, although it was mainly based on ISO and API guidelines (HSE, 2009), circa the same period the Petroleum Safety Authority (PSA) and the Norwegian Oil Industry Association (OLF) had introduced a project aimed at establishing rules and standards for the life extension of offshore assets in the Norwegian Continental Shelf (NCS) region (Selnes and Erdsal, 2011), the product of this project gave rise to NORSOK standards N-006 (NPRSPK, 2011).

The API in 2014 then issued the release of API-RP2SIM (Stacey, 2011), which was followed by the well-known DNV guidelines 2015. The DNV guidelines were based on the use of a probabilistic method for inspection planning of fatigue cracks (Selnes and Erdsal, 2011). It recommends modelling uncertainty variables (i.e., statistical, physical, measurement uncertainties) as random variables, each having a probability distribution function based on engineering expert elicitation (Det Norske Veritas, 2015).

Also, there have been several other research publications on the reliability assessment guidelines and frameworks besides guidelines that describe the reliability assessment procedure. Both the published literature and the standards discussed are all based on the generic procedure for the structural assessment of offshore structures. However, there are different types of offshore platforms, and each comes with its peculiarity, and as such, they need to develop a more concise guideline for each type of offshore structure. Developing specific guidelines for each type of offshore structure can be achieved by considering relevant models and theories that contains the effect of time-dependent variables that can best describe the degradation of the specific offshore structure. Such guidelines can correctly estimate the structural reliability of that structure detailing the remaining life and probability of failure and make suggestions of strengthening mechanism or life extension of the structure.

Available guidelines presently are also not sufficient to the extent that they cannot choose the precise fatigue strength curve or S-N curves, which are tools used to predict the time-dependent localized structural changes such as dents, erosion, cracks etc. Also, recent fatigue damage theories have not been added to the standards for a more precise estimation of the remaining life of the structure. To mitigate these limitations, the proposed framework in this thesis suggests guidelines for assessing the structural reliability of ageing offshore jacket support structure for possible life extension. The proposed framework provides recommendations on simulation of time-dependent structural deterioration, loading, soil-structure interaction, corrosion and optimized IMR planning. This framework combines guidelines from standards and theories to propose a more accurate guideline for estimating the reliability of the structure, including multi-criteria limit state assessment. Finally, the proposed framework shall be based on non-intrusive methods of assessing the reliability of the structure for possible life extension.

The models used to predict the reliability assessments are load and resistance parameters and the structural responses. Hypothetically, suppose additional factor(s) that address corrosion and the soil-structure-interaction are introduced into the models. There will be a significant improvement in the output of this assessment.

The motivation for this study is on the premise that most OWT support structures in operation are still well within their service life, consequently, in the next few years, most of the structures in operation would have outlived their design service year, and due to the tenacity of most structures, it makes economic sense to sustain its operation. Offshore support structures have also been used beyond their design service years, as witnessed in the oil and gas industry. However, the structures are different in dimensions but with the same design philosophy and operating environment. It is also known that most of the jacket structures deployed for offshore operation and outlived their initial design service life and undergone life extension. Given that the oil and gas industry is far more mature in terms of offshore operations, it is imperative that offshore renewable energy draw learning from the successes of the offshore oil and gas industry and enhance their operational efficiency and profitability.

How to approach the life extension of offshore wind structures is complex, given the limited experience in the offshore wind industry. Hence, structural reliability assessment for the life extension of ageing structures is an inherent challenge. However, existing guidelines such as NORSOK N-006, ISO 19902 etc., do provide a general process for assessment of ageing and life extension. This research aims to provide offshore wind-specific guidelines by including relevant concepts and notation such as the First Order Reliability Method and Response surface methods for the wider structural integrity assessment of offshore wind structures. This research considers analytical ageing assessment models from other offshore industries, such as the oil and gas industry, to provide a more robust ageing assessment and a life extension framework that can adequately account for the time-dependent degradation mechanism more accurately.

Hence, this research tends to review the trends and statuses of offshore structures as well as design standards, guidelines and theories of offshore structural reliability, focussing on the highly time-dependent stochastic parameters such as corrosion and fatigue on ageing structures.

However, the following research questions are pertinent and must be answered during this research, these questions are:

a) How long should a structure last?
b) What is the difference between design life, remaining life and minimum return period (MRP)?
c) What is the impact of uncertainty in determining the reliability of a structure?
d) For how long can the life of the OWT jacket support structure be extended?

1.2 Aim and Objectives

This research aims to develop a reliability assessment framework for the analysis of ageing fixed OWT, jacket support structures under a highly stochastic time-dependent parameter. This can be achieved by improving the existing reliability and ageing assessment methodology to include offshore wind-specific guidelines by incorporating relevant concepts and notation such as First Order Reliability Method, Response surface methods and parametric models, for the wider structural integrity assessment of offshore wind structures, capable of adequately account for the time-dependent degradation mechanisms more efficiently as mentioned above.

To achieve this aim, a non-intrusive multi-criteria limit state reliability assessment was performed, and major time-dependent variables that impact the reliability of the structure such as fatigue and corrosion were analysed, in line with guidelines from standards and theories to propose a more accurate guideline for estimating the reliability of ageing offshore jacket support structure.

The following objectives are set to achieve the aim;

1. A state-of-the-art review of OWT jacket support structure, considering various types of offshore structures, design philosophies, assessment methods, design analysis, assessment of ageing material and failure mechanism
2. Develop and validate a typical OWT jacket type support structure.
3. Develop and validate the proposed reliability assessment framework for assessment of the reliability of both new and ageing OWT jacket structure
4. Apply developed reliability assessment framework to the reference NREL 5MW OC4 OWT jacket support structure.

These objectives shall remain valid through this research; however, additional but more specific objectives were considered in the build-up to the computation of the structural reliability assessments of the OWT jacket support structure. This was deemed necessary as the aim was not just to postulate an improved structural reliability assessment method but also to validate the supposed improvements. The validation process shall be done at every stage of this research, structural modelling, and numerical analysis.

This study considers all reliability assessment stages, leading to the computation of the reliability index and calibration of partial safety factors.

1.3 Scope

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

It should be noted that although several studies were presented in the 80s and 90s discussing the application of structural reliability analysis concepts on oil & gas platforms. Much less work has been presented when applying such concepts in different applications such as offshore wind turbine support structures. Although this newer application has much to learn from a well-established industry, it requires additional research in advanced structural analysis methods due to the volume of manufacturing, marginal profit of assets and, of course, the highly stochastic nature of the environmental and operational loads. To this end, concepts and applications of structural reliability analysis should be further investigated as offshore wind farm operators and certification authorities are now requesting an assessment of the structures considering uncertainties more systematically than in existing design standards.

To further carry out this assessment, this thesis shall utilize a reference OWT, jacket support structure as modelled by National Renewable Energy Laboratory (NREL) (Jonkmann et al., 2015) as a case study. It will report the development and validation of an NREL 5 MW OWT model, development of the Stochastic Response Surface Method (SRSM), which allows the reliability assessment of the structure in the form of individual blocks. This approach treats the iterations in the simulation like a ‘black box’, which accounts for only the behaviour of the stimulus/response, unlike the complicated stochastic Finite Element analysis approach that requires deep knowledge mathematics to express uncertainties.

The results presented in the study are those obtained from the simulation, which presents maximum values at a given location, which is not always the case, as stress values vary across members of the structure. However, the main aim is to provide an algorithmic methodology that can be applied to individual members to determine their reliability.

For this study, ANSYS 18.2 was used to create an OWT modelled in line with the model description as given in the OC4 offshore project (Vorpahl et al., 2013), and validated based on results obtained from (Vorpahl et al., 2013), to ensure a proper representation of the structure and the environmental loads acting on the structure. The results obtained will be subjected to various sensitivity analyses and scenarios, to ascertain the versatility and applicability of the framework to different set-ups. This approach permits the use of specialized commercial tools for the probabilistic assessment of diverse engineering problems.

1.4 Areas of Contribution

The concept of structural reliability analysis has been published in several articles and journals. Most of these articles were mainly focused on oil and gas structures. Much less work has been presented for offshore wind turbine specific applications. However, the offshore wind industry is relatively new and has so much to learn from well established offshore industries such as the oil and gas industry. It requires more research advances in structural analysis methods due to the volume of manufacturing, profit margin, and the highly stochastic nature of the environmental and operational loads.

Therefore, this thesis's contribution shall be in developing an enhanced wind turbine specific structural reliability assessment method, which will be an extension of the conventional state of the art methods. The proposed method has proven to be more flexible (as it can be applied during the design, maintenance and even during life extension considerations) with a faster computational time, which could benefit academic and industrial applications.

1.5 Thesis structure

Chapter 2 shall provide a state-of-the-art review of the offshore jacket support structure, considering various types of offshore structures, design philosophies, assessment methods, design analysis, assessment of ageing material and failure mechanism

Chapter 3 will present the numerical methods of reliability analysis and proposed non-intrusive methodology of performing reliability assessment for ageing offshore structure and life extension

Chapter 4 will present the various methods of sampling and modelling offshore structures. Various load and capacity FEA models will be analysed. Uncertainties in load and material capacity modelling were considered.

Chapter 5 will demonstrate the development and application of the non-intrusive reliability assessment method of ageing offshore structure for life extension

Chapter 6 present the concept of inspection and mitigation plan for ageing offshore structures. The different types of inspection, inspection strategies and philosophy were presented. Inspection intervals and limitations were also reported

Chapter 7 is the discussion, which summarizes the most important aspects of this work. A recap of the objectives and design questions was carried out, and how they were all addressed was discussed, highlighting the pros and cons of the methods used.

Chapter 8 reports the conclusion and recommendations.

2 CONTEXT OF STRUCTURAL DESIGN, AGEING AND RELIABILITY ASSESSMENT OF FIXED OWT STRUCTURES

2.1 Introduction: Origin and State-of-the-Art of OWT

Electricity generation from wind turbine started in July 1887, by Prof James Blyth of Anderson’s College (the present-day University of Strathclyde in Glasgow), when he built a 10 metres cloth-sailed wind turbine to charge accumulators used to power his holiday cottage in Scotland (Uni. of Strathclyde, 2012). Blyth's wind turbine was welcomed with criticism and declared uneconomical. Nonetheless, by May 1895, he had improved turbine capacity, which now serves as emergency power to a rural Asylum, Hospital and Dispensary of Montrose for about 27 years (Uni. of Strathclyde, 2012). following Blyth's innovation, Charles Francis Brush built the first automated multiple-bladed wind turbine with a picket-fence rotor of 17 metres diameter to power his mansion in Cleveland (Ohio) (TelosNet, 2002). Current generated by the dynamo was used to energize batteries (Jeffrey, 1998).

Poul La Cour, in 1891, constructed the first electrical multiple bladed output wind turbine for electrification of the Askov Folk High School in Denmark (TelosNet, 2002). La Cour also developed the Kratostate regulator to address the challenges associated with producing a steady power supply, which was transformed into a prototype of an electrical power plant which was used to power a neighbourhood in Askov (Poula La Cour, 2008) as well as the first fast-rotating multi-bladed wind turbines and regarded as a pioneer in aerodynamics (The Guardian, 2008).

In 1922 Marcellus and Joseph Jacobs built their first wind turbine for their family’s Montana Ranch. After a few years of experimenting and design iterations of the turbine, they arrived at a detailed modified design in 1927, which became popular and gave rise to the first commercial wind turbine to be produced (Hoose, 2011). Two variants model was produced, rating 45 and 60 amps, which were efficient and reliable (Wind Charger, 2015). Georges Jean Marie Darrieus patented the first lift-based vertical axis wind turbine design in 1931 in the U.S. (Schelbergen, 2013).

Regardless of these efforts and those of other researchers, the global acceptance of wind turbines as a sustainable means of clean energy has suffered several setbacks due to the oil and gas industry trends. Post Second World War, mass wind energy deployment died out due to oil prices and resurfaced during the oil boom (US DOE). With the rising oil cost and future price forecast, nations abandon wind energy development. Still, following the environmental concerns associated with fossil fuels, several countries now turn to wind energy to reduce dependency on oil.

In recent times, the most widely used energy supply policy tool, Feed-in tariffs (FITs), has thrown its weight behind renewable energy development (Cork et al., 2009). The initial FIT was realized in 1978 in the U.S., the Public Utility Regulatory Policies Act in the National Energy Act (Cork et al., 2010). Thereby giving rise to the first wind farm in December 1980, at the shoulder of Crotched Mountain in southern New Hampshire (U.S.) (US DOE, 2011). It is comprised of 20, 30 kilowatts wind turbines (Wind Energy Centre, 2014). Unfortunately, the project was considered a failure due to the frequent breakdown of the system, which was traced to an overestimation of the wind resources by the developers (Guardian, 2008).

The ’90s witnessed a surge of increase in wind farms in northern Europe due to successes recorded in the region due to the high cost of electricity and shear wind availability, which led to the establishment of a firm market (Kaldellis and Zafirakis, 2011). Slowly the market migrated into the offshore environment in an attempt to exploit the abundance of wind available offshore (EWEA, 2009). This trend resulted in the installation of the foremost OWT of 220KW capacity at 250m water depth in Sweden in 1990 as a pilot project (Dalen, 2011) and the first OWT farm commissioned in Denmark in 1991, which comprised 11, 450KW wind turbines at a water depth of about 4m, 1.8km offshores

Europe has since then dominated the market with the installation of turbines in a more hash critical environment deploying the finest of technology and design of complex support structures. Wind turbine technology has since then metamorphosed into a viable source of alternative sustainable energy source.

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

2.2 Review of fixed OWT support structures

OWT support structures are critical to the installation of wind turbines. They are the foundation upon which the wind turbine is placed. In terms of capital expenditure, the foundation of OWT is more expensive than those of onshore wind turbines by about 350% (Morthorst and Kitzing, 2016). This is due to the challenges presented by the offshore environment, the difficult terrain and logistics. OWT support structures account for about 20 - 30% of the total estimated cost of a wind farm, which makes it the highest contributor (Gasch and Twele, 2011). Thus, the selection of the most appropriate support structure for each OWT deployment is of utmost importance to any OWT project.

There are various types of support structures, as presented in Figure 1, that can accommodate offshore wind turbines. The selection of the most suitable structure depends on several criteria, including the water depth, the estimated environmental loads, the cost of production and installation, the complexity of the design etc. (Lozano et al., 2016). The monopile support structure is currently employed in most existing projects in Europe due to its simple but robust design are sited at the continental shelf, about 6 miles of the coast in shallow waters (Xiaoni et al., 2019; Gentils et al., 2017), it becomes however uneconomical for offshore projects in deeper waters, yielding for a requirement for more complex structures such as the jackets which becomes more suitable.

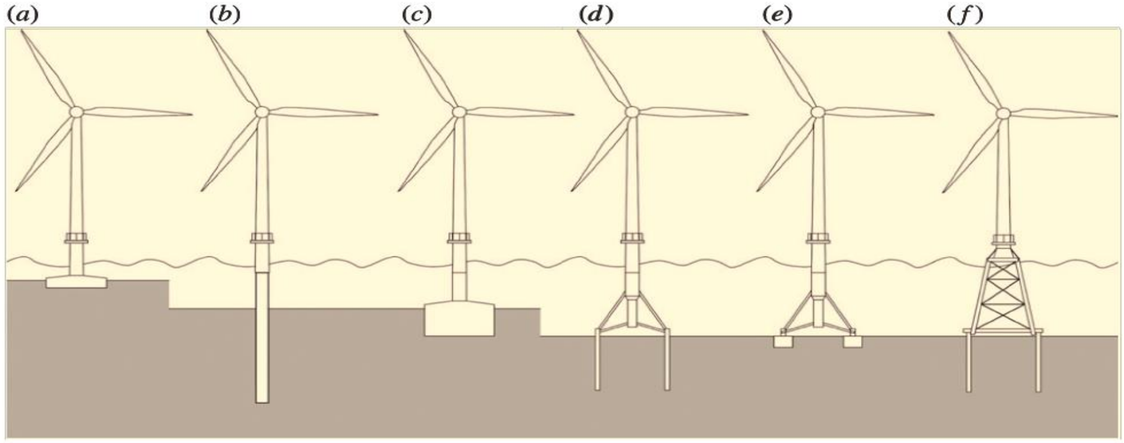

Figure 1:Support Structures in existence. (Gentils et al., 2017); (a) gravity-based foundation (b) monopole foundation (c) caisson foundation, (d) multiple foundation, (e) multi-caisson foundation (f) jacket foundation.

2.2.1 Gravity base support structure

The gravity base support OWT structures are designed based on the self-weight of the turbine and reinforced with a concrete base. The weight of the structure must be able to resist extreme overturning moments, thereby ensuring the structure always stands upright. Figure 2 show a typical gravity-based OWT support structure. The gravity-based support structure was mainly deployed in the early stages of OWT development. Its design is expected to have an adequate load-bearing capacity to withstand both the environmental and service loads. Gravity base OWT support structure is easy to construct and most appropriate for sandy soil, rock, and compact clay soil. Gravity based support structures are usually deployed in shallow waters.

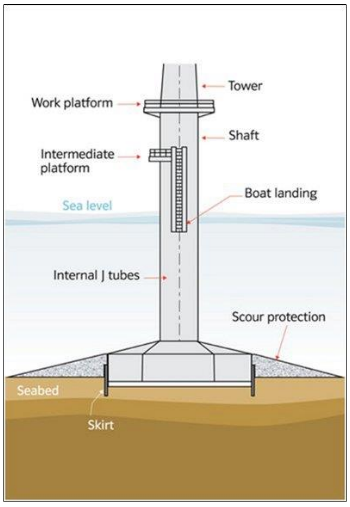

Figure 2: Schematic presentation of a Gravity Base Support Structure.

(Source: https://www.4coffshore.com/windfarms/tripod-support-structures-aid7.html) available 27/08/20.

2.2.2 **Monopile support structure**

Monopile support structures are made of steel tubes typically 4 – 8m in diameter and are mainly used for shallow to medium water depths, i.e., 0 to 30m. They are relatively easy to design by which the tower is supported by the monopile, either through a transition piece or directly. Most of the support structures used in most wind farm projects in Europe and the UK are monopiles (Xiaoni et al., 2019). Monopile are usually driven into the seabed using an impact hammer and drilling in case of a hard formation. Figure 3 shows a schematic of a typical monopile support structure. Analytical results from installed monopile structure show greater stiffness than what was predicted by design standards (Kallehave et al., 2012). Therefore, the need for a more accurate guideline to reduce over design and total mass of the structure (Byrne et al., 2017).

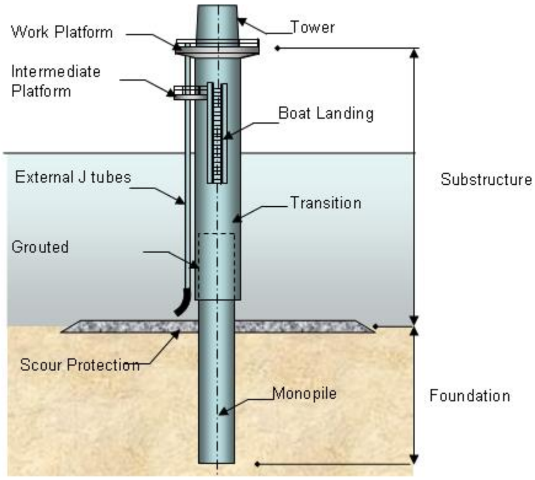

Figure 3: Schematics of a Monopile support structure

(Source: http://www.wind-energy-the-facts.org/offshore-support-structures-5.html) available 27/08/20.

2.2.3 **Tripod support structure**

Tripod structures are considered relatively lightweight, 3-leg steel jacket structures in comparison to the standard jacket structure. Depending on the application, they are made of welded steel tubes typically 1.0 to 5.0m in diameter. It is anchored on a steel transition piece just below the turbine, which is coupled to the three steel piles. The piles are generally about 0.8 – 2.5m in diameter and are driven about 10-20m into the seabed. Figure 4 presents a schematic of a tripod support structure. The Tripod support structure is suitable for deployment in water depths of about 10 - 35m and can bear the tower loads and transfer moments and stresses to the three steel piles. Examples of tripods support structure deployment are Nogersund, Sweden and Alpha Ventus, Germany (Xiaoni et al., 2019).

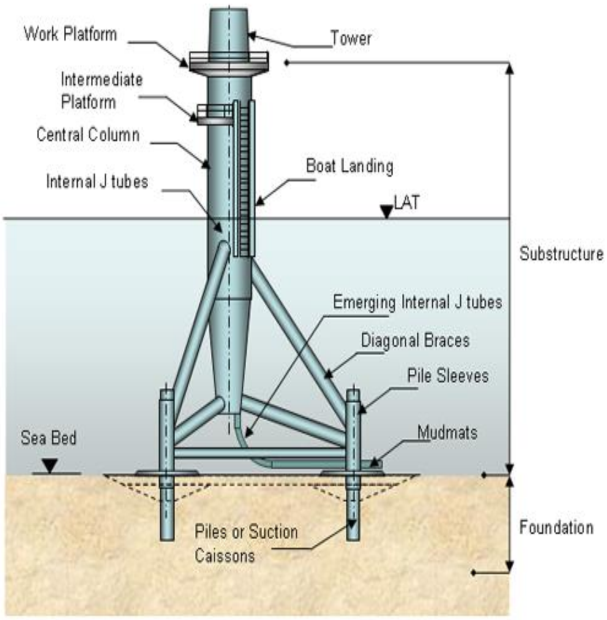

Figure 4: Schematic presentation of the Tripod Support Structure.

(Source: http://www.wind-energy-the-facts.org/offshore-support-structures-5.html) available 27/08/20.

2.2.4 **Jacket support structure**

The jacket support structure is made of cylindrical hollow steel tubes of 0.5 – 1.5m in diameter (depends on the choice of the material) that are welded to form a frame. A bolted transition piece connects the frame to 0.8 – 2.5m corner piles driven into the seabed or by a template. The jacket support structure, as mentioned before, jackets are suitable for deep water (up to 50m) deployments (Det Norske Veritas, 2014). The jacket structure has several variants of 3-leg or 4-leg and is held in place by linear and diagonal braces with smaller diameters and thickness. Figure 5 presents a schematic of a jacket support structure. Examples of jacket support structure deployments are Alpha Ventus, Germany in 2010, Beatrice and Ormonde, UK in 2006 and 2012, respectively and Bohai sea, China (Xiaoni et al., 2019).

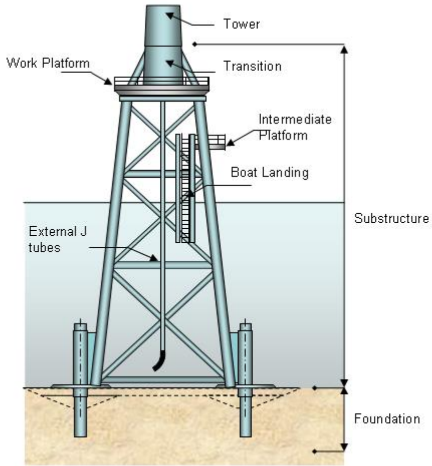

Figure 5: **Schematic presentation of a Jacket Support Structure.**

(Source: http://www.wind-energy-the-facts.org/offshore-support-structures-5.html) available 27/08/20.

2.3 Design Philosophies of Offshore Structures

There have been notable improvements in the design procedures for offshore support structures, which can be credited to a better understanding and estimation of the environmental loads, structural behaviour under different loading regimes while in service, availability of advanced weather forecasting equipment, sophisticated computer-aided applications for modelling and simulations, and other specialized programmes aimed at simplifying the design process. Classification of principles of design of offshore structures is based on stresses and uncertainties that were taking into account. Some of the widely used design principles are permissible stress, global and partial safety factor principles and recently the probabilistic and limit state design principles.

2.3.1 **Permissible stress principle**

Permissible stress is one of the most acceptable structural design methods of systematic designs. It is governed by the linear elasticity theory and is also referred to as the allowable stress method. Its criteria are based on the satisfaction of the equation below:

$$\sigma_{max} < \sigma_{allowable} \quad or \quad \sigma_{max} < \frac{\sigma_{crit}}{SF} \tag{2.1}$$

where σ is the stress, SF is the safety factor, which is a measure that accounts for all design uncertainties. Equation 2.1 implies that the allowable stress (critical stress divided by the safety factor) must be greater than the maximum stress. Simply put, the capacity of the structure should be greater than the maximum stress, which is the load acting on the structure. The general workings of this structural design approach do not include stress distribution, non-linear performance functions or ductility of the material (Holicky and Vrouwenvelder, 2005 as cited by Kolios, 2010). It is also applied locally and to sections of the structure that are most likely to experience max stress.

However, this approach is unable to consider all uncertainties from the model and variables or cases of combined loads. The outcome of this approach is regarded as inconsistent design-wise because the outcome due to changes in the parameter is not reliable. As such, it is generally seen as been conservative design.

2.3.2 **Global safety factor**

The global safety design method leverages the relationship between the mean values of loads effects $\boldsymbol{E}$ and the resistance of the structure $\boldsymbol{R}$. The global safety is thus determined based on the ratio of these quantities, which is given as:

$$S = \frac{R}{E} > S_o \tag{2.2}$$

where, S_o Is the target minimum safety factor set by the engineer, and S is the global safety factor. Unlike the permissible stress method, this method accounts for the responses of cross-sectional members of the structure, the stress distribution, and the non-linearity of the structural geometry. Although the safety factor and the resistances are determined scientifically, this method does not consider uncertainties in its modelling. Thus, it is also regarded as conservative. Therefore, it is not capable of performing analysis for the use of different material or load combinations

2.3.3 **Partial Safety Factor**

The partial safety factor design method is seemingly the most recent and robust method. It is also known as the "Limit State Method" because it is applied similarly to limit state design for various design conditions. This method is most advanced because it allows for mathematical optimization to suit various design conditions. It can be represented mathematically as:

$$E_d(F_d, f_d, a_d, \theta_d) < R_d(F_d, f_d, a_d, \theta_d) \quad (2.3)$$

$$where \quad F_d = \psi \cdot F_k \cdot \gamma_F \quad (2.3a)$$

$$and \quad f_d = \frac{f_k}{\gamma_m} \quad (2.3b)$$

where, E_d and R_d are the load effects and structural resistance, respectively, F_d and f_d are the design parameters defining the responses to the load and material properties, respectively, a_d and θ_d are the measures of uncertainties in the geometry and model, respectively, while $(F_k, f_k, a_k, \theta_k)$ Are the design values corresponding to the characteristic values of the parameter, by applying the required partial factor γ, the reduction factor ψ and any other standard factor considered in the reliability design. The partial safety factor method is an advancement of the global safety factor method. It is analytical with fewer assumptions. It is also capable of analysing load combinations and designs involving different materials.

2.3.4 **Probabilistic design principle**

The basic philosophy of the probabilistic design method is to ensure that the probability of failure of the structure is at no time during the service life of the structure below a target set value. Mathematically this can be represented as shown in equation (2.4).

$$P_f \leq P_d \quad (2.4)$$

where P_f and P_d are the target probability of failure and the design probability, respectively. This method can accomplish optimized design resulting in relatively lighter, cheaper, and efficient structures. Furthermore, the probabilistic design method can optimize the partial safety factor design (Gulvanesian and Holicky, 2005), which is critical for the design of novel structures where no reference experience or methodologies is available. The drawback of this method, which has resulted in fewer cases of use, is its complexities.

The design methods get more complicated as they require huge mathematical and engineering computation and manipulation skills, following the constant evolution of the design philosophy. Nevertheless, this procedure's resultant design output has seen significant optimization of the weight of the structure and the product of a more efficient structure.

Despite the probabilistic method, most offshore structure design guidelines by a standards body such as API RP-2A-WSD (Working Stress Design) (API, 2000), the probability of failures have been presented in the Partial safety factor format. API RP-2a-LRFD (Load Resistance Factor Design) aligns with the partial safety design method, allowing the engineer to develop an optimized design.

2.3.5 **Limit State Design**

Modern structural design trends follow a limit state design approach, which aims to derive designs with adequate safety margins to take account of uncertainties that could adversely affect the reliability of the structure. The structure is required to be checked for all categories of limit states to ensure adequate safety margins between the maximum likely loads and minimum resistance of the structure (Gulvanesian and Holicky, 2005). The limit state design criteria, in summary, the load of a structural system must at no time exceed the capacity of the structure. This can be expressed mathematically for multiple loading as:

$$D_d = \gamma_0 \sum_i D_{ki}(F_{ki}, \gamma_{ki}) < C_d = \frac{C_k}{\gamma_M} \tag{2.5}$$

In equation (2.5), the subscript k represents the characteristics value of the load variable, while d is the design value that integrates the required amplification or reduction to account for consideration of uncertainties. Load variables are amplified by a factor γ_{ki} to account for unanticipated occurrence, while the structural resistance (capacity) is diminished by the material factor of γ_M To account for uncertainties rising from material properties, corrosion etc. Furthermore, a partial safety factor of γ_0 is included to account for the significance of the examined limit state to the reliability of the structure.

Characteristically, the definition of variables is based on their statistical properties. The structural capacity variable is based on a lower bound of 95% exceedance value. In comparison, a load variable is based on an upper bound or a 5% exceedance value. The derivation of partial safety factors is based on the rigorous procedure or expert elicitation that provides acceptable safety levels and performance. Unlike in the case of the allowable stress methodology, where the concept was based on ensuring the response of the structure to the load acting on it remains below a given threshold. The Limit state design concept systematically examines the structure's response under various scenarios it might confront during its service life.

Regulatory bodies and classification societies have proposed several limit states for marine structures that should be considered for a comprehensive design. DNV-OS-C101 (2008) has proposed four primary limit states, namely; Accidental limit state (ALS), Fatigue limit state (FLS), Serviceability limit state (SLS) and Ultimate limit state (ULS), that should be considered for a comprehensive marine design.

2.3.5.1 **Serviceability limit state (SLS)**

SLS refers to service conditions were owing to extensive vibration or noise, a deformation that can influence the structure’s functionality. The established criteria are based on practice experience of the functionality of the structure expressed in the form of maximum allowable deflection (EURO CODE 3, 2005) or similar design constraints required for the structure to operate without intervention. Buckling is also considered in SLS to monitor the behaviour of the structure and prevent cases of excess deflection.

2.3.5.2 **Ultimate Limit State (ULS)**

ULS is one of the most critical design constraints in structural design. It monitors the structural behaviour of members to resist plastic deformation or attain their ultimate strength. For analysis of a structural member, the post-buckling behaviour should be considered to avoid conservatism on the strength of the structural member. Figure 6 shows this a more traditional method where the point of elastic buckling determines the strength of the element at point (A), compared to this method that measures the strength of the structural member at point (B).

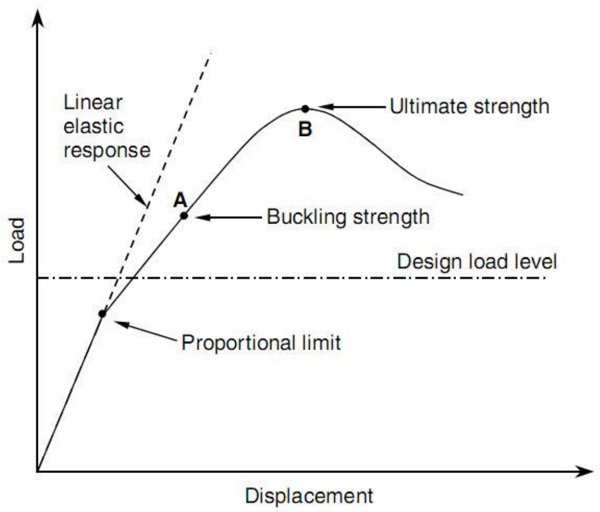

Figure 6: Design considerations for ULS (Kolios, 2010)

Modern design trend tends to model structures based on ultimate strength. For existing structures that may have undergone any type of deformation, a new ultimate strength would be required based on the damage and the new capacity of the structure or member also accounting for the probability of failure. It is best practice to design structures to fail in a ductile manner rather than brittle failure. This implies that rather than failing suddenly and offering no opportunity for intervention, as in the case of the latter, the structure should fail progressively. By redistribution of stresses into alternative load paths, therefore, offers an opportunity for intervention.

2.3.5.3 **Fatigue limit State (FLS)**

FLS is a crucial structural design factor for consideration in conjunction with the ultimate limit state. Offshore structures are exposed to continuous cyclic loading in their whole life cycle. Fatigue impacts are mainly local, usually more severe at the welded joints and areas of stress concentrations.

Therefore, design criteria for fatigue limit state account for the accumulation of fatigue load on a structure under cyclic loading. Several factors can contribute to the effects of fatigue damage at the onset of crack initiation, such as the local stress concentration characteristics, the number of stress range cycles and the stress ranges encountered during the load cycles. An in-depth analysis of the fatigue limit state will be provided in chapter 4.

2.3.5.4 **Accidental limit state (ALS)**

ALS is a design criterion that accounts for the severity of damage to the structure in the event of structural system failure. This limit state tends to forecast by way of a risk assessment the impact of a system failure. In this case, OWT would have on the environment, the asset, fatality etc., this criterion is satisfied based on different accidental scenarios and corresponding structural responses that should be decided following a comprehensive risk assessment. However, there should be a trade-off between the ultimate safety and prevention cost for an optimal economic design, thereby avoiding over design and setting a realistic survivability consequence.

In the design of the Accidental limit state, the structural integrity is assessed at a global level initially, and then the post-accident analysis to account for the impact of the analysis on members. For instance, a jacket structure requires to be able to maintain its stability post-impact, and so the remaining capacity can be evaluated to ensure it is functional post-impact.

2.4 Design Standards for Offshore Steel Structure

Ancient structural development methodologies were based on experimental, empirical and theoretical knowledge of probabilistic and structural mechanics concepts. The structured documentation of this knowledge under a scientific basis can stem from a methodology that permits the design of specific structures at different reliability levels. The documentations of these methodologies are the basis upon which the structural design codes and standards are formulated. This section shall review the various classification of standards as well as a detailed review of standards that pertain to offshore structures.

Moan (2005), the fundamental principles of modern offshore guidelines and standards, summarized that modern structural design criteria are based on the limit states. Limit states such as fatigue and ultimate strength are analysed based on the consequences of failure and reliability analysis respectively, while accidental collapse design conditions were based on damage tolerance. The loads considered were mainly wind, wave, current, ice, earthquake as well as other accidental loads. The analysis performed is mainly global structural analysis by finite element methods as well as nonlinear analysis aimed at demonstrating the damage tolerance because of planning inspection and progressive failure.

2.4.1 Design Standards Categorization

Structural design standards and codes are distinguished by either limit state design or allowable stress design concepts. The limit state codes structural design concept designs the structure to resist specific loading conditions as described in each corresponding limit state. For a combined load and resistance analysis, the loads are multiplied by a partial safety factor while the resistance is divided by a safety factor. Safety factor values are usually in the order of magnitude of 1.5 and 2.5 – 3.0 for yield and ultimate strength, respectively (Burdekin, 2006).

The allowable (working) stress codes criteria ensure that the maximum load on the structure exceeds the yield strength when divided by the safety factor. Application of these standards can be very complicated and requires several aspects of decision making guided by expert elicitation.

2.4.2 Offshore Structure Integrity Design Standards

2.4.2.1 ISO 19902:2002: Petroleum and Natural gas industries-general requirements for offshore structures

ISO 19902 (2002) provides critical guardians for the design and fabrication of offshore fixed steel structures. The standards were based on known fixed offshore structures through direct input from several countries engaged actively in the offshore industry, including the United Kingdom, France, Canada, Norway, the United States etc. ISO 19902 is, therefore, regarded as a culmination of efforts of many years in the offshore industry. It was hitherto adopted as the national standards and the legal and prerequisite requirement for certification for offshore structures by participating countries. ISO has since then been acclaimed as a global standards certification body.

2.4.2.2 BS EN 1993-1-1:2005 EUROCODE 3: Design of Steel Structure

The primary purpose for Eurocode was to establish an Act for creating harmonized technical specifications for the design, construction, and installation of engineering projects under the European Union law. This aims to provide a means to prove compliance for mechanical strength, safety, and stability requirements for the design of steel structures in case of an accident. EUROCODE is based on National experience and research, therefore, presenting a world-class structural design standard. EUROCODE, verification procedures are based on a limit state, and it allows for the application of probabilistic based design method (Eurocode, 2008).

By March 2010, the Eurocodes became mandatory for the specification of European public works and are intended to become the accepted standard for the private sector (Eurocode, 2010). The Eurocodes, therefore, replace the existing national building codes and standards by national standard organizations of all member countries. However, many countries had a period of co-existence of the standards. Furthermore, each member country was required to issue a National Annex to the Eurocodes, requiring referencing for a nation (e.g. The UK National Annex). At present, the take-up of Eurocodes is slow on private sector projects, and existing national codes are still widely used by engineers.

2.4.2.3 **ANSI/AISC 360-05 Design Specification for steel structure buildings**

ANSI/AISC design *Specification* provides the generally applicable requirements for designing and constructing structural steel buildings and other structures. The 2016 edition of the AISC *Specification* and Commentary supersedes and is an update of the 2010 edition. Both LRFD and ASD methods of design are incorporated in a single document.

This Specification is based upon past successful usage, advances in the state of knowledge, and changes in design practice. The 2016 American Institute of Steel Construction's Specification for Structural Steel Buildings provides an integrated treatment of allowable strength design (ASD) and load and resistance factor design (LRFD). It replaces earlier Specifications (AISC, 2016). This ANSI-approved Specification has been developed as a consensus document using ANSI-accredited procedures to provide a uniform practice in the design of steel-framed buildings and other structures. The intention is to provide design criteria for routine use and not to provide specific criteria for infrequently encountered problems, which occur in the full range of structural design.

2.4.2.4 **Other offshore structural design standards**

The deployment of offshore structures by the oil and gas industry and marine/naval engineering operations have contributed immensely to the strategic improvement of the safety and reliability of offshore structures. In addition to the standards that have already been mentioned too, Det Norske Veritas published "DNV: Rules for the classification of installation" in 1989 (Det Norske Veritas, 1989), and in 2008 the "offshore Standards OS-C101: Design of offshore steel structures, General - LRFD Method" (Det Norske Veritas, 2008), while the Norwegian petroleum industry has introduced the NORSOK standards, which mainly refers to ISO, EN 1993 and is currently on its 30th edition (NORSOK, 2015). Finally, Germanischer Lloyd's in 1998 and 2007 published "guidelines IV – Industrial Services: Offshore Technology" (Lloyds Register, 2007)

2.4.2.5 **Comments of offshore structural standards**

Generally, the use of design standards as guidelines to evaluate the reliability in structural design with acceptable reliability have been deployed for quite a lot of application. However, there are some drawbacks, such as the application of standards on a special or novel structure. Since standards are tailored to specific structures and are presented at a high level, providing limited detailed information and guidance based on the approach followed by (Kolios and Brennan, 2009). The concept of reliability-based design becomes very useful for such scenarios.

Furthermore, for cases of high design uncertainty, the design detail is followed in a manner where the consequence of failure is reduced to the barest minimum. This can be achieved by a proper combination of different standards, capable of providing a reliable design solution that meets the minimum safety requirements.

Following the successful deployment of offshore structures, there are statutory planned routine inspections, services, and maintenance where applicable. And in the course of these safety elements are introduced, thereby reducing the overall uncertainty, and additional real-time information of the structure are gathered and reliability calculated and compared to the initial design calculation. Results obtained from such processes can provide a periodic reliability trend throughout the life cycle of the structure. Therefore, reliability can also be calculated based on the actual conditions of the structure and subsequent intervention scheduled

2.4.3 **Offshore Structure Life Extension Standards**

2.4.3.1 **Ageing and Life Extension Regulations in the UK**

There are specific regulatory, structural integrity management (SIM) requirements for operating offshore structures in the UK Continental Shelf (UKCS), such as:

- The Design and Construction Regulations (DCRs) (HSE 1996) require the duty holder to ensure suitable plans are in place for maintaining the structure's integrity throughout its service by periodically conducting IMR. This regulation highlights the subject of Safety Critical Elements (SCEs), which primarily is the identification of some element in the structure that their failure could result in substantial damage

- The Safety Case Regulation (HSE 2015) mandates the preparation of a safety case as a formal document. This regulation explicitly provides guidelines for the first-time life extension of a structure beyond its design life, which includes a reverse safety case that considers all changes to the initial safety case. Beyond the SCE requirement by the DCR, the HSE also introduced Safety and Environmental Critical Elements (SECEs), which considers the impact of a major accident hazard and the elements that their failure could result in significant environmental damage. This guideline was introduced following the European Directive (EU 2013)

So, for all installations in the UKCS, proposing to extend the service life are required to undergo a verification process by an Independent Competent Person (ICP), who reviews the SCEs and SECEs provided. The verification exercise may be reviewed periodically or even revised or replaced in consultation with the ICP. The review usually focuses on the performance standards. Critical areas for life extension would include availability, reliability, and survivability, all of which could be imparted by ageing phenomena driven by fatigue and corrosion.

Auditing is also a vital aspect of the HSE (2015) regulation. Despite the developing state of the industry's performance standards, especially the renewable energy sector, inspection and maintenance have been identified as very important in ensuring the SIM. Sharp et al. (2015) have presented some key performance indicators (KPI) for offshore structural integrity, intending to achieve good practice, which is a requirement in the ALARP approach (HSE 2015).

2.4.3.2 **Ageing and Life Extension Regulations Elsewhere Globally**

The exploration of offshore in search for energy continues to go offshore, several parts of the world, which has resulted in the development of national regulations, of which many of those are an adaptation of on exiting regulation. In Canada, the National Energy Board, the Newfoundland and Labrador Offshore Petroleum Board and the Nova Scotia Offshore Petroleum Board are the authorities that regulate the energy sector. Offshore safety is mainly a combination of goal setting and prescriptive methods, and in most cases, fitness certifications are obtained from any accredited certification authorities.

In Australia, the National Offshore Petroleum Safety and Environmental Management Authority (NOPSEMA) is the apex body regulating safety and environmental protection. The safety regulations are like those in the UK. With safety cases and hazard management as the prerequisite for safety. in other parts of the world, assessment of existing structures is based on ISO standards, mainly the ISO (2007) and ISO (2017), and for renewable energy such as OWT structure DNVGL-ST-0262 (2016), DNVGL-ST-0126 and DNV-OS- J101 (2014) besides UK regulations and API standards are also used depending on the structure.

2.5 Ageing and Material Degradation of Offshore Structures

2.5.1 Ageing

OWT support structures experience changes daily from the onset of fabrication to installation and operations. Some of these changes are due to the choice of material used, the impact of their operating environment, unforeseen events such as hurricanes, tsunami ship impact etc. and if it was possible to replace or repair damaged or deteriorated members of the structure. Other changes such as fatigue and corrosion will become noticeable soon as the material degrades. Ersdal et al. (2019) reported and described in detail the four types of changes highlighting their nature and mechanism. This thesis would, however, focus on the physical changes that border on the structure of the system (OWT jacket support structure), the configuration hazards, loading and the integrity of the structure.

2.5.2 Structural Degradation

Structural material degradation/deterioration, as used interchangeably in this thesis, structural material degradation/deterioration is a process by which structural material begins to damage or progressively worsen. This phenomenon occurs over time in the life of a structure mainly results in the failure of the structure. The choice of engineering material at the design phase of any structure is critical, as this need to consider several factors such as fatigue, corrosion resistance etc. although the properties of materials used for new offshore deployment are better understood (HSE, 2013)

Unlike recently designed structures, ageing structures were designed with significant gaps in the knowledge of the detailed thickness, the grouting, reliability, and fatigue performance of critical sections of the structure, especially for jacket type structure (Ersdal et al., 2019). Several deterioration mechanisms are associated with ageing, such as wear, erosion, corrosion, and fatigue. However, fatigue is a significant source of hazard for offshore structures. In some instances, fatigue cracks reduce the structure's overall structural reliability and have caused the failure, such as in the case of Alexandre Kielland (Stacey and Sharp, 2007). Therefore, this thesis shall primarily focus on the overall impact of corrosion and fatigue on the jacket structure.

OWT jacket support structure is mainly built with steel materials. When the stresses in an area or a member exceed the elastic limit over and beyond the local plastic deformation of the member or area, a plastic stress-strain behaviour will occur. Repeated cycles of this kind of stress can result in softening or hardening of the material. Cyclic softening could lead to an increase in the strain range and cause a fracture, whilst the cyclic hardening can reduce the peak strain of the material and increase the resistance to static failure. Repeated environmental loading could cause plastic cycling of the material in the plastically deformed members or areas. Plastic deformation under cyclic loading can result in the nucleation of fatigue cracks which is the incipient form of crack initiation and subsequent propagation and failure (DNVGL, 2016b).

Other factors that can influence material degradation are hydrogen embrittlement (HE) and Hydrogen induced stress cracking (HISC). These phenomena are typically due to the migration of hydrogen atoms into the structural material, which then recombines in the voids of the material and form molecules, which in turn cause pressure within the material matrix and as such reduces the tensile strength of the material, and a resultant crack (Ersdal et al., 2019). The main source of hydrogen in an offshore environment is from cathodic protections, and HE susceptibility increases with the increasing strength of the material.

Therefore, their historic cathodic protection data needs to be considered for ageing structures with high steel grades. HSE (1991) presented a comprehensive study of the HE mechanism in offshore structures, while HSE (1995) section 33 provides guidance for assessing the vulnerability to HE in steel materials.

2.5.2.1 **Capacity Assessment of Ageing Offshore structures**

The capacity of a structure is the ability of the structure to withstand applied loads and loads effects such as stresses without failing or exceeding any defined limit state. Strength analysis of ageing offshore structures with possibly degraded and damaged structural elements is a developing challenge in the offshore renewable energy industry, with more support structure reaching their design life. Assessment of the structural capacity is a critical aspect of life extension. It can be helpful in identifying any form of damage or degradation of the defects that could reduce the structure's load capacity if the current loads exceed the initial design estimates or changes in the design codes or standards.

Several degradations or damage effects may be imposed on an offshore structure, which could all affect the strength of the structure, such as dents, deformation, corrosion, cracks, holes, missing members, wear, embrittlement, hardening etc. The effect of these damage mechanisms can be included in the strength assessment of the structure by taking into account the four major factors: metal loss or wall thinning, cracking, change in material property, and change in the shape of the structure. Ersdal et al. (2019) detailed the various ageing effects and their effect on the structure

Saad-Elden et al. (2012) suggest that the most common form of damage and degradation of an offshore structure is corrosion, fatigue cracks, dents, wear and buckling. The study also opined that the limit of elasticity in corroded samples was reduced, which implies the structure becomes more brittle with increasing corrosion.

Adedipe et al. (2016) identified wall thinning as one of the effects of corrosion on offshore structures. Wall thinning or metal loss results in the reduction of the cross-section and properties of the members of the support structure, such as the moment of inertia (Ersdal et al., 2019). At the same time, cracking, which is typically due to extensive damage or borehole, can result in the reduction of sections and the geometry of the members. Other degradation modes such as material hardening and embrittlement can result in material properties and geometry changes.

2.5.2.2 **Damage Quantification of Offshore Steel Structure**

Several reports have been published on the quantification of damaged members of an ageing offshore structure, mainly dented tubular braces (Smilth et al. 1979; Smilth et al. 1981; Smilth et al. 1986; Ellinas and Walker 1983; Yao et al. 1986; Landet and Lotsberg 1992; Moan 1987; Moan 1985; Taby and Moan, 1887). These research publications have based their conclusion on the requirements on design standards such as API RP 2A (API 2014), NORSOK N-004 (Standard Norge 2013) and ISO 19902 (ISO 2007), which provided guidance for the calculation of the strength of a degraded structural tubular member usually in a jacket structure, while other offshore structures such as semi-submersibles, jack-ups and ship-shaped structures are calculated based on DNVGL-CG-0172 (DNVGL 2015b). Based on these standards, the effect of the change in geometry due to bowing and the dent was considered, which also includes out of straightness which is typical of every dent. However, cracks and corrosion are considered as dents. The equivalent dent due to crack can be calculated according to NORSOK N-004 1998, which is given as:

$$\delta = \frac{1}{2}\left(1 - \cos\left(\pi \,.\frac{A_{crack}}{A_0}\right)\right).D \tag{2.6}$$

$$\delta = \frac{1}{2}\left(1 - \cos\left(\pi \,.\frac{A_{corr}}{A_0}\right)\right).D \tag{2.7}$$

where δ is the equivalent dent due to crack, $A_{corr}$is the area of the corroded part of the cross-section, A_{crack} is the area of the cracked segment of the cross-section, A_0 is the cross-sectional area of the undegraded member, D is the outer diameter of the undegraded member.

2.6 Fatigue Analysis

Fatigue is the cumulative material damage due to several loading cycles during a structure's service life, which could lead to crack initiation and subsequent propagation. Areas of the structure with high stresses discontinuities and defects such as welded joints are most susceptible to fatigue failure. Fatigue cracks are time-dependent and accumulative degradation mechanisms. Consequently, fatigue cracks are usually prominent in the latter part of the service life of the structure. Nevertheless, there are instances where fatigue cracks have been observed early in the life of a structure. These sorts of cases are due to unattended defects from construction and fabrication phases. A typical example is the fatigue failure of the Alexandre Kielland platform, which eventually resulted in the capsize of the platform in 1980.

Depending on the scale of the fatigue damage, there are implications on the integrity of the structural capacity, such as a high possibility of brittle or ductile fracture, increased crack growth rate, water ingress into members, and ultimately reduce the strength of the structure. Mohaupt et al. (1987) and Stacey et al. (1996), as cited in Ersdal et al. (2019), in their findings, reported that fatigue damage could result in the reduction of the remaining fatigue life as well as reduce the static strength by 40%. A thorough understanding of the consequence of fatigue damage is essential for managing risk in an ageing offshore structure.

There are mainly two methods of performing fatigue analysis for the life assessment of a structure under stochastic cyclic loading before failure; the S-N and the Fracture Mechanics (FM) approach, which is very important in the design and life extension of ageing offshore structures. The S-N approach, which is primarily based on the S-N curves generated from experimental data, is widely applied for design. The FM approach is used to estimate the fatigue life based on the propagation of an initial defect or defects observed during an inspection of the structure and limiting flaw sizes and plan for IMR strategies.

2.6.1 **S-N Fatigue Analysis**

The S-N method is the traditional method of assessing fatigue life, based on S-N curves and long-term stress range distribution or spectrum providing the number of fatigue cycles (N) and its corresponding stress range (S). Figure 7 illustrates a general form of the S-N curve for a tubular joint in air and seawater with cathodic protection. In recent decades, a significant amount of effort has gone into generating the S-N curve in general and for specific cases like offshore structures.

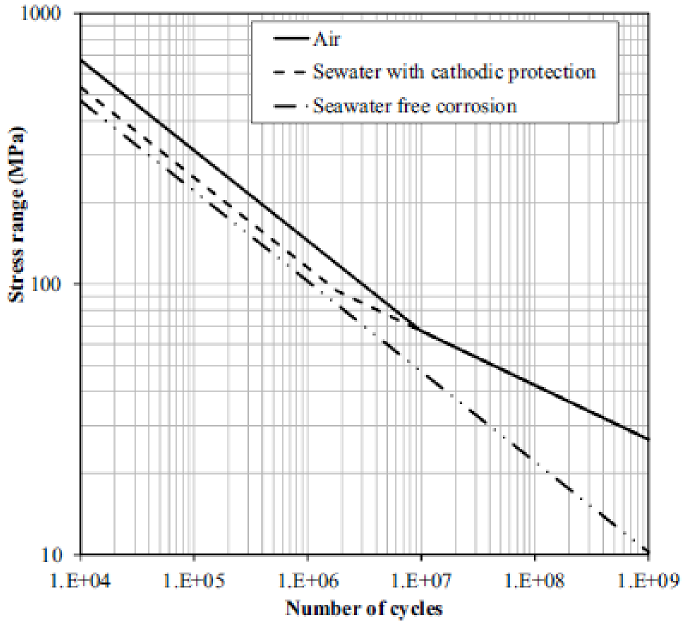

Figure 7: Typical S-N Curve for tubular joints in air and seawater with cathodic protection. DNVGL-RP-C203 (DNV, 2019)

Fatigue loads considered in the thesis are all static and fluctuating, acting on the structure or any of the structural components such as live load, dead weight, transient temperature changes and dynamic responses. Most importantly, the global cyclic loading due to wind and waves are the critical loads. The cyclic nature of these load loads tends to induce movements and acceleration in the fixed structure's static state, resulting in a tilting moment. Insufficient data of fatigue loads and stresses are a significant source of uncertainty that can result in an inaccurate estimation of the fatigue life. Description of the loads and modelling will be further treated in the next chapter.

2.6.1.1 **Fatigue Analysis: S-N Approach**

There are mainly three methods of evaluating fatigue stress based on their corresponding S-N curve: the nominal stress method, structural hot-spot stress (SHSS), and the notch stress method. The most applied methods are the nominal and SHSS methods.

The nominal stress approach is based on extensive tests of standardized weld joints and connections. These welds and connections are divided into several classes depending on the geometry and the direction of the stresses. And each class has a designated S-N curve. Each construction detail, at which fatigue crack may develop potentially, is placed in its relevant joint class based on the guidance provided by codes or standards such as DNVGL-RP-C203.

Experiments confirm a general assumption that welds of a similar shape have similar fatigue behaviour. So a single design S-N curve can be applied for any connection in this weld class (Ersdal 2019). This analysis does not consider stresses because of the macro-geometric shapes of the component around the joint and their effect (Ersdal 2019).

The structural Hot-Spot stress approach of fatigue design is mainly applied for cases where the dimensional variation of a specific detail must be considered or the absence of a standardized weld class. This method can incorporate the geometric stress concentration as a representative of the stresses experienced by the detail. SHSS is generally local stress at the weld toe, and as recommended by in recent DNVGL-RP-C203 guideline, the stress at the weld toe is extrapolated from two or three points adjacent to the weld toe. Based on the S-N approach, the computation of the hot-spot stress is given as a product of the nominal stress and the geometric stress concentration factor:

$$\Delta\sigma_{Hot-Spot} = SCF \cdot \Delta\sigma_{nominal}$$

Alternatively, the SHSS can also be obtained directly from FEM analysis. The hot-spot approach is usually deployed for tubular structures with parametric SCF equations in simple tubular joint configuration (see, Efthymiou, 1988, for example). Significant effort has been put into the research and development of parametric equations for offshore tubular joints, which is the cause of the evolution of SCF for tubular joints. Consequently, the assessment of tubular ageing offshore structures using SCF equations in current design standards guidelines, which are more advanced, are likely to produce different fatigue life estimates from the original one calculated during the initial design.

2.6.1.2 **Design S-N Curves**

S-N curves are developed based on characteristic experimental valves from fatigue tests such as mean-minus-two-standard-deviation. They are developed for all fatigue analysis methods. The basic design S-N curve equation is giving as:

$$\log N = \log A - m \log S \tag{2.8}$$

where N is the estimated number of cycles to failure for a known stress range S. A is the intercept on the $\log N$ axis on the S-N curve while m is the negative inverse slope of the S-N curve (usually 3)

S-N curve for nominal stress approach of fatigue analysis is dependent on the direction of the stress in the weld joint and the geometry weld joints. There are various classes of the S-N curve. DNVGL-RP-C203 (DNV, 2016a) recommended that each member of the structure susceptible to fatigue crack be placed in an appropriate joint class. For cases of multiple fatigue crack sites, each site would have to be classified independently based on the guidelines provided by DNVGL-RP-C203.

For the hot-spot stress method, the choice of the S-N curve does not require curves that consider the effect of geometry SCF. DNVGL-RP-C203 recommends the D curve DNVGL-RP-C203 (DNV, 2016). Jacket structures are usually analysed based on hot-spot methods in conjunction with relevant parametric equations for the SCFs. Most design guidelines have considered the inclusion of a specific T-curve, which was developed for tubular joints. Certain environmental factors do, however, influence the choice of an appropriate S-N curve, such as the environmental conditions of the structure.

2.6.1.3 **Environmental Impact on the Choice of S-N Curves**

Various offshore design codes and standards such as DNVGL-RP-C203 and ISO 19902:2007 has provided a guideline for the selection of S-N curve in seawater with corrosion protection. It is widely accepted based on previous research work, that the fatigue life in seawater under free corrosion is about one-third of the fatigue life in air at high-stress ranges (i.e. when N is less than 10^7 cycles). Fatigue life data shows that for strictures with cathodic protection (CP), whilst the fatigue life is lower when compared to that of air, the CP significantly improves the fatigue life compared to fatigue under free corrosion. The CP system is, however, expected to be appropriately maintained, especially when life extension is being considered, as a failure of the CP system can be a catalyst to the ageing of the structure (Ersdal et al. 2019)

2.6.1.4 **Damage calculation**

The S-N curves are mainly results from testing specimens under cyclic loads at a given stress level when realistically the structural members of an offshore structure undergo varying stresses. To mimic the offshore environment, fatigue data obtained through S-N curves are normally under variable loads as represented by the stress ranges. Palmgren Miner (Miner) proposed a rule for calculating the cumulative damage of a structure. The rule assumes the total damage accumulated is obtained by the linear aggregation of the individual damages and is given by;

$$D = \sum_i D_i = \sum_i \frac{n_i}{N_i} \qquad (2.9)$$

Where D is the cumulative total damage, n_i and N_i are the number of cycles and the number of cycles to failure of constant amplitude, respectively.

2.6.1.4.1 Design Fatigue Factors

The design fatigue factor is a safety margin introduced in the S-N approach. The value depends mainly on the accessibility and criticality of the components of the structure or critical elements in the structure. DFF ranges from 1 to 10 in ISO

19902 (ISO 2007) and NORSOK N-001 (Standard Norge 2012), whilst for DNV, it ranges from 1 to 5, taking into account the location DNV-OS-J101 (DNV, 2014)

2.6.2 **Fracture Mechanics**

Fracture mechanics analysis of fatigue is a complementary approach to offshore structures' stress life approach (S-N) approach. The fracture mechanics approach is more sophisticated in estimating the remaining life of an ageing offshore structure. It can assess the defects detected during construction and in-service, unlike the S-N approach (Ersdal et al., 2019), and enables the analysis of critical parameters such as geometry and changes in loading patterns during life extension assessment.

Fracture mechanics is based on a simple assumption of the general linear elastic principles and the existence of a defect in the structure. Nevertheless, where there are no data of the size of the defect or defects not found, a defect size equivalent to the limiting size detectable by the applied NDT is assumed. The assumed minimum detectable defect size also depends on an expert elicitation to recommend suitable dimensions based on the probabilities of detection and sizing, taking safety margins into account and shun conservatism.

Following extensive research on the detection of defects and technological advancements, there have been several developments and a better understanding of the failure mechanism of offshore structures and, thus, the formulation of improved guidelines on structural integrity assessment. The principal standards for offshore structures are API 579 (API 2016) and BS7910:2015 (BSI 2015). However, DNVGL-ST-0262 (DNV 2016) also provided a guideline for offshore wind turbines.

2.6.2.1 **FM – Crack Growth Analysis**

Predicting crack growth using the FM approach is based on the famous fatigue crack growth law by Paris and Erdogan (1963). Paris law opined that the rate of fatigue crack growth is proportional to the range of the stress intensity factor, $\Delta \boldsymbol{K}$, as presented in Figure 8. There are three steps of crack growth:

- Stage I: This stage is referred to as the crack propagation step, which is considered to occur only when the range of the stress intensity factor exceeds the threshold stress intensity factor range, $\Delta \boldsymbol{K}_{th}$
- Stage II: This stage is referred to as the crack growth step where at intermediate values of $\Delta \boldsymbol{K}$, there exists an approximately linear relationship between the rate of crack growth and $\Delta \boldsymbol{K}$ on a log-log scale. The most used model of this relationship is given as:

$$\frac{da}{dN} = C(\Delta K)^m \qquad (2.10)$$

  where $\frac{da}{dN}$ is the crack growth rate, C and m are material constant and load conditions (N-006, 2009; Moan and Ayala-Uraga 2007)
- Stage III: This stage is usually known as the accelerated crack growth step, which is where the material becomes unstable and results in fracture due to the stress intensity factor reaching its critical level, K_{cr}.

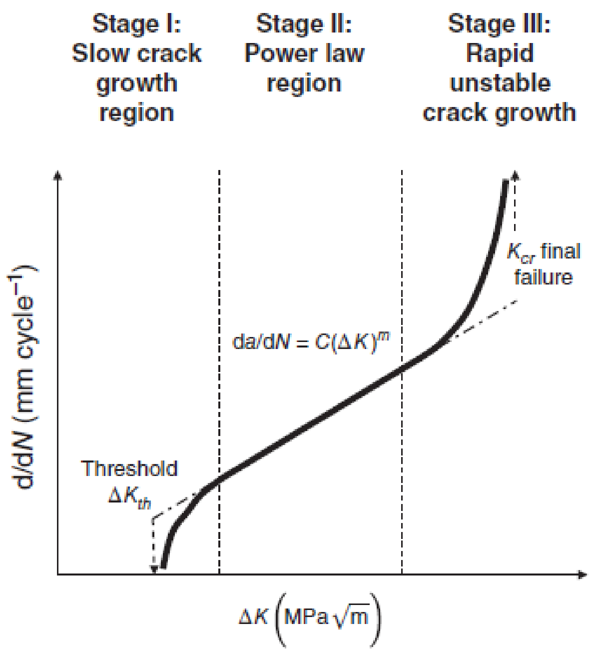

Figure 8: Fatigue Crack growth rate (Erdal et al., 2019)

The Paris law is only applicable to the Stage II region. Several other crack growth laws are proposed to account for Stage I crack growth, which accounts for a significant part of the entire fatigue life. Bathias and Pineau (2010) presented a detailed review of crack growth assessment. The fatigue life is generally determined by the integration of the fatigue crack growth law which considers the number of fatigue cycles, geometry, defect dimensions, applied load, and properties of the material. See King et al. (1996) for a detailed review of fatigue crack growth data for offshore structural steels. The computation of the integrated crack growth law is given as:

$$N = \int_{a_i}^{a_f} \frac{da}{C \cdot (\Delta K)^m} \tag{2.11}$$

Where, a_i and a_f are the initial and final crack sizes of the structural components. The range of the stress intensity factor $\Delta\boldsymbol{K}_{th}$is related to other factors such as the R-ratio to account for mean stress effects. The R-ratio varies from environment to environment. The equation of the R-ratio in air and seawater with cathodic protection is giving as:

$$\Delta K = Kmax + Kmin \tag{2.12}$$

where,

$$K = Y \cdot \sigma \cdot \sqrt{n \cdot a} \tag{2.12a}$$

where σ is the stress and a is the crack depth,

$$\Delta K = Y \cdot \Delta \sigma \cdot n \cdot a \tag{2.12b}$$

2.6.2.2 **Assessment of Fracture**

ABS (2014) ‘Guide to Fatigue Assessment of Offshore Structures’, DNV-OS-J101 (DNV 2014), DNVGL-RP-C203 (DNV 2016) and DNVGL-ST-0262 (DNV 2016) have dealt extensively with the subject of fatigue life as it relates to life extension of a fixed offshore structure. The widely deployed methodology is that of the design factor to fatigue life. This approach utilizes the concept of fatigue utilization index (FUI), which is the ratio between the fatigue design life and the effective operational time. Simply put, the ratio of the design life to the actual remaining fatigue capacity.

Therefore, if the FUI is equal to one, it implies that the remaining fatigue capacity is equal to the design life. However, if the structure has cracks due to fatigue before the FUI reaching one, the structure can be assessed, and the critical members of the structure are strengthened either by replacement or grinding out of defects. Alternatively, suppose the FUI is yet to reach one, and there are cracks observed and no strengthening or compensating measures in the form of structural improvements. In that case, DNVGL-ST-0262 (DNV 2016) recommends the structure should undergo a non-destructive examination (NDE) at an intermediate survey which corresponds to the larger extent required for the renewal survey. Nevertheless, no actions are needed if no cracks were observed before the FUI reaches one (Ersdal, 2019).

Other codes such as ISO19902 recommend that structures be re-used or converted to different operations after the design life has been extended before fatigue damage would have to be predicted by observations during an inspection. Also, the absence to crack does not imply the absence of defect, as it is generally assumed that before damage in terms of Miner's is 0.3 for welded tubular join and 0.5 for welded plate (Twed and Freeman 1987). ABS (2014) Guide to Fatigue Assessment of Offshore Structures applies a value of zero for those structural details that will be modified to eliminate prior damages. This rule is usually applied to an existing structure that is converted or re-used.

2.7 Context of Structural Reliability

2.7.1 **Background**

Structural reliability design generally aims to provide reliable structures with high performance, considering the uncertainties with some degree of conservatism. Subsequent requalification of structures that have exceeded their original design service life requires reassessment with the intention of ensuring the integrity and safety of the structure for extended service. Reliability analysis is used to perform the integrity assessment for both new and existing structures as well as provide a framework for decision making in situations where uncertainties cannot be incorporated, giving the general deterministic methods.

The initial application of structural reliability was to quantify the safety of the structure considering the uncertainty of the variables by using the mean and standard deviation (Stiansen and Thamyabali, 1987). The initial reliability theory was in the field of aerospace, nuclear and electronics. With widening interest from other fields of study and the evolution of computerized programming, structural reliability has been adopted in most manufacturing, production, and engineering designs.

In Cornell (1969), the Mean Value First Order Second Moment method (MVFOSM) of reliability analysis was proposed, which also did mark the initial method of deriving the reliability index based on mean values of the load and resistance variables. This method did not consider the formulations of the limit state functions, giving rise to the introduction of the Hasofer and Lind reliability index method (Hasofer and Lind, 1974), which also was the first approach to use geometric calculations to determine the reliability index.

In the past few decades, there have been various developments in reliability assessment tools and methodology, mostly aimed at improving the analytical constraints and achieving a more accurate result. MCS techniques have been widely used for reliability assessment as it provides an alternative for the limitations of using deterministic techniques. While First and Second-order reliability methods (FORM and SORM) are also widely used as they offer an alternative of accounting for complicated formulations of limit state or performance function and transformation to a complex statistical distribution.

At the system level of reliability analysis, different approaches from a component-based to a probabilistic analysis approach are employed with the aid of sophisticated computational effort and mathematics. In Thoft-Christensen (1986), the “branch and bound” method of system reliability is elaborately described.

The evolution of structural reliability now tends towards structural optimization schemes, and reliability stands as a design constraint. This scheme is expected to consider the time performance of the structures in the time-variant analysis (Stocki et al., 2001). Also, this new scheme is expected to transform the state-of-the-art designs into state-of-practice methods to allow the application in more essential engineering problems.

2.7.2 Structural Modelling

The harsh environmental conditions associated with operations in the offshore environment requires the concept of structural reliability to be factored into the whole life cycle of the structure. The introduction of the Load Resistance Factor Design (LRFD) approach in the design standards has contributed to ensuring a more structured design of offshore structures. Also, the introduction of target reliability for various offshore designs depending on the application has offered the engineer a design guideline, resulting in the formation of design standards that provide a thorough guideline for reliability-based design (Onoufriou, 1999; ABS 2014).

These changes in structural design and the quest for a comprehensive understanding of the behaviour of a structure in response to its environment through its whole life cycle have resulted in broader acceptance of the reliability assessment framework. In this era of technological advancement and the availability of sophisticated computers and computing tools with higher modelling precision, reliability assessment methodology is being deployed to the global structure and the localized section of the structure to obtain responses of that section under different load scenarios. In the nearest future reliability, assessment can be more beneficial as it is also able to improve cost and safety.

2.7.3 Computational Fluid Dynamics (CFD) Modelling and Simulation

The accuracy of a reliability assessment depends mainly on the model precision, uncertainty consideration and the design assumptions made. Structural complications are usually non-deterministic, especially in the offshore environment, due to the restrictions and difficulties in understanding the terrane. Therefore, quantification of risk yields for random (stochastic) parameters to be considered. Critical parameters for consideration in either the conceptual, preliminary or detailed design are those of the environment because of the level of randomness. Although there are extensive metocean data of some sites, due to the extensive number of projects and data capturing that has been carried out. For green offshore sites, detailed research and data capturing are required using both learnings and advanced technology from previous projects as a baseline and calibrating the parameters' statistical properties to reflect the current sites to avoid inaccuracies in the results.

In modelling the capacity of the structure and its members, stochastic parameters modelling tends to be less sensitive to the overall reliability index (Sigurdsson et al., 1994). This position has also been substantiated by the reports from the design standards consideration for partial safety factors, which showed that the parameters for the environmental loads were more sensitive than the material properties.

OWT’s support structures model can be categorized into two groups, i.e. the 1D (one dimensional) beam model and the 3D (three dimensional) shell model. The 1D beam model generally represents the support structure into sequences of elastic beam elements. This method has been widely used in structural modelling of OWT support structures because of its computational efficiency and acceptable accuracy for modelling global structure behaviour (Bosanyi, 2009).

Though efficient, the beam model has a limitation of accurately representing the local structural responses such as local stress concentrations (Petrini et al., 2010). The 3D shell model, on the other hand, generally constructs OWT support structures using shell elements and is capable of accurately estimating the structural responses and examining the detailed stress distributions across the support structure (Wang et al., 2016). In a bid to accurately capture the structural responses of the OWT support structure under the various loads, this thesis adapted the 3D FEA (finite element analysis) method to construct the structure model due to its high dependability for accurate results over the 1D beam model.

OWT installation and its support structures is indeed a promising approach for renewable energy generation to meet the global demand for clean energy. However, the impact of corrosion and fatigue damage due to the offshore marine environment and the soil properties are major factors responsible for the degradation of the steel material, which ultimately affects the resistance of the material (Figueira et al., 2017; Yeter et al., 2017). Due to the predominant level of fatigue loads accompanied by many load cycles owing to the combined actions of wind and wave loads, fatigue performance of welded connections is a design driving criterion for offshore wind turbine support structures (Dong et al., 2012). Corrosion can reduce the material thickness, thereby making it susceptible to fatigue crack initiation and buckling, which may lead to failure of the structure. Several methods, such as the S-N curve and Miner's rule, have been analysing the effect of fatigue. In contrast, for corrosion NDT (non-destructive testing) methods are used for the analysis. While the formation can influence the stability of the foundation in the face of adverse environmental conditions.

2.7.4 Basic Formulation of Structural Reliability Analysis Principles

Generally, the behaviour of a steel structure can be assessed based on the loads L impacting the structure and the load-bearing capacity/resistance of the structure R. This can be represented mathematically to reflect the acceptable design criterion for a specific failure mode or limit state as:

$$D_d = \gamma_0 \sum_i D_{ki}(F_{ki}, \gamma_{ki}) < C_d = \frac{C_k}{\gamma_M} \tag{2.13}$$

The safety margin Z, of the structure, can then be given as:

$$Z = R - L \tag{2.14}$$

I reality, both load and the resistance involve several variables subject to a wide degree of uncertainty. For instances where the load and resistance are equal in terms of values, the limit state expression can be written as:

$$Z(X) = 0 \tag{2.15}$$

This implies when $Z(X) > 0$ the structure is operating in a safe region and when $Z(X) < 0$, the structure is said to be operating in an unsafe or failure region. For an individual limit state, the probability of failure can be determined by:

$$P_f = P\{Z(X) < 0\} \tag{2.16}$$

Considering probabilistic models for the assessment, the variables $X = [X_1, X_2, \dots X_n]$ further factorization of the fact that the variables are time-independent joint probability density function $\varphi_x(x)$, the equation of the probability of failure can be described as an integral:

$$P_f = \int_{Z(X)<0}^{Z(X)=0} \varphi_x(x) \tag{2.17}$$

The above equation can be expanded to apply to specific time-dependent quantities, which is further transformed to time-independent quantities (Holicky and Vrouwenvelder, 2005). For instances where this is not possible, the procedure for calculating the probability of failure becomes more complex and, in practice, is performed with the help of computer-aided software or programs (ISO, 2008).

The term "probability of failure" is equivalent to the reliability index 'β' and is commonly used in most design standards and other relevant documentation. The probability of failure is, however, correlated to the reliability index in the equation given below as:

$$\beta = -\phi_U^{-1}(P_f) \tag{2.18}$$

where, $-\phi_U^{-1}(P_f)$ is the inverse standardized normal distribution function. Figure 9 presents a correlation between the probability of failure and the reliability index. For a sample case where a single point load acts on a beam, the load and resistance are assumed to be normally distributed. And the correlations of their means, standard deviation and reliability index is given in Figure 10. For a safe system the magnitude of the strength of the structure is expected to be greater than that of the load at all times.

Quantitatively the definition of risk is the product of the probability of failure and the consequence of failure. While the probability of failure depends on the structure's reliability, the consequence of failure is dependent on the function and specifications. This means that there could be different values of risk for different combinations of function and specification. For example, a failure in an unmanned offshore structure such as OWT could stop operations but not necessarily cause fatalities. Hence, the measure of the consequences can be considered low. A similar outcome can be the case for a design with a higher probability of failure but a lower consequence of failure. The calculated reliability levels can thus be interpreted in these lines.

A possible reliability index trend that occurs during the operation of an offshore structure is the increase of load effect, which occurs over a period due to crack growth etc. Therefore shifting the load curve to the right, whilst the resistance may decrease correspondingly due to deterioration of material properties. This change would result in a reduction in the relative difference between mean values, and subsequently, the reliability index but increases the probability of failure. These phenomena are an important design factor for consideration during the modelling of the structure to avoid undesirable residual uncertainties in the calculations.

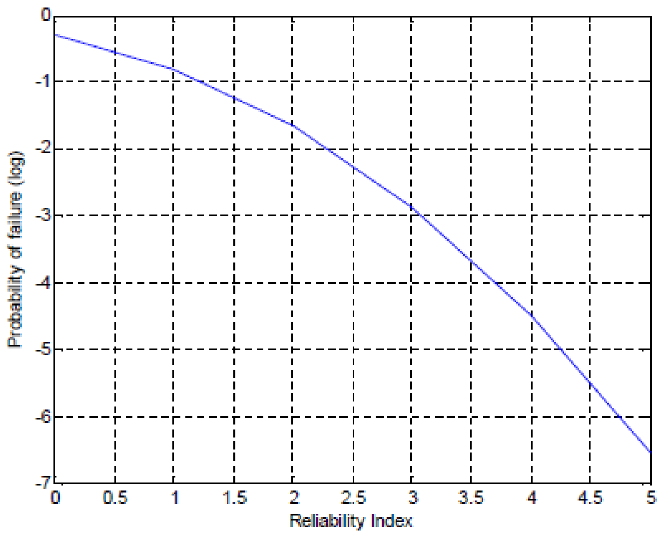

Figure 9: Correlation of Probability of failure and reliability index

Quantitatively, risk can be defined as a product of the probability of occurrence and its severity (Burdekin, 2006). While the probability of occurrence is influenced by structural reliability, function and specifications influence the severity or consequence. It then means that the value of the risk would be different for different combinations of design parameters. So, an unmanned offshore structure can suffer failure but not record any fatalities, which implies that whilst the probability of occurrence is high, the severity is low. This understanding is very critical in the interpretation of reliability levels.

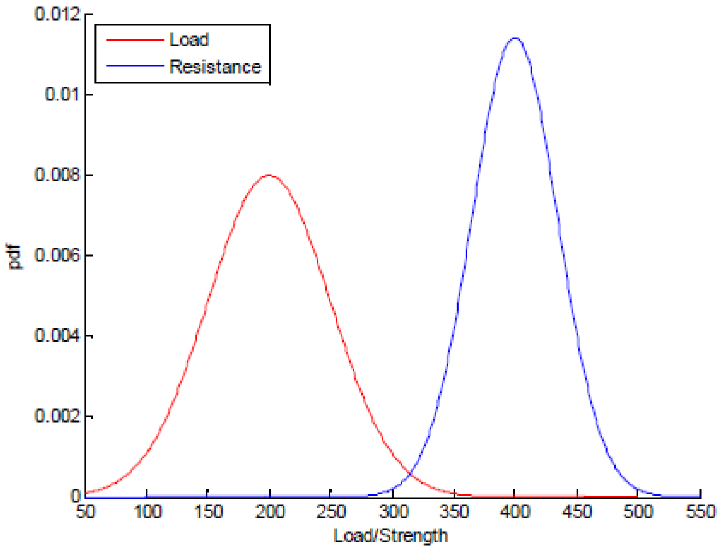

Figure 10: Reliability Index Definition

2.7.5 **Response Analysis**

Several analytical methods are used to analyse structural reliability, which may be categorized as linear or non-linear, deterministic, stochastic or static or dynamic. The decision for the approach to be adopted for any analysis must be made before the commencement of the analysis. And it shall be a combination of the categories mentioned above. The selection would be based on the properties of the environment and the structure under consideration. Depending on the complexity of the structure, it might require a different type of analysis. Skjong et al. (1995) presented a block diagram of a global response analysis procedure for marine structures, which provides a guide on the appropriate selection of the analytical method.

2.7.5.1 **Static Response Analysis**

The equilibrium equation for static analysis is expressed as follows:

$$K_r = R \tag{2.19}$$

Where $\boldsymbol{K}$ is the global stiffness matrix derived from the combination of the elemental stiffness matrices; $\boldsymbol{r}$ is the unknown nodal displacements, and $\boldsymbol{R}$ is the nodal load vector. Generally, the finite element analysis based on the equilibrium equation should follow the following procedure defined by Bell et al. (1970).

- Discretization, which implies an assembly of finite interconnected elements, approximates the structure
- Element Analysis, which implies the stiffness properties of each member element is determined, and the loading is transformed into nodal equivalent forces
- System analysis, in which the individual elements of the structure are integrated with a system-level singular element stiffness matrix $\boldsymbol{K}$ and load vector $\boldsymbol{R}$. The equilibrium equation would then be determining the nodal displacement vector $\boldsymbol{r}$.
- Post-processing of the results will be based on stresses from rotational and translational displacements of an individual member of the structure.

Cases involving non-linear problems can be resolve using both numerical and analytical methods such as the Ritz method. Although a numerical approach is mainly used due to the stepwise integration principle i.e., non-linear problems are further broken down into a series of linear problems. The reduction of a non-linear analytic structural system can reduce the problem of calculating the displacement vector $r(t)$ that produces an internal force vector $\boldsymbol{F}_{int}(t)$ that balances the applied force $\boldsymbol{R}(\boldsymbol{t})$. The equilibrium equation can be rewritten as:

$$\boldsymbol{F}_{int}(\boldsymbol{t})\boldsymbol{r}(\boldsymbol{t}) = \boldsymbol{R}(\boldsymbol{t}) \tag{2.20}$$

This equation can be solved incrementally with a corrective iteration.

2.7.5.2 **Dynamic Response Analysis**

Dynamic analysis is likened to static analysis but includes time dependency, inertia and damping as a local and global effect. The equilibrium equation can thus be given as:

$$\boldsymbol{M}\ddot{\boldsymbol{r}} + \boldsymbol{C}\dot{\boldsymbol{r}} + \boldsymbol{K}\boldsymbol{r} = \boldsymbol{R}(\boldsymbol{t}, \dot{\boldsymbol{r}}, \ddot{\boldsymbol{r}}) \tag{2.21}$$

where $\boldsymbol{R}(\boldsymbol{t}, \dot{\boldsymbol{r}}, \ddot{\boldsymbol{r}})$ is the time-dependent load, $\boldsymbol{M}$ is the global matrix, and $\boldsymbol{C}$ is the global damping matrix. The structure's global mass and damping properties can be derived from the cumulative mass and damping properties of individual member elements. This equation can be applied for both linear and non-linear systems

2.7.5.3 **Deterministic and Stochastic Methods**

Deterministic methods are analytical methods where the magnitude of variables per time is known. Deterministic analysis requires the initial consideration of the statistical data for environmental lading. For instance, the extreme response analysis for wave would be based on the most severe wave expected to cause the highest response (impact). This entails the exposure of the structural model to a unidirectional periodic wave. The calculated loading effect is based on the loading in the time domain at giving points during the wave cycle.

While stochastic methods are analytical methods in that variables are defined by probabilities (statistical distribution), this implies the magnitude of the load and response is not definite at any given time.

2.7.5.4 **Analytical Method Selection Criteria**

There are several methods for selecting the appropriate analytical method that adequately takes into account the local and global, non-linear and dynamic effects in the analysis (Barltrop and Adams, 1991; Faulkner et al., 1990; SNAME, 1993). City University, London, in a joint industry project, conducted a comparative study on the various types of analysis for a reference structure under the same loading conditions, including a combination of linear and non-linear analysis with random and regular wave loads. The results obtained are presented in Figure 11; the results compared the maximum base shear that was derived.

For the various methods considered for the dynamic effects, either direct application of dynamic analysis or by applying correction factors to the result of a static analysis, which could also be referred to as ‘quasi-static. Skjong et al.

(1995), as cited by Kolios (2010), have recommended criteria to be considered when choosing between static and dynamic analysis methods. For fixed offshore structures such as the jacket type, the dynamic effect of the global response analysis should be considered.

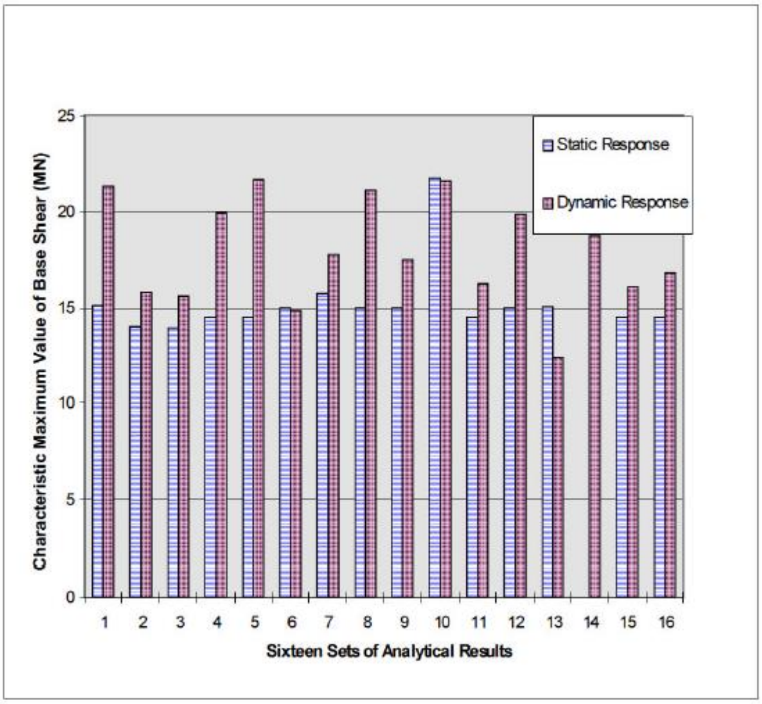

Figure 11: Scatter Response (Max Base Shear-MN) by Different Analytical Methods. Source: (Kolios, 2010)

2.7.5.5 **Reliability Assessment Methods Gap Analysis**

System reliability assessment has witnessed several important improvements in the last decades. A complex structural system with complex failure paths can be disintegrated into a network of subsystems connected in parallel and series configuration, with individual subsystems representing a failure mode. The overall structural reliability can be derived from the calculation of the network of individual reliability at the components and subsystems level. However, a complex offshore structure such as the jacket support structure with a potentially high number of structural components and failure paths tends to defy this methodology. Hence, a more accurate system reliability method for such structures is required, and Figure 12 presents some alternative methodology Onoufriou and Forbes (2001).

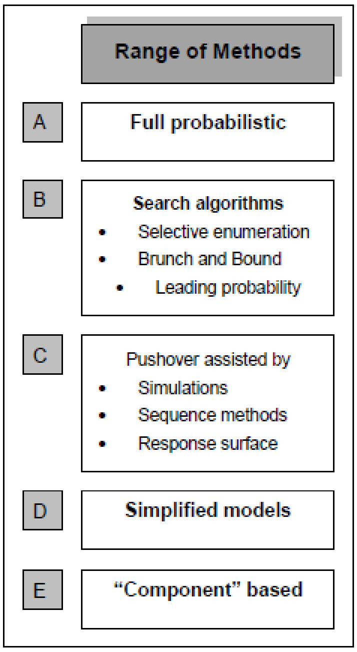

Figure 12: Range of systems reliability assessment methods for complex structures. Onoufriou and Forbes (2001).

With the availability of more advanced and efficient computerized systems, computation of system reliability of complex structures like the jacket support structures has driven the development of search algorithm aimed at identifying the most dominant and probable failure path as well as compute the reliability of the system (Onoufriou and Forbes 2001). The Branch and Bound method (Murotsu and Okada) is the most used search algorithm technique for system reliability computation for jacket support structures. This comes with a drawback due to its cost of computation. Other methods include the Marginal and leading probability method (Thoft-Christensen and Murotsy, 1986) and selective enumeration techniques. However, they are also limited by their inability to capture all potential failure paths.

Identification of the most probable path of failure is a crucial aspect of system reliability. Pushover analysis is one of the potent means of identifying deterministically the critical components of the structure that is most susceptible to failure. Still, those not necessarily taking into account the residual strength after failure post-failure may re-channel loads and result in different failure sequences and a different response of members. Sigurdsson et al. (1994) reported an analysis for extreme loading conditions. They found the calculated reliability index of the failure path identified using the pushover analysis, identified the failure path after extensive simulations.

The simplest method in the list of methods in Figure 12 is the component-based method. The component-based method treats the entire system as a component, with either an associated coefficient of variation (CoV) suitable to represent the probability of failure of the system or a deterministic resistance value (Van de Graaf et al., 1994; Tromans and Van de Graaf, 1992). This method is dependent on a realistic representation of the resistance. It can provide a good nonlinear model with a possibility of less uncertainty depending on the competence of the user.

The choice of method in Figure 12 depends mainly on the application, whereas the classification differentiates the methods. As you go down the categories from 'A' to 'E', the complexity of the structure reduces but is more appropriate for practical reliability assessment (Onoufriou and Forbes 2001). This thesis adopts a non-intrusive simplified method with an associated simulation method that takes into account the effects of uncertainties by performing sensitivity analysis. This method is a probabilistic approach that treats the system as a component. Load and response parameters are discretized and are then post-processed using multivariate regression analysis to formulate the performance function. The reliability index under the pre-defined limit state is then estimated using FORM (First Order Reliability Method), which also allows for the calculation of low reliabilities, which is not tenable with other methods.

This method enables the prediction of different limit states, therefore, provides a faster return time and easier for asset owners and service providers to deploy. The limitation of this method is a limited number of performed simulations given the size of the problem and high computational demands. Tromans and Van de Graaf (1992) and Frieze et al. (1997) provided simple procedures helpful in implementing the combination.

2.7.5.6 **Practical analytical methods**

All the systems reliability methods discussed are based on a common fact that failure will occur at some instance if the maximum load acting on the structure is sustained over a period. Failure in this context would be based on when the structure collapses or have a large displacement. Local failure of an individual member would affect the overall structural stiffness of the structure, and as such, stresses would be redistributed to alternative load paths. Bearing in mind the structure has a residual strength due to hysteresis, that strength of the structure can then be modelled by applying appropriate resistance at the nodes of the failed structural members and the system redefined. The stress would then be re-evaluated, and the procedure is repeated iteratively until all member failures are identified and strengthened.

2.7.5.7 **Post failure Behaviour Modelling**

One of the most critical factors that determine the reliability of a structure is the post-failure behaviour of the elements. There are two extreme means of failure, namely brittle and ductile failure. The brittle failure is without warning and renders the structure entirely out of use after failure. In contrast, in the case of ductile failure, the structure can still retain its load-bearing capacity and could be repaired.

Actual engineering materials used for structures are, in most instances' combination of the two in the form of alloying to achieve the best composition for the intended application. For probabilistic design analysis of offshore structures, a bi-linear model is used. Before the failure, the components behave in a linear elastic manner, and at failure, they behave linearly with a modified stiffness matrix. Figure 13 presents the models of material post-failure behaviour.

The semi-brittle model refers to a state of the material where the resistances capacity of the member increases elastically with an increase in the load acting on the member. At post-failure, if the axial deformation of a structural element exceeds the design failure value, the element force drops to a fraction φ, of its non-failure capacity. Karamchandari and Cornel (1991) have reported values of

φ *as* 1.0 and 0.4 for tension and compression failures, respectively. This thesis would adopt the ductile tension failure load.

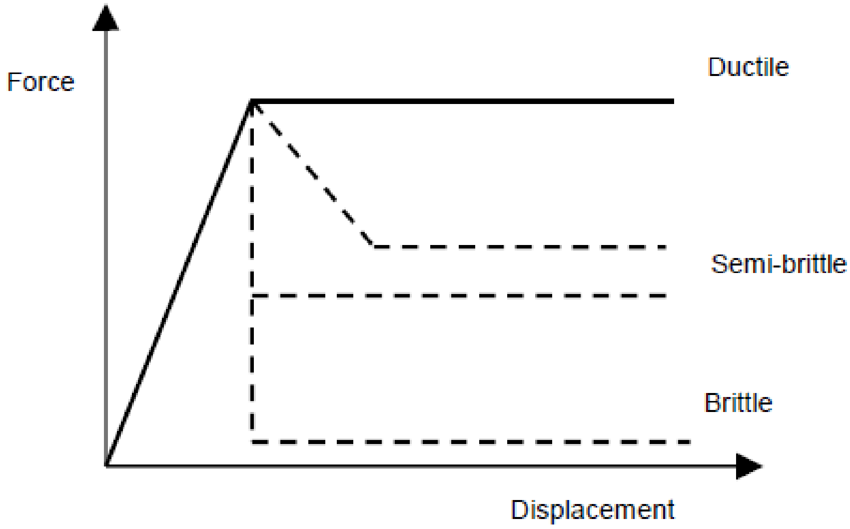

Figure 13: Post-failure Behavioural Models (Kolios, 2010)

2.7.5.8 **Computational Methods of Systems Reliability**

There are generally two main types of system reliability computation methods: Hohenbichler approximation (Hohenbichler, 1982) and bounding methods (Pham, 2003). While the former computes the probability of failure from the multi-normal distribution function, which has so far provided an accurate estimation of the probability of failure of structures, the bounding methods (simple and Ditlevsen (Ditlevsen, 1979)) are also capable of estimating the probability of failure especially for complex systems where Hohenbichler approximation difficult to use (Thoft-Christensen and Murotsy, 1986).

The application of simple bound approximation in the computing reliability of complex systems have limited requirements and is widely used. However, in terms of accuracy, it is less accurate compared to Ditlevsen bound approximation. Ditlevsen bound method is a narrower bound compared to simple bound, providing a smaller range of upper and lower bound and is calculated through numerical to account for the joint probabilities.

The simple bound uses the range between the minimum and maximum values of probability of failure, which could be very wide in some instances and can only provide rough estimates of the reliability. The simple bound method also requires the definition of the system's configuration, whether it is parallel or series or both.

2.7.5.9 **Types of System Reliability Method**

Series or weakest link system is a system whereby system failure can be traced to the failure of the weakest element in the system. Such an individual element as addressed has been in either functioning or failed state. This can be expressed by introducing a binary state indicator variable a_i with corresponding values of 0 and 1 for failed and functioning states, respectively. Therefore, for n number of elements, the probability of failure for a series system can be defined according to Thoft-Christensen and Murotsy (1986) as:

$$\max_{i=1} P(a_i = 0) \leq P_{fs} \leq 1 - \prod_{i=1}^{n} (1 - P(a_i = 0)) \tag{2.22}$$

In the above equation P_{fs}corresponds to the lower bound when there is a correlation between all elements, and the upper bound is uncorrelated to any pair of elements. If the probability of failure of an element is significantly greater than that of other elements, the probability of failure of the entire system would be approximately equivalent to the element with the significant probability of failure, thereby narrowing the gap between the upper and lower bound. Conversely, if the probabilities of failure are in the same range, the simple bounds are wide.

Parallel systems are regarded as functioning, provided at least one element is functioning. Simple bound for the probability of failure can thus be given as:

$$\prod_{i=1}^{n} P(a_i = 0) \leq P_{fp} \leq \min_{i=1} P(a_i = 0) \tag{2.23}$$

In the equation above P_{fp} corresponds to the lower bound when there is no correlation between any pair of members, with the upper bound corresponding to a full dependence between elements.

2.7.5.10 **Global Effect**

There are two main types of system effects, namely the deterministic and the probabilistic. The deterministic system effect refers to the system effects relating to redundancy incorporated into the structural design, which ensures the redistribution of loads in case of failure of the first member and increases the ultimate load-bearing capacity of the system.

The probabilistic effect refers to the randomness in the member capacities (Sigurdsson et al., 1994). The structural behaviour after the failure of the first member depends on several variables, such as the ability or the structure to redistribute the load and the ductility of individual members. The degree of static indeterminacy including other aspects such as the behaviour of the joints and foundation. For a holistic assessment of the system effects, certain factors such as the residual strength obtained from the structural model analysis must be assessed.

Generally, the failure of an individual member may not necessarily result in the total failure of the structure. Still, it may rather lead to a sequence of failure of other members until it attains ultimate strength. The reserve strength ratio is given as:

$$RSR = \frac{ultimate\ platform\ resistance}{design\ load} \tag{2.24}$$

RSR, values vary from load case to load case on every platform. For purposes of comparison between RSR values, the definition of the variable used for each load case must be given attention. Bea and Mortazavi (1996) and Aggarwal et al. (1990) have conducted an extensive study on the use and definition of RSR values. A four-tier system of structural analysis was developed to allow a simple assessment and requalification of platforms by way of best defining the content of the RSR to give an appropriate interpretation of the values. The first definition is given as:

$$RSR = \frac{ultimate\ lateral\ load\ capacity\ of\ the\ platform}{reference\ lateral\ loading} \tag{2.25}$$

Another definition used by shell Van de Graaf et al. (1994) is given as:

$$RSR = \frac{environmental\ load\ at\ collapse}{original\ design\ enironmentsl\ load} \tag{2.26}$$

For an undamaged structure with residual capacity, the magnitude of which can be described by its degree of indeterminacy. The concept of residual strength can evaluate the effect of a damage scenario. This could be a critical indicator for structural behaviour and can be defined as the residual resistance factor (RRF). The RRF is given as:

$$RRF = \frac{damagedstructure'senvironmental\ load\ at\ collapse}{intact\ structure'senvironmental\ load\ at\ collapse} \tag{2.27}$$

The ratio of the capacity of the damaged structure to that of the capacity of the undamaged structure can provide an essential indication of the platform behaviour (Choi et al., 2004). It can be defined as the damage tolerance ratio (DTR), which is given as:

$$DTR = \frac{ultimate\ capacity\ of damaged\ structure's}{ultimate\ capacity\ of\ undamaged\ structure} \tag{2.28}$$

The magnitude of DTR could further define the structural weakening due to structural damage. For instance, a DTR of 0.25 would imply a 25% loss of reserve strength.

2.7.6 **Stochastic Expansion**

The evolution of finite element methods in engineering now tends towards the use of stochastic methods. The important subject with regards to the scope is the discrete representation of the stochastic variables and the conforming interpretation of the stochastic responses (Choi et al., 2004). The use of stochastic expansion is aimed at considering a series of polynomials to assess the reliability of the system. There are several methods of polynomial expansion, namely Polynomial Chaos Expansion (PCE) and Karhunen-Loeve expansion (K-L), suitable for applying structural reliability assessment. In some cases, PCE and K-L methods are combined with finite element analysis methods, which serve as a helpful tool, otherwise known as Spectral Stochastic Finite Element Method (SSFEM), used for analysing structural reliability of the systems (Schueller, 1997).

Apart from being applied in SSFEM, PCE has also been successively deployed in other areas of study to represent uncertainty in the analysis of structural response. This method has been used for the analysis of soil mechanics (Ghanem and Brzakala, 1996), composite material (Soares and Chena, 1999), heat conduction (Ghanem, 1999) and two-dimensional elasticity (Spanos and Ghanem, 1991).

Stochastic expansions can be categorized into intrusive and non-intrusive formulations, as presented in Figure 14, Choi et al. (2004). Intrusive formulations are those in which the uncertainty is expressed explicitly inside the analysis of the system, such as the SSFEM (Spectral Stochastic Finite Element Method) and KL (Karhunen-Loeve) expansions (Babuska et al., 1991), to modify the stiffness matrix of the finite element analysis. Non-intrusive formulations are those in which uncertainties are represented in a non-explicit manner, therefore, treating the analysis code as a “black box” without requiring access to the analysis code. This approach is also regarded as the Stochastic Response Surface Method.

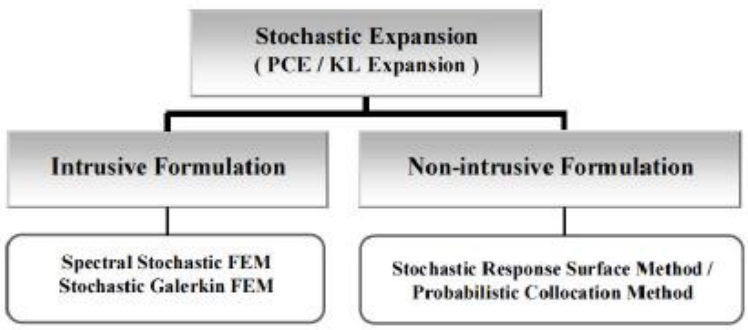

Figure 14: Stochastic expansion approaches (Choi et al. 2004)

2.8 Discussion

The chapter has dealt with the basic principles of structural design, reliability assessment of offshore steel structures. Basic mathematical models that form the basis of the formulation of the reliability prediction problem, the evolution of the application of reliability analysis and the motivation for the reliability framework was also discussed. Review and selection of the appropriate analytical method were discussed with its merits in conjunction with methods of integration from a system-level of reliability to a component level. Review and discussion of relevant design standards applicable to jacket structures were also presented. Finally, stochastic expansions methods were reviewed, followed by a brief mention of the spectral stochastic finite element method, which is intended to set the stage for the development of the stochastic response surface method that will be discussed in the next chapter.

3 NUMERICAL METHODS AND NON-INTRUSIVE RELIABILITY ANALYSIS OF AGEING OFFSHORE STRUCTURE

3.1 Introduction

The major drive for the offshore wind energy industry is to provide clean and affordable energy. Still, with the current volume of manufacture and the marginal profit, advance research would be required to drive the cost of wind power further down to a more competitive level, hence, the need for a reliable and economical design. Reliability-based design optimization (RBDO) has been an emerging promising extension of the conventional structural reliability analysis approach, as it provides cost-effective and reliable designs than conventional methods. RBDO employs more efficient sampling methods such as the response surface methods (RSMs), which improve the consideration of uncertainties in the structure's response.

This chapter will present the processes and development of the proposed methodology. Various numerical concepts and notations such as sampling and regression analysis methods are generally used to assess ageing and life extension of offshore structures. The strength and weaknesses of the proposed framework compared to state of the art are analysed using established scientific methods, providing justifications for proposing an improvement to state of the art.

The contribution of this chapter is to develop the proposed offshore wind turbine specific reliability analysis methodology. The proposed method shall be developed based on known numerical models, concepts and notations used to assess the structural reliability of offshore structures.

As mentioned above, the proposed methodology, which will be based on RBDO and first-order reliability methods (FORM), are validated against known standard examples. A comparison of the proposed and conventional methods will be addressed, highlighting their strengths and weaknesses. The proposed methodology is applicable for the reliability assessment of both ageing and new designs.

The major aim of assessing the reliability of an ageing OWT structure is to quantify the remaining fatigue life of the structure by quantifying how much damage has been done to the structure. Miner's rule has been widely used, and it is highly efficient when the loading history is known. However, its limitations come to bear when the loading history is unknown, and thus the loading sequence effect cannot be accounted for under variable amplitude loading scenarios (Aeran 2017). For ageing structures where the probability of unknown loading history is high, the non-intrusive method of reliability assessment is recommended. The structure is replicated in a CFD environment. In this case, ANSYS and the FEA model were developed. A probabilistic fatigue analysis would then be performed, which provides stress range as its output.

Fatigue limit state (FLS) is the predominant limit state. It is the primary cause of failure in welded offshore structures. The structural integrity is mainly dependent on the fatigue criteria (Lotsberg, 2016). Fatigue in conjunction with corrosion leads to corrosion fatigue which can drastically impact the reliability of an offshore structure, just like the stress concentration factor and the uncertainty in the design parameter. Proper quantification of uncertainties and accurate estimation of the design parameter is crucial in determining the reliability of the structure and, by extension, the remaining life of the structure.

This framework is divided into building blocks, which are further subdivided into processes. The building blocks are an aggregation of processes and stages of structural reliability assessment ranging from the requirements of the turbine to development of the model, data gathering and processing, selection of appropriate fatigue methods, simulation (ageing OWT), damage assessment, determination of the structural reliability and remaining life calculation by extension life extension of the structure.

The proposed framework can also be used for both probabilistic and deterministic analytical methods. While probabilistic methods involve using statistical distribution functions to represent the design parameters, deterministic methods employ the use of design values (mean, standard deviation). Also, the distribution functions can be imported into a probabilistic finite element method utilization simulation tool for the computation of the probability of failure. This might require additional computational effort depending on the complexity of the structure and the number of variables considered.

3.2 Proposed Framework

The proposed framework is birthed on a significant amount of research and well-established scientific backings and references of models within the energy-related research, such as the National Renewable Energy Laboratory (NREL) wind turbine models (for loads) and the OC4 jacket support structure (geometry and materials) which has been the basis of multiple highly cited research studies.

This study mainly focuses on the fatigue assessment of the main part of the braces using the SN approach rather than analysing the structural details of the brace. The proposed approach can employ appropriate stress concentration factors (SCF) to account for these geometrical details. However, the SCF is assumed to have been accounted for in the materials factor for ease of computation.

The main aim of a life extension assessment is to ensure the ageing offshore structure has sufficient remaining life and is safe in line with general design standards recommendations. Figure 15 shows a schematic flow diagram of the proposed framework.

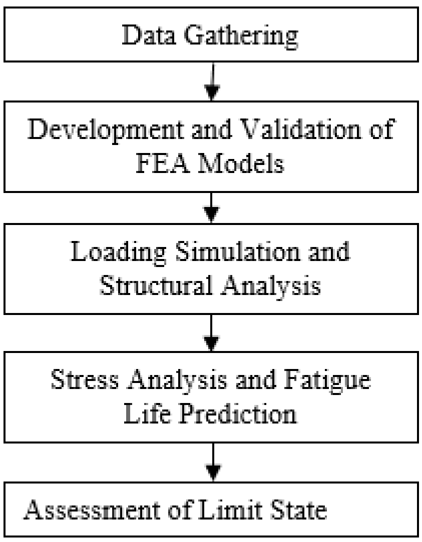

Figure 15: **Schematic flow diagram of the proposed framework**

Life extension projects often come up with surprises. Therefore, planning for both the expected and unexpected is essential. Before commencement, it is vital to ensure all resources are available and roles and responsibilities are understood, especially historical and live data collected of the structure.

3.2.1 Data Gathering and Screening

Data gathering is a crucial phase of this project. Data gathered are screened and properly indexed into the respective project phases and documented. Data of fatigue cracks or any other related fatigue information is crucial. However, in the absence of relevant fatigue data, a detailed fatigue analysis would be required by mostly assuming a crack initiation. Figure 16 shows an illustration of the process of data gathering

Data is gathered during design, fabrication, installation, and operation phases, such as structural drawings, details of relevant sections of initial codes and standards used during the initial design. FEA model, loading details, fabrication and installation reports including any modifications, accident and incidents report during service including any maintenance or repair works, potential fatigue crack sites, risk assessment reports, material testing, all revisions of FEA model etc. are all compiled for analysis.

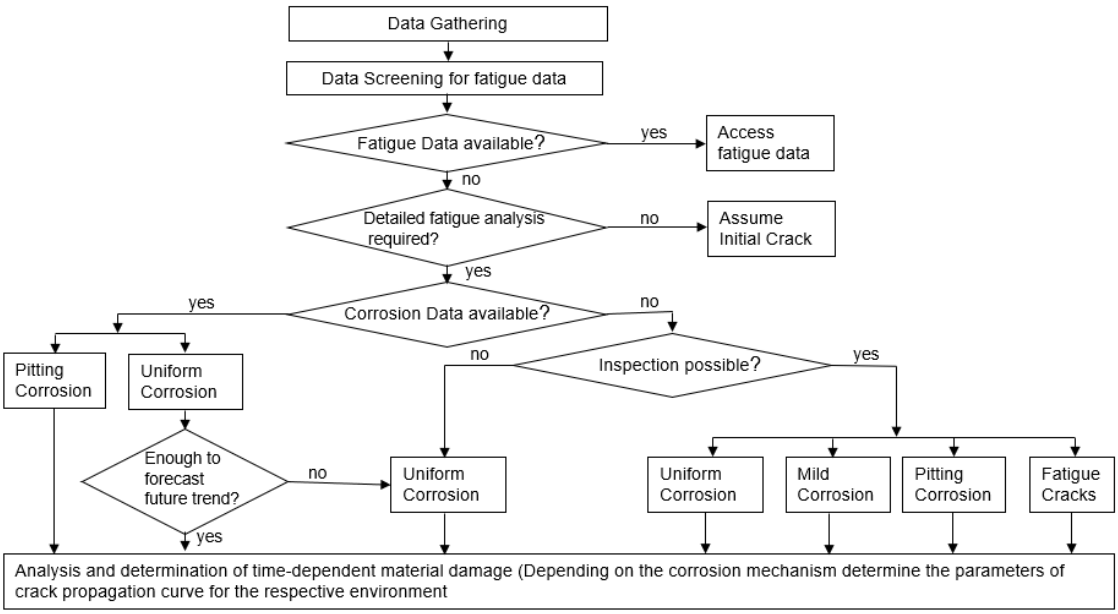

Figure 16: Data Gathering flow chart

However, quantifying uncertainty will be a major challenge if some, or most of these historical data are unavailable. Additionally, documents during the life extension phase such as the latest environmental loading conditions required modification based on the future application, technology gaps, equipment modification, latest industry codes and standards addressing the life extension should also be documented.

3.2.1.1 **Choice of Fatigue analysis method**

The choice of fatigue analysis method is a very critical aspect of this assessment. It is, however, dependent on the quality and quantity of data gathered. Important data such as structural degradation, the thickness of critical members and fatigue cracks are essential. As mentioned earlier, the industry practice for fatigue design of offshore jacket structure deploys the S-N method. Still, if there are enough credible data on fatigue crack, it is advisable to use the fracture mechanics method.

3.2.2 **Development and Simulation of Degraded FEA Models**

The development of the degraded finite element model of the structure to be as precise as possible to the information obtained from inspection data gathered is also crucial to assessing ageing structures for life extension. The data collected will be used to develop an FEA model of the structure. Simulations based on previous inspections and including measurements of thickness will be performed and results recorded. Simulation of the degraded model shall be based on the choice of degradation mechanism, so, for uniform corrosion, the cross-sectional area will be reduced accordingly. Geometrical modifications during the structure's life would also be considered in the model, including material properties of any structural detail replaced. In the next chapter, a description of the process of simulating the degraded FEA model will be discussed in detail.

3.2.2.1 **Stochastic Response Surface Method (SRSM)**

Offshore jacket support structures are complex three-dimensional structures, which makes the derivation of a mathematical expression capable of representing the load acting on the structure and the member's resistance capacity subjected to various forces and moments becomes extremely difficult. In such instances, the deployment of relevant enhanced computer-aided programs capable of managing the complexity of the problem is required. However, they become inefficient when required for determining extremely low values of probability of failure due to the number of iterations that may be required for convergence. Rubinstein (1981), Melcher (1999), and Broding et al. (1964) have al presented extensive analytical procedures as well as alternative methods of calculating the optimal sample numbers.

SRSM is mainly based on the approximation of the limit state function, which could be unknown in some instances, and so using an explicit mathematical function of random variables to determine the response of the system or member. The functions developed are usually in the form of a simple polynomial with coefficients that can be computed by fitting the response surface function to several sample points from the system response computation. First or Second-Order Reliability Methods can then treat the limit state function expression, and the reliability index and the probability of failure obtained.

The drawback in applying this approach is when it is applied for cases with non-linear limit state functions due to the difficulties introduced by improper presentation of response surface based on arbitrary points that might be further away from the MPP (Zhiyu et al., 2017; Cox and Baybutt, 1981; Kim and Na, 1997). To address this drawback, several methods have been suggested for the adaptation of the response surface function to a point close to the design point, which will be evaluated using FORM (Breitung and Faravelli, 1994; Bucher and Bourgund, 1990; Gupta and Manohar, 2004; Liu and Moses, 1994; Rajashekhar and Ellingwood, 1993). The accuracy of the adaption response method for non-linear functions depends on the accuracy of the adapted initial point (ABS, 2014; Olivi, 1980).

Generally, a quadratic polynomial is selected as the approximation of response surface since it requires only $2n + 1$ variables. Olivi (1980) has demonstrated the applicability of higher-order polynomials in detail. However, the limited sample point may also result in ill condition matrices for the derivation of the polynomial coefficient needed for regression (ABS, 2014; Gavin and Yau, 2008; Engelund and Rackwitz, 1992), which can also be addressed using Chanbyshev polynomials (Gavin and Yau, 2008).

3.2.3 Loading Simulation and Structural Analysis

This phase of the assessment requires the historic loading data, and the same magnitude of load and loading pattern would be applied to the FEA model. Loads of interest are those of operational loads and environmental loads. Changes in operational loads during the life of the structure should be considered in the simulations; however, if either due to loss of data or insufficient information on the operational loads, DNVGL-RP-C203 (DNV 2016) has recommended some operational load values for different applications.

For environmental loads, which mainly includes wind and wave loading, it is recommended to use 100-year and 10000-year return period wave for ultimate and accidental limit states, respectively. These could be found in the original design documents, and if not readily available, standard values from DNVGL-RP-C203 (DNV 2016) can be applied. For fatigue limit state assessment data, operational phases gathered and screened should be applied. The wave can be simulated based on a long-term wave scatter diagram for given locations (Aeran et al., 2017). Figure 17 presents an illustration of the simulation of loading and structural analysis.

3.2.3.1 Methods of Fatigue Analysis for Jacket Structures

There are mainly two methods of fatigue analysis of a jacket structure. The frequency response fatigue analysis includes the stochastic or spectral fatigue analysis and the deterministic fatigue analysis. The deterministic method would require the computation of the fatigue damage for every sea state in the wave scatter diagram to ascertain the overall fatigue damage. This method is generally recommended for jacket structures in shallow waters as these does not account for the dynamic response (DNV, 2015; DNV 2016; Lotsberg 2016).

The fatigue analysis is expected to be performed at identified critical locations based on the loading history, if available or the wave scatter diagram. Suitable wave theory should be adopted depending on the water depth and wave parameters and should be applied between 0 to 180º at 30º spacing (Aeran et al., 2017).

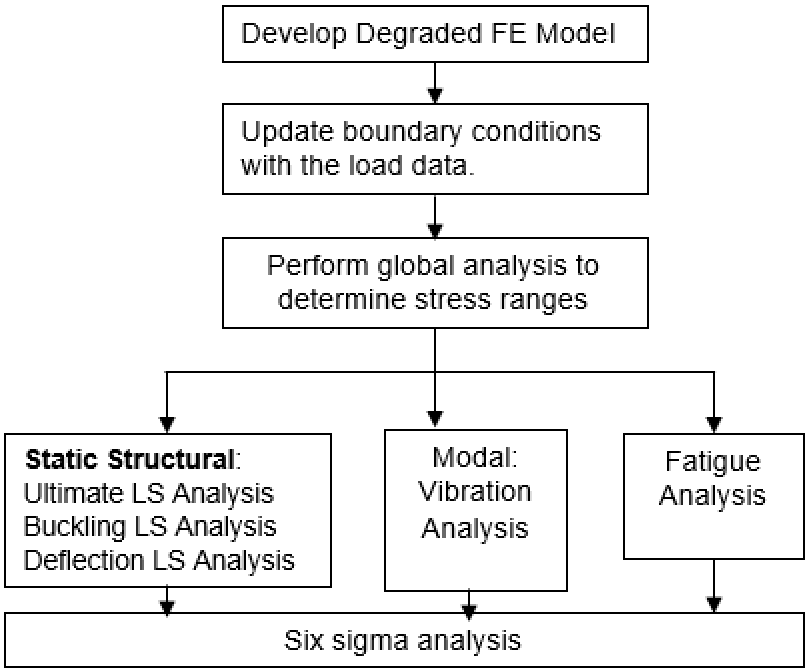

Figure 17: Simulation of loading and structural analysis

3.2.4 **Stress Analysis and Fatigue Life Prediction**

3.2.4.1 **Evaluation of Stress Ranges**

The CFD modelling was performed using ANSYS to obtain stress ranges based on the FEA and loads modelling, and the remaining life, based on fatigue analysis, can be evaluated. Stress ranges obtained from the parametric FEA fatigue analysis simulations are used to develop the performance function, which is then used to calculate the reliability index. The performance function of the fatigue limit state used in this study implies that if the number of loading cycles to failure is less than the design life cycles, the structure will experience fatigue failure**.** The fatigue life, presented as the annual reliability index, can also be deduced to present the remaining fatigue life. The analysis performed shall be for the various sea state of water, wind directions and over-turning moment.

3.2.4.2 **Selection of S-N Curve**

The choice of a suitable S-N curve is crucial in determining the fatigue life of an offshore jacket structure. The selection criteria are mainly the stress evaluation methodology, the environment, and the type of detail category. Other factors that could influence the selection are the state of deterioration of the structure, as hydrogen embrittlement and surface roughness are major factors determining fatigue strength (Aeran et al., 2017).

DNVGL-RP-C203 (DNV 2016) has presented different S-N Curve types depending on the environment and structural detail. Generally, it is advisable to use the cathodic protection (CP) S-N curve for corrosion-free structural details, while for cases with severe uniform or pitting corrosion of the structural details would require free corrosion (FC) S-N curve.

For tubular joints in the same fashion as those in the jacket structures, hot spot stress-based T curve can be selected for any environment (air, seawater with and without CP) from design standards and recommended practices such as DNVGL-RP-C203 (DNV 2016)

3.2.4.3 **Fatigue Reliability Computation**

Generally, the reliability of offshore structures can be predicted from the computation of the annual fatigue damage, which can be calculated using Miner's rule. Details of this method are presented in Mesmacque et al. (2005) and Miner (1945). The annual fatigue damage for all sea states can be computed as follows

$$D_L = \sum_{i=1}^{n_s} D_i P_i \tag{3.1}$$

where D_L is the annual fatigue damage for all sea states, D_i is the annual fatigue damage for individual sea states, P_i is the sea state probability and n_s is the number of discrete sea states in the wave scatter diagram. A DFF would be required to compute the fatigue damage, depending on the severity of the failure and the probability of inspection and repairs. An appropriate DFF can be obtained from relevant standards such as DNV-OS-J101 (DNV 2015).

Due to the limitations of Miner's rule, as mentioned earlier, the stress range obtained from ANSYS fatigue analysis is then used to calculate the number of loading cycles to failure, N, based on the S-N curve method. And the fatigue limit state can be calculated (Gentils et al., 2017; Ivanhoe et al., 2020). Details of the fatigue limit state calculation will be presented in a later chapter. The reliability index for fatigue limit state is then calculated using FORM (first-order reliability method), which allows for the calculation of low probability values. Details of the reliability assessment will be presented in the next section. The proposed framework has been applied to the NREL 5MW OWT OC4 jacket to assess the reliability of critical components of the structure

3.2.5 **Assessment of Limit States**

If the fatigue limit state result is satisfactory, it is recommended to check other limit states further. According to DNV-OS-J101 (DNV 2015), which is the most widely applied standard in the design of offshore wind turbines, limit states to be considered can be categorised into three groups, i.e., ULS, FLS and SLS. The ULS accounts for the failure due to the loss of structural resistance (such as excessive yielding and buckling). The excessive deflection and vibration will affect the serviceability of the structure, and they can be categorised as SLS. To this end, the five limit states considered in this study are: buckling (ULS), ultimate (yielding stress, ULS), fatigue (FLS), frequency (SLS) and deflection (SLS).

These can be characterised as design constraints/limiting factors, but in multiple references, the term limit state is used. Although certain limit states have a higher influence on different types of offshore support structures, this thesis considers it relevant to include all five. With all limit states checked and returned satisfactory for both future and current degradation, the life extension of the structure can then be proposed.

It is highly recommended as part of the life extension of an ageing OWT jacket support structure, a robust inspection and maintenance plan should be developed for the structure during its extended life based on appropriate standards (DNV 2015; DNV 2016; NORSOK 2015). Condition monitoring plan for the structure should be updated to include an increased likelihood of fatigue cracks due to stress accumulation during its original design life. See NORSOK N-006 (NORSOK 2015) for more details on condition monitoring steps.

Also, for unsatisfactory limit state cases, appropriate mitigations would need to be recommended. These can comprise strengthening, changes in operation, change in the application, etc. Strengthening mechanisms such as adding or replacing extra braces to over-stressed sites or adding brackets or stiffeners, grouting of members and joints, and other technics such as welding can also be used. It is also important to assess the feasibility of the site where the proposed life extension is to be conducted before the commencement of any strengthening of the structure. Mitigations should be in place in case the proposed structural strengthening is not feasible. If there are no alternatives, it is highly recommended that the structure should not be used any further.

3.3 Numerical Methods

In the previous chapter, the concept and context of structural reliability index and analysis were discussed. In this chapter, reliability analysis, approaches will be discussed in detail. There are several approaches to evaluating the reliability of a component or system, provided its safety margin (limit state function) is known. Every known approach has its merits and demerits. The most applicable and widely acceptable approaches would be discussed. Based on their merits and applicability, the most appropriate method of evaluating the reliability index of an offshore wind turbine support structure would be adopted for this thesis. At every phase of the work, an algorithm would be developed to show the component reliability analysis of the structure.

This analysis will focus on developing a generic reliability-based methodology for evaluating the fatigue life of offshore fixed structures. The study will focus on the jacket support structure supporting a wind turbine will be analysed based on the S-N approach and on the assumption that the fatigue damage accumulation follows Miner's rule. As discussed earlier, this approach is based on a fundamental structural reliability theorem applicable in either a load and resistance factor design (LRFD) or a direct reliability-based design. The design method requirements include nominal load and stress values, conventional design codes, and target reliability levels. The FORM/SORM will be used for estimating the reliability index. The analytical procedures presented in this section will be applied in later parts of this thesis. The analytical procedures comprise both probabilistic and deterministic approaches. The utilization of appropriate engineering techniques can be vital in solving complex engineering problems.

3.3.1 Deterministic Methods

First-order reliability methods are one of the most widely used and efficient methods of evaluating the reliability of a component (Holicky, 2009). These methods are based on an earlier second-moment method developed by Cornell (Cornell, 1969). The second-moment method uses the derivatives of the first and second moment of the variable to evaluate the reliability. The variables (Stochastic) are categorised by their moments. First-Order Second Moment Reliability Method (FOSM) will be presented as a basis for the FORM/SORM. Detailed analytical explanations of those approaches can be found in (Choi et al., 2006).

The introduction of multiple variables in determining the probability of failure has resulted in the development of several approaches that aim to simplify the process of calculations. Taylor's and first and second-order expansion series have been a standard mathematical method of linearizing non-linear equations and, in this case, the limit state. The FOSM is generally regarded as the mean value first-order second-moment method (MVFOSM), which is a simplified approach for determining the failure probability. However, its drawback is its inability to accurately estimate the failure probability for nonlinear limit state functions (Society of Automotive Engineers, 1997). The addition of a second term in the SOSM approach still does not adequately give accurate estimations.

A geometrical solution, the safety index approach, can be used to adequately manage the drawbacks. This approach transforms the problem into a mathematical optimization problem to locate the point at which the limit state surface is closest to the origin of the standard normal space. Hasofer-Lind (1974) introduces an algorithm that transforms the design to stochastic variables vector X to a standardized independent variable vector U see Figure 18

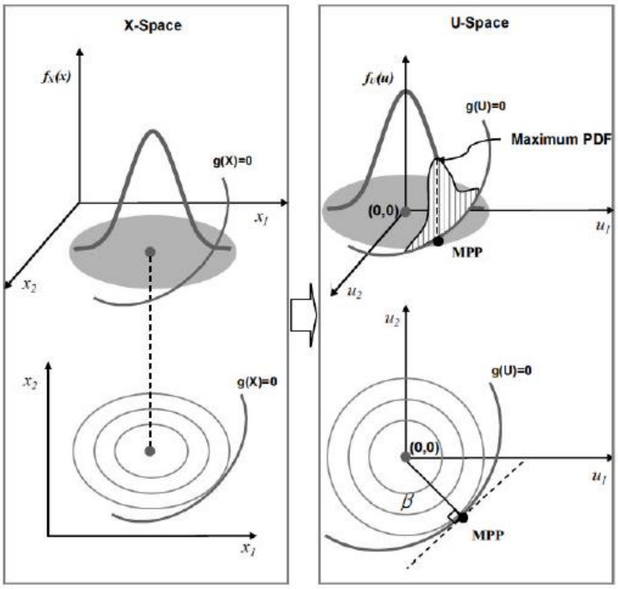

Figure 18: Transform to the U-space (Choi et al., 2004)

The design point in the U-space characterizes the point of the highest probability density and is referred to as the Most Probable failure Point (MPP). The transformed limit state surface $g(U) = 0$ can then be treated using first or second-order approximations, thus accounting for the FORM/SORM, respectively. Summarily, in using the approach, the limit state surface approximated either at the tangent of a curve at the MPP for FORM and SORM, respectively. See Figure 19 because FORM provides inaccurate results for nonlinear limit state surface large circumference.

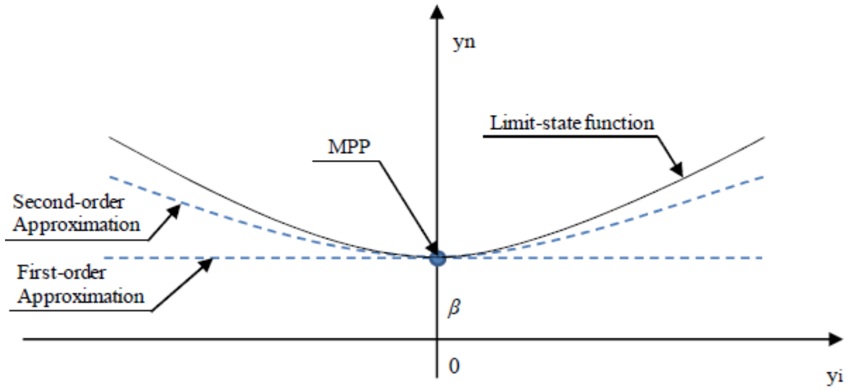

Figure 19: FORM/SORM approximations (Choi et al., 2006)

3.3.1.1 **First Order Reliability Methods**

3.3.1.1.1 Mean Value FOSM (MVFOSM)

MVFOSM is a simplified means of calculating the probability of failure for a given limit state function. This approach approximates the limit state function using the Taylor series expansion at the mean value point. The first order derivatives obtained from the Taylor series expansion is a linearized model of the initially given performance function. Its parameters are expressed in terms of their mean and standard deviation. This simplified approach does not apply to higher moments and thus increases the model's uncertainty level. Assuming a vector X that is statistically independent, the approximate limit state function at the mean value point can be given as:

$$\tilde{g}(X) \approx g(\mu_X) + \nabla g(\mu_X)^T \cdot \left(X_i - \mu_{x_i}\right) \tag{3.2}$$

where the vector of the mean values is given as $\mu_X = \left\{\mu_{X_1}, \mu_{X_2}, \ldots \mu_{X_n}\right\}^T$, and the gradient g is evaluated at μ_X as:

$$\boldsymbol{\nabla} g(\mu_X) = \left\{\frac{\delta g(\mu_X)}{\delta x_1}, \frac{\delta g \mu_X}{\delta x_2}, \cdots, \frac{\delta g(\mu_X)}{\delta x_n}\right\}^T \tag{3.3}$$

The expected mean value of the approximated limit state function is given as:

$$\mu_{\tilde{g}} \approx E[g(\mu_X)] = g(\mu_X) \tag{3.4}$$

From basic statistical transformations, the standard deviation of the limit state can be deduced as:

$$\sigma_{\tilde{g}} = \sqrt{Var[\tilde{g}\,(X)]} = \sqrt{[\nabla g(\mu_X)^T]^2 \cdot Var(X)} \tag{3.5}$$

$$\sigma_{\tilde{g}} = \left[\sum_{i=1}^{n} \left(\frac{\delta g(\mu_X)}{\delta x_i}\right)^2 \cdot \sigma_{x_i}^2\right]^{\frac{1}{2}} \tag{3.5a}$$

The reliability index can then be calculated as:

$$\boldsymbol{\beta} = \frac{\mu_{\tilde{g}}}{\sigma_{\tilde{g}}} \approx \frac{g(\mu_X)}{\left[\sum_{i=1}^{n} \left(\frac{\delta g(\mu_X)}{\delta x_i}\right)^2 \cdot \sigma_{x_i}^2\right]^{\frac{1}{2}}} \tag{3.6}$$

For linear limit state functions, the reliability index can be deduced analytically by expressing the safety margin between the load (S) and resistance (R) as a deterministic variable such as:

$$\boldsymbol{g}(X) = R(X) - S(X) \tag{3.7}$$

Consequently, the resultant mean value μ_g and standard deviation σ_g are given as:

$$\mu_g = \mu_R - \mu_S \tag{3.8}$$

$$\sigma_g = \sqrt{\sigma_R^2 + \sigma_S^2 - 2 \cdot \rho_{RS} \cdot \sigma_R \cdot \sigma_S} \tag{3.9}$$

where, μ_R, μ_S, σ_R , σ_S are the mean values and standard deviation of the resistance and the load, respectively, and ρ_{RS} is the correlation coefficient between R and S. The reliability index can be re-written as:

$$\boldsymbol{\beta} = \frac{\mu_{\tilde{g}}}{\sigma_{\tilde{g}}} = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2 - 2 \cdot \rho_{RS} \cdot \sigma_R \cdot \sigma_S}} \tag{3.10}$$

When the load and resistance are not correlated, the reliability index is:

$$\boldsymbol{\beta} = \frac{\mu_g}{\sigma_g} = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} \tag{3.11}$$

However, for non-linear performance functions, the limit state function can be obtained by linearizing the performance function at the mean value point. Generally, for multiple independent variables, the failure surface is represented by a hyperplane defined as a linear failure function is given as:

$$\tilde{g}(X) = C_0 + \sum_{i=1}^{n} C_i \cdot x_i \tag{3.12}$$

$$\mu_{\tilde{g}} = C_0 + C_1\mu_{x_1} + C_2\mu_{x_2} + \cdots + C_n\mu_{x_n} \tag{3.13}$$

$$\sigma_{\tilde{g}} = \sqrt{\sum_{i=1}^{n} C_1^2 \cdot \sigma_{x_1}^2} \tag{3.14}$$

The MVFOSM approach is a simplified and crude method for calculating the reliability index, with a minimum representation of basic variables. It is also impracticable to have all design variables normally distributed. In most instances, the stochastic variables are correlated, which implies that this method won’t be able to provide credible reliability analysis. Furthermore, this method relies on the form of the limit state function, as it is more efficient in analysing linear limit state functions. For nonlinear limit state functions, the results obtained are unrealistic reliability indices. For instance, different reliability index reports are obtained for limit state function expressed as $R - S < 0$ instead of $\frac{R}{S} < 0$ (Haldar and Mahadevan, 2000). Due to these limitations, there is a need for an enhanced method to overcome these drawbacks.

3.3.1.1.2 Hasofer-Lind Reliability Index

The Hasofer-Lind method was introduced in 1974 (Hasofer-Lind, 1974). This method was meant to address the drawbacks of using the FORM. This method approaches structural reliability from a geometrical perspective. Here the reliability index is defined by the closest distance from the origin of a u-

dimensional space to the Most Probable failure Point (MPP) on the failure surface. The Hasofer-Lind method transforms the expansion point from the mean value to the MPP, which is an improvement of the MVFOSM approach. For multiple variable scenarios, this approach proposes a linear transformation of the basic variables x_i into a set of normalized and independent variables u_i.

For a simple normally distributed two independent variable cases, with Load S and resistance R Hasofer-Lind approach transforms the initial variables to a standard normalized variable and is given as:

$$\hat{R} = \frac{R - \mu_R}{\sigma_R}, \qquad \hat{S} = \frac{S - \mu_S}{\sigma_S} \tag{3.15}$$

Consequently, the limit state surface $g(R, S) = R - S = 0$ can also be transformed from (R, S) coordinate system to the limit state surface, in the standard normalized $(\hat{R}, \hat{S})$ a coordinate system as shown below:

$$g\left(R(\hat{R}), S(\hat{S})\right) = \hat{g}(\hat{R}, \hat{S}) = \hat{R} \cdot \sigma_R - S \cdot \sigma_S + (\mu_R - \mu_S) = 0 \tag{3.16}$$

The reliability index can then be calculated by the distance from the origin in the $(\hat{R}, \hat{S})$ coordinate system to the failure surface $\hat{g}(\hat{R}, \hat{S}) = 0$.

$$\beta = \widehat{OP}^* = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} \tag{3.17}$$

This implies that point $P^*(\hat{R}^*, \hat{S}^*)$ on $\hat{g}(\hat{R}, \hat{S}) = 0$ is the MPP, or otherwise the closest distance to the origin. Therefore, for n independent normally distributed variables, the failure surface would be defined by a nonlinear limit state function such as:

$$g(X) = g(\{x_1, x_2, \cdots, x_n\}^T) = 0 \tag{3.18}$$

And the variable transformation into their respective standardized forms is given as:

$$u_i = \frac{x_i - \mu_{x_i}}{\sigma_{x_i}} \tag{3.19}$$

Consequently, $g(X) = 0$, (failure surface) in the X-space is mapped into its corresponding failure surface $g(U) = 0$ in the U-space. Based on the circumferential symmetry of the second-moment representation of $U,$ the distance from the origin in the U-space to any point on the $g(U) = 0$ is the number of standard deviations from the mean value point in the X-space to correspond on the $g(X) = 0$. The reliability index can then be given as:

$$\beta = \min_{U\epsilon g(U)=0} (U^T \cdot U)^{\frac{1}{2}} \tag{3.20}$$

The reliability index obtained from this approach is called the Hasofer-Lind reliability index (β_{HL}). The points in the U-space are the design point, which has corresponding vector points in the X-space.

There are several available procedures used for solving these problems, either iterations or mathematical optimization. Freudenthal et al. (1966) present several constrained methods that were used to solve the optimization problem, such as the primordial method, Lagrange multiplier and dual methods (Choi, 2006). The applicability of this method relies on the nature of the problem of interest.

The Hasofer-Lind method was aimed at allowing normally distributed stochastic variables. At the same time, Rackwitz and Fiessler did an extension of the Hasofer-Lind method to account for non-Gaussian statistical distributions, which gives rise to the extended Hasofer-Lind method otherwise called the Hasofer-Lind – Rackwitz Fiessler (HL-RF) method. For instance, let the limit state surface (linear or nonlinear) with n independent normally distributed random variables X, can be expressed as:

$$g(X) = g(\{x_1, x_2, \cdots, x_n\}^T) = 0 \tag{3.21}$$

According to the Hasofer-Lind transformation, the equation (3.21) can be re-written as:

$$g(U) = g\left(\left\{\sigma_{x_1}u_1 + \mu_{x_1}, \sigma_{x_2}u_2 + \mu_{x_2}, \cdots, \sigma_{x_n}u_n + \mu_{x_n}\right\}^T\right) = 0 \tag{3.22}$$

The vector from the origin to the limit state surface $g(U)$ creates an intersection point P^*. The reliability index can be derived from the first-order Taylors expansion of $g(U)$ at MPP (P^*) and is given as:

$$\tilde{g}(U) \approx g(U^*) + \sum_{i=1}^{n} \frac{\delta g(U^*)}{\delta U_i} \cdot (u_i - u_i^*) \tag{3.23}$$

This can further be reduced to:

$$\frac{\delta \hat{g}(U)}{\delta U_i} = \frac{\delta g(X)}{\delta x_i} \cdot \sigma_{x_i} \tag{3.24}$$

The closest distance from the origin $\hat{O}$, to the MPP at $\tilde{g}(U)$ the surface is given as:

$$\hat{O}P^* = \beta = \frac{g(U^*) + \sum_{i=1}^{n} \frac{\delta g(U^*)}{\delta U_i} \cdot \sigma_{x_i} \cdot u_i^*}{\sqrt{\sum_{i=1}^{n} \left(\frac{\delta g(U^*)}{\delta x_i} \cdot \sigma_{x_i}\right)^2}} \tag{3.25}$$

The sensitivity factor, otherwise known as directional cosine of each of the transformed variables, is given as:

$$a_i = \cos\theta_{u1} = \cos\theta_{u1} = -\frac{\frac{\partial g(U^*)}{\partial u_i}}{|\nabla g(U^*)|} = \frac{\frac{\partial g(X^*)}{\partial x_i} \cdot \sigma_{x_i}}{\left[\sum_{i=1}^{n} \left(\frac{\delta g(U^*)}{\delta x_i} \cdot \sigma_{x_i}\right)^2\right]^{\frac{1}{2}}} \tag{3.26}$$

The coordinates of the point P^* are computed as such:

$$u_i^* = \frac{x_i^* - \mu_{x_i}}{\sigma_{x_i}} = \hat{O}P^* \cos\theta_{x_1} = \beta \cos\theta_{x_1} \tag{3.27}$$

Where the subscripts and asterisk imply, the expression is evaluated at the design point. The design points in the original space can be deduced from the expression below:

$$x_i^* = \mu_{x_i} + \beta\sigma_{x_i} \cos\theta_{x_1} \ ,(i = 1, 2, \cdots, n) \tag{3.28}$$

Since P^* is a point at the limit state surface, it should satisfy:

$$g(\{x_1^*, x_2^*, \cdots, x_n^*\}^T) = 0 \tag{3.29}$$

For multiple MPP problems where several points are corresponding to stationary values on the failure surface, it may be helpful to use various start points to find all the reliability index values driving the overall HL reliability index as:

$$\beta_{HL} = min\{\beta_1, \beta_2, \cdots, \beta_m\} \tag{3.30}$$

The probability of failure can further be obtained from the calculated reliability index, using the expression below:

$$P_f = 1 - \Phi(\beta) \tag{3.31}$$

Where $\Phi(\beta)$ represents the cumulative distribution function of a normal distribution.

Summarily, the underlying difference between the Hasofer-Lind method and the MVFOSM is that the Hasofer-Lind method uses an iterative method to obtain the reliability index at the convergence point, particularly for nonlinear functions. At the same time, MVFOSM is a simple straight calculation. Also, the Hasofer-Lind method approximates the limit state function using Taylor's first-order expansion at the design point $X^{(k)}\ or\ U^k$ rather than the mean value point μ_X (Choi, 2006). The calculations performed in obtaining the reliability index in Hasofer-Lind are based on coordinates of the design point, which are unknown at the beginning.

This means the process starts with an initial guess, and then the values are changed for each iteration till the output values converge based on a given convergence criterion. Also, a significant drawback of using this approach is the inability of this method to provide credible results for non-normal or Gaussian statistical distribution or in cases of non-linear ar limit state function (Haldar and Mahadevan, 2000). HL-RF method proposes a process of getting the design point.

A flow chart of Hasofer-Lind method of reliability index calculations is presented in Figure 20.

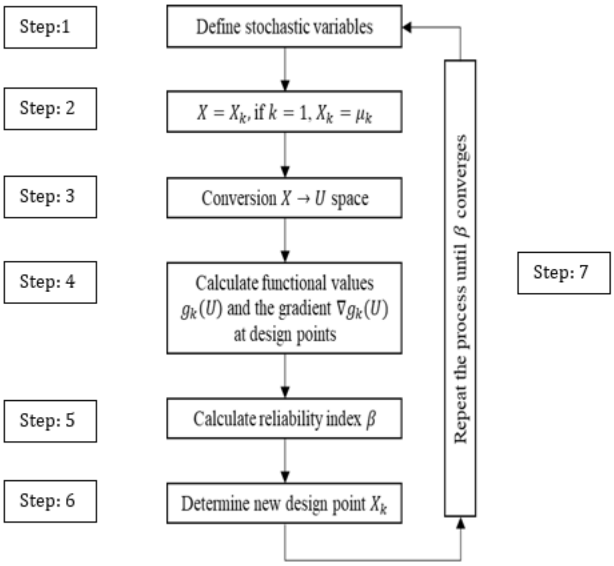

Figure 20: Hasofer-Lind reliability index algorithm

3.3.1.1.3 Hasofer Lind – Rackwitz Fiessler (HL-RF) Method

Rackwitz-Fiessler method is a known efficient method of calculating structural reliability index for non-Gaussian variables (Holicky, 2009). In the previous method described earlier, the stochastic variable X are assumed to be normally distributed. As such, the expressions were insensitive to the type of distribution. Many structural reliability problems involve random variables that are not necessarily normally distributed. Rackwitz-Fiessler method, however, takes the statistical distribution of the stochastic variables into account. So, for non-Gaussian variables, an equivalent normal distribution is calculated, which makes the computed reliability index sensitive to the statistical distribution of the stochastic variable.

There are several approaches to performing the transformation of the non-Gaussian system to the normalized space in the literature (Hohenbichler and Rackwitx, 1981; Rosenbaltt, 1952). For non-linear statistical variables, the equivalent normal mean and standard deviation can be derived by employing the normal tail approximation method.

Consider a mutually independent variable, with the transformation expressed as:

$$u_i = \Phi^{-1}[F_{x_i}(x_i)] \tag{3.32}$$

Using Taylors series expansion to transform the expression at the MPP X^*, ignoring the non-linear terms (Merchelers, 1987), would result in the equivalent normal distribution and is giving as:

$$u_i = \Phi^{-1}\left[F_{x_i}(x_i)\right] + \frac{\delta}{\delta_{x_i}}(\Phi^{-1}\left[F_{x_i}(x_i)\right])|{x_i}^* \cdot (x_i - x_i^*) \tag{3.33}$$

where,

$$\frac{\delta}{\delta_{x_i}}(\Phi^{-1}\left[F_{x_i}(x_i)\right] = \frac{f_{x_i}(x_i)}{\varphi(\Phi^{-1}\left[F_{x_i}(x_i)\right])}$$

After substitution, the equation can be rewritten as:

$$u_i = \frac{x_i - [x_i^* - \Phi^{-1}\left[F_{x_i}(x_i)\right]\varphi(\Phi^{-1}\left[F_{x_i}(x_i)\right])/f_{x_i}(x_i^*)]}{\varphi(\Phi^{-1}\left[F_{x_i}(x_i)\right]/f_{x_i}(x_i^*)} = \frac{x_i - \mu_{x'_i}}{\sigma_{x'_i}} \tag{3.34}$$

where, $f_{x_i}(x_i)$, is the probability density function, $F_{x_i}(x_i)$ is the marginal cumulative distribution function and $\sigma_{x'_i}$ and $\mu_{x'_i}$are the equivalent standard deviation and means of the approximately normal distribution, respectively, which can be computed as:

$$\mu_{x'_i} = x'_i - \Phi^{-1}\left[F_{x_i}(x_i)\right] \cdot \sigma_{x'_i} \tag{3.35}$$

And,

$$\sigma_{x'_i} = \frac{\varphi(\boldsymbol{\Phi}^{-1}\left[F_{x_i}(x_i)\right])}{f_{x_i}(x_i)} \tag{3.36}$$

Alternatively, the equivalent normal distribution can be obtained by matching the cumulative distribution functions and the original, non-Gaussian variables' probability density function and the approximate normal random variable at the MPP (Rosenbaltt, 1952). The normalized-tail approximation is presented in Figure 21. The described approach can easily be used to transform random variables from the X-space to the U-Space.

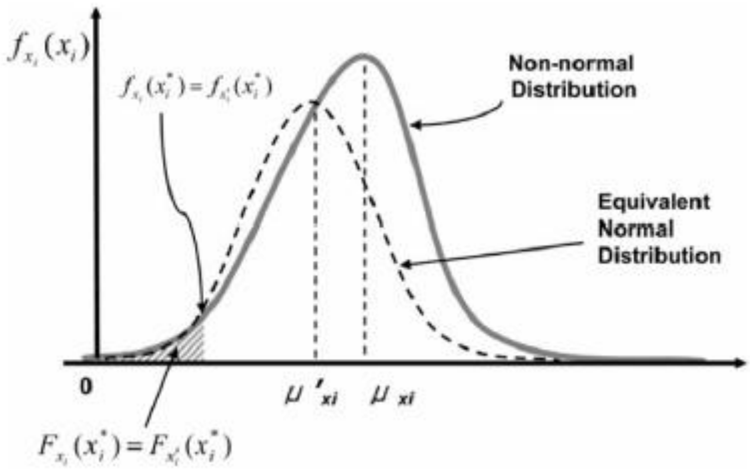

Figure 21: Normalized Tail Approximation (Choi et al., 2006)

This method is like Hasofer-Lind. As described in previous sections with the calculated normal distribution random variables, the derivatives for the design points are calculated. However, a large structure that involves implicit limit state function could require a more sophisticated computerised program using FORM. The directional cosines are then computed based on normally distributed statistical variables, and the iteration for the reliability index is performed until the reliability index converges.

3.3.1.2 Second Order Reliability Method (SORM)

SORM, unlike FORM approximations which applies only to instances when the surface of the limit state has a single minimal distance point and the function is very close to the design point. However, for large or irregular curvatures, estimation of the probability of failure using FORM may be inaccurate (Merchelers, 1987). Therefore, adopting second-order Taylor‘s series expansion would produce a more reliable result.

Several non-linear approximations approaches are available in the literature, such as the orthogonal, first and second order approximations, that have been used to simplify the original surfaces. Although the SORM can provide a relatively more reliable result for a wide range of design limit functions, it requires substantial computational resources and experience to compute the derivatives of $\boldsymbol{\beta}$**.** Particularly for multiple variable cases that increase the size of the matrix as the variables increase and become even more complex and extensive, yielding an appropriate choice of calculation method. This thesis would also limit its analysis to FORM, so further details of SORM are not covered in this work.

3.3.2 Simulation and Sampling Methods

The previous FORM was discussed in detail. FORM is mainly applicable for problems with explicit limit state function. Alternatively, the probability of failure and reliability index can also be calculated by using simulation methods. Simulation methods are applicable for both implicit and explicit limit state functions. Simulation methods employ several random samples generated to represent their actual probability distributions and stochastic characteristics. Utilizing the random variables in the problem and evaluating it deterministically will result in a realisation of the problem itself. Huge simulation trials, which are individual simulations, will give the overall stochastic characteristics of the problem, especially with high simulation trials (Haldar and Mahadevan, 2000).

For instance, assume the limit state function for a beam with respect to its moment-bearing capacity is formulated with a performance function of two random variables. The simulation method can then generate several random samples for each variable according to their probability distributions and deterministically evaluate the performance function for each realisation of both random variables. This will result in obtaining stochastic characteristics of the limit state, which is then used to calculate the probability of failure and the reliability index. MCS, including importance sampling and Latin Hypercube Simulation, will be presented as sampling methods in the following sections.

3.3.2.1 **Monte Carlo Simulation (MCS)**

The MCS is arguably the most common method of simulation. MCS generates random samples giving to the probability distributions of the basic random variables, which are treated as inputs. These inputs are used for the deterministic analyses of the limit state function. The results obtained are evaluated to obtain the reliability index and the probability of failure. This tool has witnessed a significant transformation during the last decade. This approach is like random experiments, for which the specific result is not known in advance. In this context, MCS shall be considered as a methodical means of doing so-called what-if analysis.

In calculating the probability of failure using simulation methods, failure is defined as when the magnitude of the limit state function is lesser than zero. So, a binary vector can be formulated for each calculation of the performance function. If the calculated limit state function is less than 0 (failure), the corresponding value in the binary vector is 1 and 0 otherwise. This is presented in equation (3.37). The summation of the binary vector components will give the total number of failures for the limit state function. Thus, If N_f is the number of failures (total number of times when g < 0 due to the substitution of sample values into the limit state function), and n is the total number of simulations, an approximation of the failure probability can be calculated as thus:

$$I(i) = \begin{cases} 0 & if \quad G(x) \geq 0 \\ 1 & if \quad G(x) < 0 \end{cases} \tag{3.37}$$

$$P_f = \frac{N_f}{N} \tag{3.38}$$

While random variables of a stochastic distribution can be generated based on the inverse transform method. For example, consider F_x as the cumulative distribution function of a stochastic variable x_i with possible values ranging from [0,1]. Let v_i be the randomly generated variables that follow a normal uniform distribution. This method can correlate v_i and x_i as follows:

$$F_x x_i = v_i \rightarrow x_i = F_X^{-1}(v_i) \quad (3.39)$$

The inverse transformation method can be illustrated further in Figure 22.

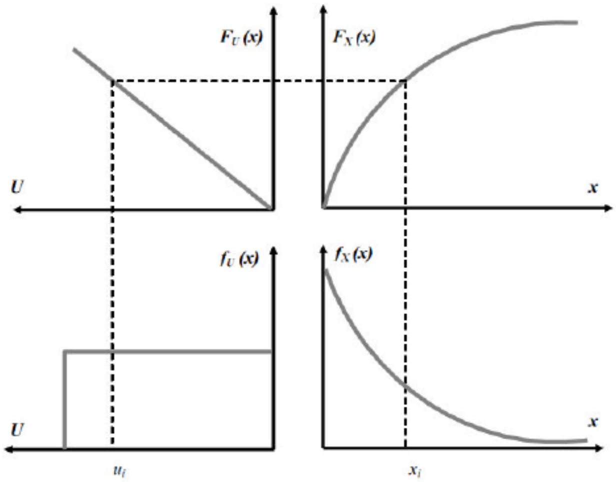

Figure 22: **Inverse Transformation (ABS, 2014)**

3.3.2.2 **Latin Hypercube Method**

Latin Hypercube sampling method is a simulation-based reliability evaluation method. The domain of each random variable is decomposed into intervals, that are assigned the same probability. The number of intervals depends on how many samples would be generated for each variable. One value from each interval is selected at random for the probability density in the interval. To form the hypercube combination of the interval, a random variable is performed. When sampling a function of N variables, the range of each variable is divided into M equally probable intervals. M sample points are then placed to satisfy the Latin hypercube requirements; this forces the number of divisions M, to be equal for each variable. The maximum number of combinations for a Latin Hypercube of M divisions and N variables can be computed with the relation as follows:

$$\left(\prod_{n=0}^{M-1} M - n\right)^{N-1} = (M!)^{N-1} \tag{3.40}$$

This methodology can further be illustrated in Figure 23. The random variables X1 and X2 are divided into nine equal intervals with the same probability. For each interval, there is a value taken randomly. Notably, for an interval of X1, there is only one value in X2. For these two variables to be matched using this method, we can get a combination 181440 of samples. The calculation P_f follows the same analogy as it was with the MCS method. The governing equation is given as:

$$P_f = \frac{1}{N}\sum_{k=1}^{N} I_g(x_k) \tag{3.41}$$

where I_g is an indicator and x_K is the samples space

The number of required samples for simulation is independent of the dimension of the problem or the number of random variables. For a confidence level of 95%, the number of required samples can be calculated based on the expression below:

$$N_x = \frac{10}{P_f} \tag{3.42}$$

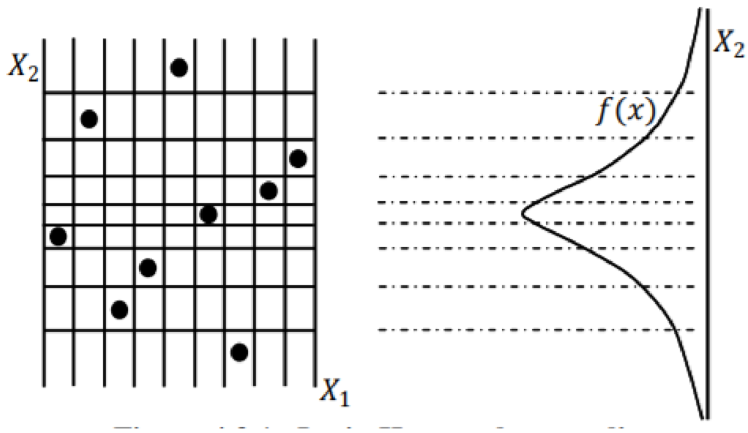

Figure 23: Latin Hypercube sampling (ABS, 2014)

3.4 Regression Analysis

There are several regression analysis methods, such as the central composite design method (Myers, 1971), kriging and linear (multivariable) regression method. The central composite design method is mainly applied for a higher quality of response surface design, as it permits redundancy in some of the data sets. However, it requires a large number of data sets ($(N = 2^n + 2n + 1)$ for its analysis to approach the exact performance function with a quadratic function. This limitation makes it impossible to use this method for cases that involve a smaller probability of failure, like determining the probability of failure of members of the jacket structure.

On the other hand, Kriging is a geostatistical regression method that takes into account both the degree of variance and distance between known data points when estimating the true values of unknown locations (Toal et al., 2008). Geologists originally utilized this method has found its usefulness in other fields of study, the extensive theoretical background is contained in (Forrester et al., 2008; Sobey et al. 2010).

3.4.1 Linear Regression

Regression analysis is a statistical tool used to examine the relationship between two or more variables of interest. It could also be used to predict correlated variables as well as understanding the impact of independent variables on the dependent variable. Linear regression refers to the measure or extent to which a linear relationship exists between the dependent and independent variables. It can also be used to predict the value of the dependent variable based on the value and correlation of the independent variable. Gauss and Legendere proposed the Least Square Method (LSM) of performing regression analysis with minimal distance between the data provided and the potential function (residuals) to obtain the optimum fit. LSM can be expressed mathematically as:

$$y(x) = a_0 + a_1 \cdot f_1(x) + a_2 \cdot f_2(x) + \cdots + a_n \cdot f_n(x) + e \tag{3.43}$$

Where $a_0, a_1 \ldots a_n$ are constants regression coefficient, and e is the error of the equation. (3.43) can be represented as a matrix as:

$$Y = X \cdot \alpha + e \tag{4.44}$$

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

The LSM can further be expressed in a matrix format in order of the regression coefficients vector a, as follows:

$$a = (X^T \cdot X)^{-1} \cdot X^T . Y \tag{3.45}$$

Following the computation of the regression coefficient, the values of the sample dependent variables and error for can be obtained by:

$$\bar{Y} = X \cdot a \qquad e = Y - \bar{Y} \tag{3.46}$$

The regression, total and error sum of the square can then be computed from:

$$\begin{gathered} SSR = \overline{Y^T} \cdot \bar{Y} = a^T \cdot X^T \cdot Y \\ SST = Y^T \cdot Y \\ (SSE = SST - SSR \end{gathered} \tag{3.47}$$

Validation of the analysis, a coefficient of determination (R^2) can be computed as follows. This implies that the modelled function is deemed valid when the sum of the regression errors equals zero and $R^2 = 1$. The coefficient of determination can be obtained from:

$$R^2 = 1 - \frac{SSE}{SST} \tag{3.48}$$

3.4.2 **Multivariate Regression**

Multivariate regression is a regression method that estimates a single regression model with more than one outcome variable. The availability of adequate can analyse the fundamental equation (y, x_i) data set. The general statement of the problem can be described as:

$$y(x) = \sum_{i} a_i \cdot p_i(x_1, x_2, \cdots, x_n) + e \tag{3.49}$$

Where a_i is the regression coefficient, and for a second-degree polynomial with 2-independent variables and, this expression can be written as:

$$y(x) = a_0 + a_1 \cdot x_1 + a_2 \cdot x_2 + a_3 \cdot x_1^2 + a_4 \cdot x_2^2 + a_5 \cdot x_1 \cdot x_2 + e \tag{3.50}$$

Consider a dependent variable Y as a $(n\ X\ q)$ matrix and an independent variable X containing $(n\ X\ p)$ matrix with a regression coefficient A containing $(p\ X\ q)$ matrix and the error term E containing $(n\ X\ q)$ matrix. The correlation for regression analysis can be written as:

$$Y = \tilde{X} \cdot A + E \tag{3.51}$$

where $\tilde{X}$ represents a matrix formed from X, containing the various power values of X.

To solve the above equation, the matrix $(p\ X\ q)$ set of data must be known, and the accuracy of the regression coefficients result is dependent on how well conditioned the $X^T \cdot X$.

Notably, despite the simplicity of direct MCS, it could be exposed to significant levels of error if the number of simulations is not large enough. There are some methods to determine the required minimum number of simulations to attain an acceptable level of accuracy. One such method is to determine the accuracy of an MCS is to approximate a binomial probability distribution with a normal distribution and make an estimation of about 95% confidence interval of the calculated failure probability. It will lead to Equation 3.40 (Haldar and Mahadevan, 2000).

$$e\% = \sqrt{\frac{1 - P_f^T}{N\ \times P_f^T}} \times 200\% \tag{3.52}$$

Where N is the number of simulations and P_f^T is the true probability of failure.

Certainly, the probability of failure is not known prior. Still, suppose the objective is to confirm the results of other reliability assessment methods such as FORM. In that case, this formula can be helpful in determining the number of necessary simulations since already an estimation of the failure probability is available. Usually, in engineering problems, it is recommended to have k million numbers of simulation with k being the number of basic random variables (Haldar and Mahadevan, 2000).

3.5 Discussion

This chapter has presented a proposed reliability assessment methodology for the analysis of ageing OWT structures. The proposed approach is based on stochastic and deterministic approaches, defining the statistical distributions and the means and standard deviation respectively when determining the reliability index. Most importantly, the proposed method aims to close the gap of producing under or over design structures without a view of the economic viability. Therefore, the proposed methodology will provide not just a dogged structure but a cost-effective, reliable support structure with a 50 – 100 years MRP and a design life of 20 years with little or no interventions.

Due to over-design, the conventional structural reliability method has produced several dogged offshore structures that are probably still in use after the projected MRP. Some others have failed even before the end of the design life due to under design. The sampling method used in obtaining the load and the capacity data of the structure does not correctly account for uncertainties. However, the use of RSM of sampling in the proposed methodology better accounts for uncertainties and can easily be applied to a wide design process of structural details of the wind turbine. Ideally, if the structural elements are designed based on consistency reliability level, then the failure rates of the OWT is expected to decrease consequently.

However, analysing the reliability of a non-linear limit state using the proposed methodology remains a significant drawback due to the difficulties of improper presentation of the response surface based on arbitrary points that are further from the most probable point of failure. To mitigate this drawback, the SORM approach can be replaced by FORM and the reliability index calculated. Also, the RSM-FORM requires advanced data management skills to formulate the performance function. Despite its limitations in managing non-linear limit states, RSM-FORM better captures the response of the structure. Given its flexible sampling technique, it can be easily adapted for optimized design at a low cost, given that it requires less time to compute. RSM-FORM can also be deployed in situ for the assessment of the reliability index of a specific component of the structure without changing the configuration of the global assessment.

Conventional methods have been used in analysing the non-linear limit state functions. However, they lack the flexibility to allow manipulation in the sampling technique without recourse to altering the FEA model. Conventional methods like MCS has been a handy tool used by others for sensitivity analysis and the analysis of multiple component systems. However, this conventional method does not have the flexibility for in-situ sampling without having to redefine the sample space. Given its computational complexities (such as longer computational time and difficulties in error traceability), the proposed methodology is preferable.

Moreover, the accuracy of any methods used largely depends on the quality of the data available, e.g. measurements resulting from the methods used to detect and accurately measure the sizes of the defects in the structure. Thus, if defects are outside the measurement range of the tools used to collect data from the structure, the calculated reliability would not be correct.

Furthermore, numerical methods used for the analysis of structural reliability include linear and non-linear methods and deterministic methods, including the First and Second Order reliability methods from their respective first principles, which was also the basis for the formulation of codes. Simulation and sampling methods were briefly discussed as well as the Stochastic Response Surface Method (SRSM) derivations. Data post-processing methods such as regression analysis were discussed with emphasis on Multivariate Polynomial Regression as it was required for the transformation of stochastic variables of the simulation output to performance function, which is needed for the calculation of reliability index.

4 MODELLING OF LOADS AND CAPACITY OF OWT JACKET SUPPORT STRUCTURES

4.1 Introduction

This chapter aims to review the general stochastic modelling of environmental loading and the load capacity of the structure. FEA modelling and the computation of the reliability index, using (i) conventional direct simulation methods such as MCS and (ii) the proposed Response Surface Method combined with the First Order Reliability Method, are compared. The application of the proposed methodology and the MCS of a hypothetical jacket structure has provided valuable results and conclusions.

Generally, the analysis of the design variables is essential and should be based on well-informed engineering judgment and should be robust enough to account for uncertainties such as extreme conditions the structure might encounter during its service life.

Research has shown that depending on the magnitude and application of the structure, wind or wave force is the most potent environment load in the offshore environment due to its magnitude. While the wave is the most potent load consideration for oil and gas facilities (Muga and Wilson, 1970), the wind has been identified as most potent for offshore wind turbines (LWST, 2004). A schematic of the various categories of offshore loads loading an OWT is presented in Figure 24.

Other loads considered in the design of offshore structures are the wave, current and operational loads. The load-bearing capacity design of the structure is governed by the material as well as the geometrical properties. The material properties are defined by variables such as yield strength, Young's modulus of elasticity, Poisson's ratio etc.) while the geometrical conditions define the geometrical properties. Finally, understanding the structural deformation due to these loads is also essential due to its increasing tendencies throughout the service life of the structure. Incorporation of the above in the design and how their dependencies would be the focus of the following sections.

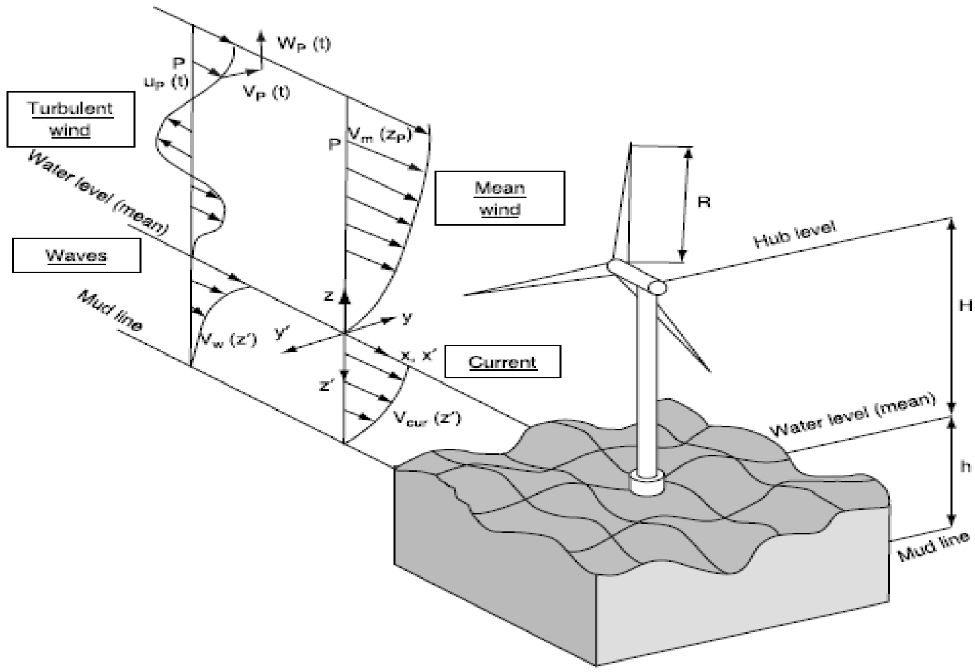

Figure 24: **Schematic of an Offshore Wind Turbine under environmental loading (Petrini et al., 2010)**

4.2 Loads Classification

To design a reliable offshore structural system, proper considerations must be given to the loading pattern and magnitude. This is because offshore structures are exposed to different types of loading during their service life. Due to the uncertainties associated with environmental loading conditions, adequate consideration should be given, and decisions based on an experienced, well-informed judgement. Generally, offshore loads can be classified as; dead loads, live loads, and dynamic loads (DNV 2016; Damiani 2016).

Dead loads are the self-load of the structure, including any permanently fitted equipment station on the platform. Dead loads include hydrostatic forces acting on the structure beneath the waterline, as well as buoyancy and external pressure. Live loads are the operational loads. They include any force or forces that is exerted on the structure. These loads are mainly due to the facility's operations, such as load (weight) from production and drilling equipment, helicopter landing pad, accommodation, temporary storage areas, utility equipment, tanks and vessels, crane etc. Live loads are generally treated as uniformly distributed loads (Nowak and Collins, 2000).

Environmental loads are natural phenomena that exert force on the structure. They include wind, wave, current, ice, snow, earthquakes, hurricane, Tsunami, earth movement etc. Variations in hydrostatic pressures due to the buoyancy of members caused by changing water levels due to actions of waves and tides are also included in environmental loads consideration. These loads are stochastic and can be estimated by a combination of relevant and credible met-ocean data with probabilistic descriptions.

Other loads such as dynamic are loads exerted on the platform due to response to any excitation of cyclic loads from wind, wave, etc., including the impact caused by landing boats or barges at the platform. Construction loads are loads exerted on the structure due to the structure's installation, retrieval, and transportation.

These environmental loads described are key load considerations for the design of offshore structures due to their randomness. Therefore, this chapter shall present appropriate analytical models of the environmental loads, especially for offshore wind turbines, with wind force as the most potent load (LWST, 2004).

4.2.1 **Environmental loading**

Offshore structures are subjected to both steady and time-dependent environmental loading conditions throughout their service life. Depending on the application and magnitude of the structure, hydrodynamics and wind play a significant role in the loading. Fixed offshore structures are configured for parts of the structure to be above the waterline, and the remaining is submerged in water and retrofitted to a foundation assembly that is driven into the seabed. The topside of the structure is subjected to both steady wind forces and gusts, which induces high unsteady local forces on structural members. The submerged sections are subjected to steady hydrostatic forces and localized vortex shedding, which induce substantial unsteady forces exerted on the structural components.

For a credible design modelling of offshore structures, it is crucial to establish and properly choose methods for the transformation of environmental loads to their resultant steady and time-dependent forces acting on the structure. The procedures of transforming global level-environment to a structural level can be classified into the following (IEC2005; DNV 2015; Kallehave et al., 2015; Athanasios 2010; Efthymiou and Graham, 1990);

- Description of environmental conditions, i.e., design parameters to describe wind, current and wave
- Analysis of water-particle motion resulting from movement of water due to the wind, current and wave assuming the structure does not interfere with the gross particle movement
- Derivation of the impact of external forces on the structural members of the structure

4.2.2 Environmental conditions modelling

As part of the critical parameters to be considered in a reliability assessment, the Reference Return Period (RRP) is a significant factor for design consideration. It is used to relate the environmental conditions that are exceeded on average on N-years. Generally, for offshore structures, an RRP of about 100-years is recommended (API RP 2A-LRFD, 2014). Although this period may vary for different standards, organizations have their own recommended RRP. For instance, the UK Department of Energy recommends a 50-years RRP. The application of statistical techniques can be used to scale up observations of a limited period of about 1-5-year time scale to forecast the probability of failure of the 50 or 100 years RRP basis of design.

The resultant load that acts on the structure is a combination of loads, and each of the constituting loads is assumed to be acting in the same direction and time to obtain a worst-case scenario result. This assumption is mainly conservative but can provide a realistic result for the modelling structure against structural failure. For modelling extreme environmental loading, the critical design parameters are the average wind speed over a period, the significant wave height, current speed profile and the mean zero-crossing period. Modelling of these parameters will be presented in detail in the following sections.

4.2.3 **Geotechnical Properties Modelling**

Soil is a complex material with non-linearly behaviour and often shows anisotropic and, in some instances, time-dependant behaviour when subjected to stresses. Soil does not follow the basic loading, unloading and reloading pattern but instead undergoes plastic deformation and is inconsistent in dilatancy (Thi et al., 2014; Kok et al., 2009). Soil follows a non-linear behaviour well below failure condition with stress dependant stiffness. These characteristics make it nearly impossible to model it as simple elastic-perfectly plastic material. Thus Mohr-Coulomb model for soil is the most applied model even if the model is not necessarily the most preferred soil model option (Kok et al., 2009). More read of the five basic aspects of soil behaviour can be found in (Athanasios 2010; Brinkgreve, 2005).

Several models have been used to represent the stress-strain and failure behaviour of soils in recent years. However, they all constrain certain advantages and limitations which depend on their application. Chen (1985) recommends basic model criteria to consider in the selection of modelling methodology.

The first criterion is a theoretical assessment to ensure the model adheres to basic principles of continuum mechanics and inconsistency with the theoretical requirements of continuity, stability, and uniqueness. The second criterion is based on an experimental assessment of the models with regards to their adaptability to fit experimental data from an available test and the ease of determining the material parameters from standard test data. The third criterion is based on a computational and numerical assessment of the models for the facility in which they can be implemented in computer calculations.

Summarily, the criteria for the appropriate soil model for any analysis should strike a balance between the requirements of the realistic soil characteristics from the laboratory test report, the continuum mechanics, and the ease of computational application.

The Drucker–Prager failure criterion model is a 3-D pressure-dependent model mainly used to estimate the stress state at which the soil reaches its ultimate strength. The criterion assumes that the octahedral shear stress at failure depends linearly on the normal octahedral stress through material constants. The Drucker–Prager failure criterion was established as a generalization of the Mohr-Coulomb criterion for soils (Drucker and Prager 1952). This material model utilizes a yield function as presented in Figure 25 and the function defined by:

$$F(t) = \gamma I_1(t) + \tau(t) - \beta \tag{4.1}$$

where

$$I_1(t) = \tau_{11}(t) + \tau_{22}(t) + \tau_{33}(t)$$

$$\tau(t) = \sqrt{\frac{1}{2}\sum_{i.j=1}^{3} S_{ij}\,(t) S_{ij}(t)}$$

And,

$$S_{ij}(t) = \tau_{ij} - \tfrac{1}{3}\delta_{ij} I_1(t)$$

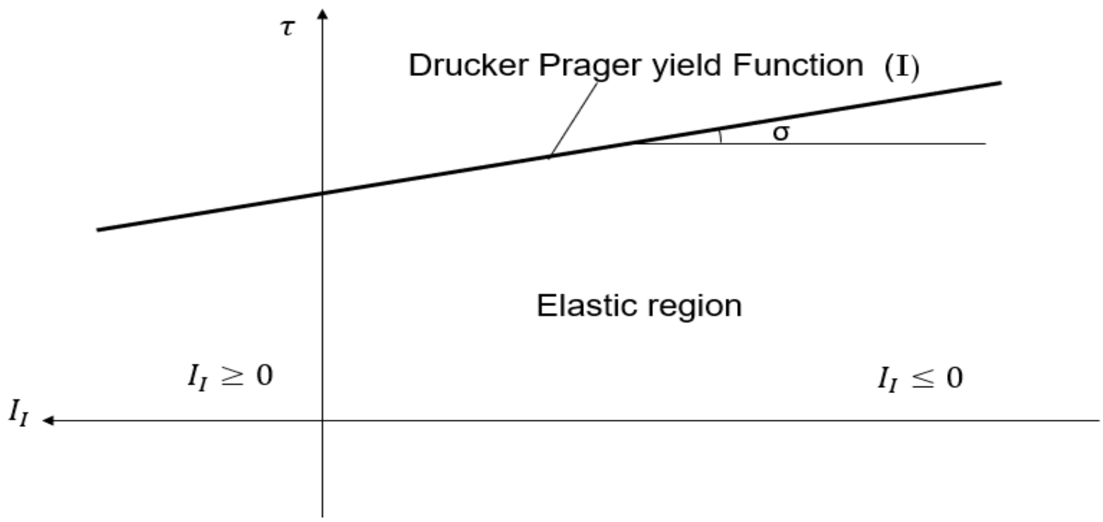

Figure 25: Schematic of Drucker-Prager Yield function (Drucker, 1952)

The yield criterion is like von Mises perfect plastic yield criterion. The parameters γ and β are determined by standard compression triaxial tests and can be expressed in the form of internal frictional angle and cohesive intercept (Leandro, 2012; Colmenares and Zoback 2001, 2002; Yi et al. 2005, 2006), by the following expression:

$$\alpha = \frac{2sin\ \theta}{(3 - \sin\theta)\sqrt{3}} \tag{4.2}$$

$$\beta = \frac{6c\ Cos\theta}{(3 - \sin\theta)\sqrt{3}} \tag{4.3}$$

Where θ, is the angle of friction and c, is the cohesion value.

Due to the limitations of access to live and verifiable data, this study would adopt data available from selected literature to develop and analyse the soil model. This is giving that the parametric modelling method used for this work allows us to develop a 3-D model based on the numerical model, which is also a module in ANSYS, which is widely used for full-scale research and development.

4.3 Loads of OWT Structures

OWT jacket support structures are generally exposed to complex and variable loads. Therefore, in conducting any structural analysis, adequate consideration of the loads is essential. The loads relevant to OWT jacket support structures can be roughly categorized into three groups, i.e., dead load, live load and environmental loads, which are site-specific. The design loads to be taken into account during the design phase of the support structures are generally suggested in relevant design standards such as DNV-OS-J101 (Det Norske Veritas, 2014) and IEC 61400-3 (IEC, 2009). This study's numerical computation of loads was based on DNV-RP-C205 (Det Norske Veritas (DNV), 2010). The schematic diagram in Figure 26 illustrates the various loads acting on the jacket support structure.

4.3.1 **Inertia load**

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

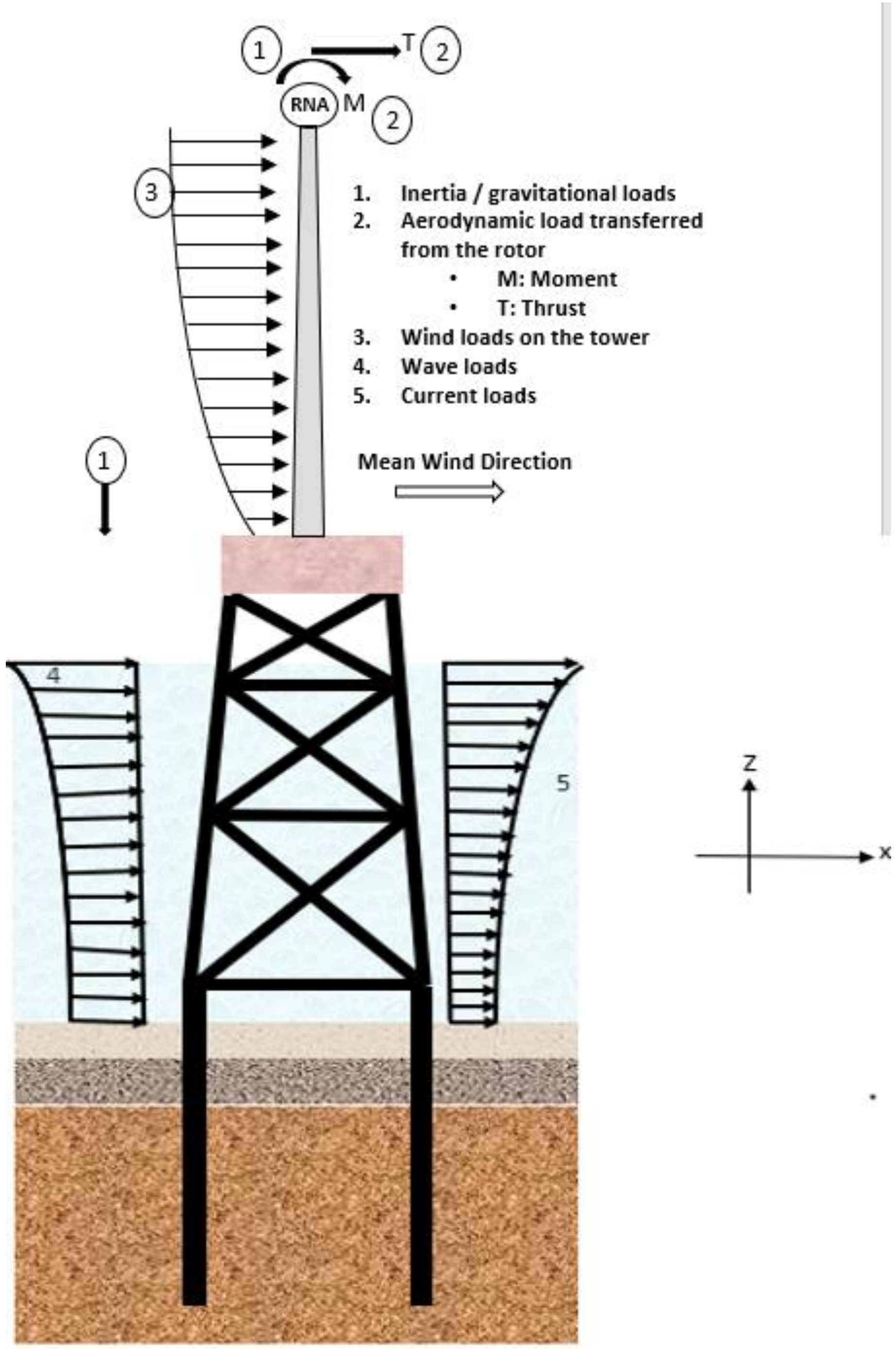

Figure 26: Offshore Jacket Structure under Environmental Load

4.3.2 **Wind load**

Wind load or wind force is the obstruction of wind flow by the structure, which produces a differential pressure. The static drag force generated due to the wind on the wind turbine structure accounts for about 15% of the total force on the structure and 25% of the total overturning moment (Wilson, 2003). The magnitude of the drag is dependent on the met-ocean data, such as the mean wind velocity $\bar{V}(z)$. Mathematically, the wind speed for any elevation above the mean sea level is given by:

$$\bar{V}(z) = \bar{V}_r \left(\frac{z}{z_r}\right)^{\propto} \tag{4.4}$$

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

$$F_{tower}(z) = \frac{1}{2}\rho_a C_{D,T} D(z)\bar{V}_r^2(z) \tag{4.5}$$

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

The wind force acting on the front (upwind) or back (downwind) of the rotor also generate a load on the rotor that is transferred from the topside to the support structure. Realistically, aerodynamic loads acting on the rotor are usually transferred to the top of the jacket and are generally decomposed through the load matrix defined in the turbine's axis. Typical design load values for fatigue and extreme loads used for this study were extracted from the WindPACT (Wind Partnership for Advanced Technologies) report on Turbine Rotor Design Study (Lanier and Way, 2005).

4.3.3 **Wave load**

Wave loads are dynamic loads due to waves and their interface with the support structure of the turbine. The magnitude of the load acting on a structure depends on the geometry of the structure, the ratio of the diameter of the support structure to the wavelength, the hydrodynamic conditions as well as the rigidity of the structure (Haritos, 2007). The choice of wave theory depends on the ratio of the height to the diameter of the structural member. There are three approaches by which the wave load acting on an offshore structure can be modelled, namely:

- Diffraction theory – is applied for cases when the ratio of the diameter of the structure, D, is greater than one-fifth of the wavelength, λ, diffraction theorem can be applied for wave estimation (DNV, 2014).

$$D > 0.2\lambda$$

- Morrison's equation – Morrison's equation is used to model wave load that involves a small scale for the wavelength, and the induced drag force is significant. For instance, when the diameter of the structure, D, is less than one-fifth of the wavelength, λ, Morrison's equation can be applied for the wave estimation (DNV, 2014).

$$D \leq 0.2\lambda$$

- Froude-Krylov theory – is applied when the drag force is not very significant, and the inertia is the principal force, whilst the structure is still relatively regarded as relatively small.

This thesis shall adopt Morrison's equation since fixed offshore jacket support structures are generally drag dominated, and the diameter to wavelength ratio is usually less than 0.2. Morrison's equation assumes that the wave load is a combination of drag force and inertia force (Hsu, 1984). Morrison's equation can be expressed as:

$$\boldsymbol{F_{wave}(z) = \frac{1}{4}\rho_w \pi D^2 C_M \dot{u}(z,t) + \frac{1}{2}\rho_w D C_D u(z,t) \,.\, |u(z,t)|} \tag{4.6}$$

where ρ_w is the density of water with a typical value of 1000 kg/m^3, C_M and C_D are the coefficient of inertia and drag of the piles, respectively, and their corresponding values are 1.6 and 1.0, respectively, according to (DNV GL AS, 2016). while $u(z, t)$ $u(z, t)$ horizontal velocity and acceleration of water particles, which can be obtained from linear/Airy wave theory (Chakrabarti, 2005).

4.3.4 **Current load**

Current accounts for the movement of water, and such movement around a support structure can induce a drag acting on it. Depending on the sites of deployment, the current load could impact the total hydrodynamic load bearing on the structure. Circulation currents, tidal currents, and storm-generated currents are the most common type of current. The current profile consists of the vector sum of the currents, the speed and direction specified elevations. Based on the location, the current characteristics could be considered a major design factor of the structure as it might impact the location and orientation of deployment as well as that of boats approaching the structure, which can induce accidental loads. Current loads are in most cases added in those due to the wave. Thus relative direction and magnitude of the wave and current should be modelled realistically to characterizes the current's profile from seawater level to the seabed.

The current force induced on the structure is based on the maximum design velocity. For shallow water applications with limited data availability, the vertical distribution of current velocity can be described as (Athansios 2010; Gaythwaite, 1981). A deepwater application corresponds to slender structures design, wave and current interaction should be considered. For such instances, the drag loading is proportional to the square of the wave and current velocity. Thus the structural response will be considerably affected by any change in the current velocity. Generally, the superposition of current and wave velocity is mainly used in modelling the current-wave interaction. The current velocity at MSL (mean sea level) can be estimated using an exponential profile, given as:

$$u_c(z) = u_{c,\ MSL} \left(\frac{d+z}{d}\right)^{\frac{1}{7}} \tag{4.7}$$

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

4.3.5 Extreme and Design Values Estimation

Extreme values of environmental variables can be estimated by marginal probabilities of exceedance which is determined by statistical processing of an extreme value probability distribution of available data. Extrapolation of the data to small exceedance probabilities can be realized by applying an appropriate fitting tool. Long term values of environmental variables can be obtained depending on empirical procedures. Vledder and Zitman (1992) presented statistical procedures to follow in estimating extreme values, which are;

- Obtain dataset via hindcast or measured data
- Apply statistical model fit
- Derive the required return value

4.4 Load Cases

IEC61400-3 (IEC, 2009) defines 32 DLCs (design load cases), covering various operational modes of the turbine such as start-up, normal operation, shut down and 50-years extreme conditions. These DLCs can be roughly categorized into two major groups, namely ultimate and fatigue DLCs. The typical load cases applied in the structural design of OWT is the fatigue load under normal sea conditions and the ultimate load under 50-year extreme conditions (Gentils et al., 2017). As mentioned earlier, it should be noted here that on a real application of the proposed method, reliability indices should be calculated for all applicable limit states to calculate the worst-case scenario from a structural reliability perspective. In this study, both ultimate and fatigue DLCs are considered.

4.4.1 **Fatigue load case**

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

4.4.2 **Ultimate load case**

For the extreme environmental conditions experienced by the OWT, the 50-year return period is generally considered a critical ultimate load case. It has been demonstrated in previous studies that the NREL 5MW OWT is predominantly governed by the impact of the aerodynamics (wind) load rather than the hydrodynamic (wave and current) loading (Baniotopoulos et al., 2011). Therefore, the critical load case for ULS is mainly considered to correspond to the parked turbine under the 50-year EWM (extreme wind model) with a 50-years RWH (reduced wave height) and ECM (extreme current model). The loading characteristics as described above correspond to the IEC61400-3 DLC 6.1b and 2.1 (IEC, 2009) and GL regulation (GL, 1995), respectively. Load safety factors for gravitational load and other loads (such as wind, wave, and current loads) are given as 1.1 and 1.35, respectively (International Electrotechnical Commission, 2005). The design loads and design load cases used for this study are summarized in Table 1 and Table 2, respectively

Table 1: Summary of aerodynamic loads (Lanier and Way, 2005)

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

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

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

For life extension assessment, load data extrapolated from the sensors by the controller is a source of uncertainty. This is because the controller is not necessarily guaranteed to behave linearly to respond to extreme environmental load events in the same manner as it does for regular loads. A typical example would be a vibration or load sensor used in an active load curtailment functionality that is activated when extreme loads are detected.

4.5 Design Load Data Sampling

The primary statistical methods used to obtain a subset of data for the derivation of extreme values include:

- Initial Distribution method
- Annual Maxima method
- Peaks Over Threshold method

Tucker (1991) presented the Initial Distribution method, where all available data, including data with multiple values generated from the same event, are considered for extrapolation. The estimation is performed based on the application of an appropriate statistical model to a distribution of data that does not, in most cases, adequately describe the extremes. This method can introduce errors in the derived extreme design values.

The Annual Maxima, in contrast, derives its extremes from a single most severe observation within the given year, providing a series of uncorrelated observations. There are, however, issues on the definition of a year in temperature climate like that of the North Sea, and for a location that does not necessarily experience significant rare events annually, such as in tropical climate (Gulf of Mexico) (Ling et al. 2015; Grosskopt and Sharma, 1994).

In Peaks Over Threshold method considers extreme whenever a given threshold is exceeded. Therefore, appropriate estimation of the reference threshold must be based on a knowledgeable and experienced judgement. This method applies to regions like the Gulf of Mexico, with the wave as the predominant force (Haring and Heidenman, 1980).

4.5.1 Statistical model Fit

After a successful section of the data set, extreme values will be obtained from statistical fitting based on an appropriate theoretical model. The choice of a proper fit model should be based on the best fit of the available statistical distribution (Muir and El-Shaarawi, 1986). From the available probability distributions extreme values calculations, the most applied methods are:

- Log-Normal
- Fisher – Tippett Type I (FT- I), Type III (FT – III)
- Weibull 2 and 3 parameter

Log-Normal distributions are commonly used in reliability analysis because they take only positive values and are used for extreme values computations. Alongside the Weibull distribution, they are the two typical representative distributions used in describing the long-term behaviour of the significant wave height. In addition to the choice of the distribution function, the following methods can be used to obtain the parameters of the extreme distributions:

- Least squares method
- Maximum likelihood method
- Moment method

The least-squares method transforms the variable to linearize the cumulative density in the distribution curve, with new sets of variables, fitting a line to the transformed data set will determine the parameters of the distribution. This method is mainly used for Log-Normal, FT-I and FT-II distributions. Although this method is widely used, Muir and El-Shaarawi, (1986) highlighted some drawbacks of a bias.

The Maximum likelihood method is an improvement of the least square method. It provides estimated parameters with minute variance based on the construction of the likelihood function based on the probability density function of the observations expressed in terms of the unknown distribution parameters. Maximization of the logarithm of the function determines the values of the parameter. This method has an advantage of its asymptotic properties due to the vast number of observations collected, which increases the estimated convergence to be close to the true value (Soares and Scotto, 1996).

The moment method is a direct application of the principles of moments and is based on the equation of the second and third moments of the distribution of the data, and thus established correlations between the estimated parameter and the variance, the sample-mean and skewness.

The choice of an appropriate distribution for estimating extreme values is based on the representation of the available data by a line if the variation of the line can be evaluated by a fitness test. This method can be applied based on the empirical distribution function. The test for empirical distribution functions is usually distribution-free and could help characterise the various possible distributions (Muir and El-Shaarawi, 1986). Kolmogorov-Smirnov statistical test, Anderson-Darling Statistical test and the Crammer-von Mises test can be used to perform this kind of test. Details of the statistical test are not covered in this thesis; however, it is essential to carefully select every plotting position throughout the testing procedure.

4.5.2 **Design Values**

Soon as the choice of probability distribution has been made based on the steps described above, the design value can be obtained based on the return period (T_R), or encounter probability (E_P). The T_R is given by the average time interval between successive events being exceeded, and is directly proportional to the probability of exceedance and is given as:

$$T_R = \frac{r}{Q(X)} = \frac{r}{1 - P(X)} \tag{4.8}$$

where r is the time interval associated with the individual data point and $Q(X)$ is the probability of exceedance. The E_P is given as the probability that the successive event is exceeded during a given period L_fWhich defines the design life of the structure. Borgman (1963) developed an empirical correlation that shows a relationship of the $E_P\ to\ T_R$ and is given as:

$$E_P = 1 - \ exp^{\left(\frac{-L_f}{T_R}\right)} \tag{4.9}$$

4.6 Fluid Loading on Offshore Structure

Loading is an essential criterion in reliability analysis, as it is responsible for the response of the structure. During the service life of a structure, it is exposed to various kinds of loading, which vary with location and the type of application the structure is used to support. Early sections of this chapter have treated the method of derivation of environmental parameters. Application of the derived parameters to a structural analysis would, however, require a combination of good engineering judgement as well as sound scientific knowledge for result validation and to minimize errors.

Existing methods for fluids loading on the structures can be grouped as deterministic, probabilistic, and spectral. The most used and simplest of them is the deterministic method, which analyses the fluid by considering the non-linearities. Still, it is limited by the fact that it has a restriction to incorporate uncertainties of the sea state conditions (Vugts, 1979). Previous research has shown that although easy to use, this method does not produce realistic force distribution among the structures. In most cases, it underestimates the loads by 10% compared to a probabilistic model (Hagemeijer, 1989).

The probabilistic method is a more robust method that accounts for the randomness of the environmental forces with their statistical properties, allowing for elaborate period's statistical properties before evaluating the design return period. In applying the probabilistic method, distribution is formulated to describe the significant environmental conditions such as the significant wave heights, zero mean crossing period as well as extreme conditions. To transform deterministic parameters to a probabilistic model, wave theories would be required to select wave height. Upon selection of an appropriate design wave, a corresponding period can then be assigned as described in relevant sections of literature such as (Longuet-Higgins 1983; Mathisen and Bitner-Gregersen 1990; Constantine and Tzanis, 1994).

The spectral method is the most straightforward method to represent fluid loading because it holistically describes the loads and response of the structure statistically. It considers the variability of sea surface-associated kinematics. This method is based on the linearization of non-linear processes. Spectral is generally applied for offshore structures' dynamic and fatigue response assessment in harsh environmental conditions (Bea and Lai, 1978). Figure 27 presents the sequence of spectral analysis, discretization of time is applied to make up for variation in time. Transfer functions are established to calculate the wave force response from the sea surface spectrum. The sea surface spectrum $S_W(\omega)$ is the wave force transfer function input, while $S_F\,(\omega)$ is the output. In performing structural analysis using this approach, the wave force spectrum is used to calculate the displacements by multiplying the wave force spectrum by the transfer function of the structure.

4.6.1 Structural Response Under Environmental Loads

Hallam et al. (1977) and Sarpkaya and Isaacson (1981) performed an extensive study on the characterization of the environmental conditions. Deployment of the appropriate wave theory will be helpful in the analysis of the kinematics properties of the fluid driving the hydrodynamic forces, which is a significant factor in determining the response of the structure. The response analysis can be dynamic or static. The choice, however, depends on the natural frequency. Vugts (1979) summarized the static or dynamic analysis decision criteria for structural response modelling as follows;

When the natural frequency of the structure is lower than the response frequency, in this scenario, static analysis can be used. However, if any of the natural frequencies of the structure is in the range of the resonance frequency, a dynamic analysis would be required.

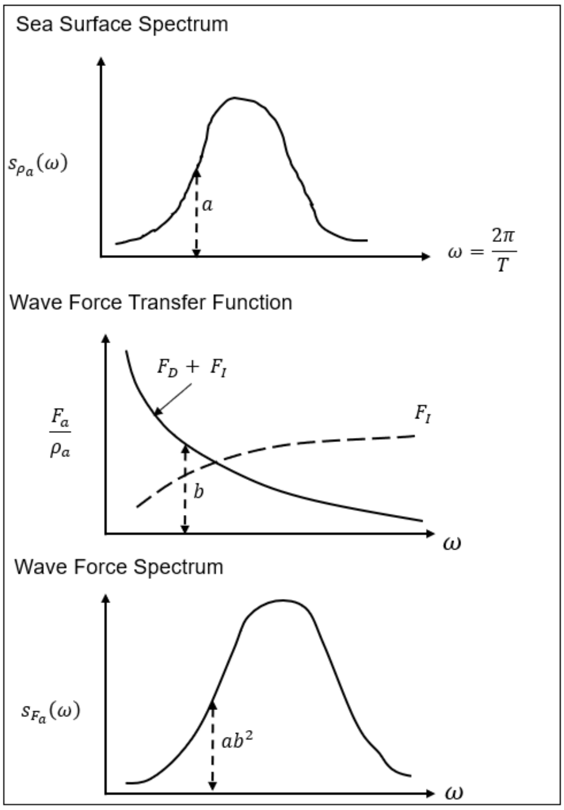

Figure 27: Frequency Domain analysis (Hallam et al., 1977)

4.7 Structure Capacity Modeling

Structural resistance or capacity is a critical aspect of limit state functions, as it describes the ability of the structure to withstand loads and resist failure under loading conditions. The capacity model is a function of several variables. There are empirical and theoretical approaches to deriving the resistance model of a steel structure. However, for appropriate modelling of structural resistance, all parameters that contribute to the capacity of the structure must be taken into consideration and their properties well represented during model formulation.

For a robust design such as the OWT, the validation of the design is based on a comparison of the experimental and theoretical data. The comparison can be made by using appropriate plots to demonstrate the correlation between results obtained from these techniques. Plots used for such correlation would be either a plot to observe the calculated resistance values against the experimental values or plots that monitor the performance of the resistance values against standards. Significant deviation from the experimental values in the plot would result in a reconsideration of the theoretical models.

4.7.1 **Design Material Modelling**

Properties of construction material for every steel grade is mainly derived from relevant experimental procedures based on standardized practice. To assess the reliability of a steel structure, the critical material properties of interest needed for the modelling are the yield strength, Poisson's ratio, Young's modulus, and fracture toughness. Details of this will be discussed in this section.

4.7.1.1 **Yield Strength**

Yield strength can be defined as the point at which a predetermined amount of permanent deformation occurs. Eurocode (2002) and Hart et al. (1985) have described the characteristics yield strength as the five per cent (5%) quantile of the test data. Hart et al. (1985) conducted a study on the correlation of Coefficient of Variation (CoV) with yield strength using different steel grades. The result of the study is presented in Table 3. The result suggests that yield strength follows a log-normal distribution, which is affirmative of DNV standards' position (DNV-OS-J101, 2014).

Table 3: Properties of Yield Strength. Source: Hart et al., (1985).

Material	Statistical Distribution	Yield Strength (MPa)	CoV (%)
		<350	8.0
Steel	Log-normal	350 - 400	6.0
		>400	5.0

4.7.1.2 **Young's Modulus and Poisson's Ratio**

The Young's modulus of steel material is found to be normally distributed with a mean value of 210 GPa, and a CoV of 5% (Smith et al., 1985), while Galambos and Ravindra (1978) proposed a value of 6% Poisson's ratio was represented by a constant value of 0.3 for steel. A table of cumulative data for Young's modulus and Poisson's ratio is presented in Table 4.

Table 4: Poisson's ratio and Young's modulus (Kolios, 2010)

Material	Distribution	Poisson's ratio		Young's Modulus	
		Mean	CoV (%)	Mean (GPa)	CoV (%)
Steel		0.3	-	210	5.0
Aluminium	Normal	0.3	-	70	-
Concrete		0.2	-	30	-

4.7.1.3 **Fracture Toughness**

Fracture toughness is a material property that is used to define the stress intensity factor K_{IC} and Crack Tip Opening Displacement (CTOD). Although its statistical distribution was once considered to be log-normal, Tronskar et al. (1992) reported a two-parameter Weibull distribution as a better fit for CTOD. Since this thesis does not consider fracture mechanics, this work will not cover further details and in-depth analysis.

4.8 FEA Modelling of Degraded Structure

The modelling and simulation of the ageing structure that has suffered degradation due to the combined action of corrosion and fatigue were performed based on the data obtained from the structural inspection. The developed FEA model is modified to reflect the structural degradation as well as any structural modifications. The FEA was developed in ANSYS using beam/frame elements. Details of the process of FEA development are presented in the subsequent section of this chapter. The fatigue module in ANSYS DesignXplorer© (ANSYS, 2019) is used to perform the simulation-based of the proposed framework present in section 3.2.4. it also recommended that the FEA model should be verified against available standard structural response measurements. The most common form of degradation is mainly from corrosion. It is modelled by reducing the cross-sectional area of the structural details, depending on the magnitude of the material loss due to corrosion.

4.8.1.1 **Failure Modes and Corrosion Modelling**

Failure modes related to offshore jacket type support structures include several time-dependent phenomena which are predominant for their design. Such as the impact of corrosion and fatigue damage due to the marine environment results in the degradation of the material (i.e., steel), which ultimately affects its resistance (Figueira et al., 2017). Due to the amplitude of fatigue loads in combination with large numbers of load cycles due to the combined actions of wind, wave and operational loads, fatigue performance of welded connections is a design-driving criterion for OWT support structures (Dong et al., 2012). Corrosion can reduce the material thickness, thereby making it susceptible to fatigue crack initiation and buckling, which may result in failure of the structure (Adedipe et al., 2016, 2015).

Data of structural degradation are expected to be obtained corrosion data obtained from the inspection reports. Reduction in thickness of structural members, if available, is essential for the assessment. These data are expected to be transferred into the FEA model for appropriate case study simulation. However, suppose no significant localized corrosion or fatigue cracks were reported in both the submerged and splash zone of the structure. In that case, it is recommended to assume a uniform corrosion model parameter. Several factors could affect the rate of material wastage (corrosion) of the structure, such as the temperature, water velocity, salinity, period of exposure etc. (Melches 1999; Melchers 2005; Pierre 1999). However, typical values of material wastage still are between 0.4 to 1.2 mm/year (Moan, 2005).

For credible reliability assessment, the FEA model should as much as possible represent the status of the structure at the time of the analysis, based on available data. This should include geometrical alterations or modifications during the installation and operation phases and should be modelled accordingly. Additionally, a precise model of the foundation should be ensured considering the soil stiffness information, angle of friction and all relevant coefficients. The FEA model should be validated against available structural reliability data. Modifications can then be executed to account for the effect of time-dependent (Uniform corrosion or localized corrosion) structural degradation of the structure.

4.9 Development and Simulation of Structural Degradation

The modelling and simulation of the degraded ageing structure due to the combined action of corrosion and fatigue was performed by reducing the cross-sectional area of the members based on the data obtained from the structural inspection. These data shall include all previous reports of inspections carried out and the measurements of the thickness of the critical members. It is highly recommended that the FEA model should, as much as possible, be a replica of the structure at the time of the analysis.

Furthermore, a precise model of the foundation should be ensured considering the soil stiffness information, angle of friction and all relevant coefficients. The FEA model should be validated against available structural reliability data. Modifications can then be executed to account for the effect of time-dependent (Uniform corrosion or localized corrosion) structural degradation of the structure.

The simulation is conducted by creating an FEA model based on the initial structural design and then modifying the model to reflect the structural degradation and any structural modifications. The FEA was developed in ANSYS using beam/frame elements, details of the process of FEA development is presented subsequently in a later section of this chapter. The fatigue module in ANSYS DesignXplorer© (ANSYS, 2019) is used to perform the simulation-based of the proposed framework present in section 3.2.4. it also recommended that the FEA model should be verified against available standard structural response measurements. The most common form of degradation is mainly from corrosion

4.9.1 Development of Parametric FEA Model

Structural models to determine the response of OWT support structures can be roughly categorized into two groups, i.e., the 1D (one dimensional) beam model and the 3D (three dimensional) FEA (finite element analysis) model. The 1D beam model generally represents the

support structure into a sequence of elastic beam elements. This method has been widely used in structural modelling of OWT support structures due to its computational efficiency and acceptable accuracy for modelling global structural behaviour (Bossanyi, 2009). Though efficient, the beam model has the limitation of accurately representing the local structural responses such as local stress concentrations (Petrini et al., 2010). The 3D FEA model, which generally constructs OWT support structures using shell elements, can accurately estimate the structural responses and examine the detailed stress distributions across the support structure (Wang et al., 2016). Therefore, the 3D FEA model is used further in this study for modelling the support structure, ensuring accurate prediction of structural responses subjected to complex loads

A parametric FEA model of an OWT jacket support structure must first be developed using any reputable CFD software to perform a structural reliability assessment. For this thesis, ANSYS was used, such that respective input parameters were assigned their corresponding distributions. The developed FEA model is then used to perform series of simulations for the respective case studies through the DoE module in the DesignXplorer© tool in ANSYS to map the response domain and report the response surface model. Finally, the six-sigma analysis tool in the DesignXplorer© in ANSYS is used for the probabilistic analysis of the response surface (ANSYS, 2019).

4.10 Limit State Assessment Formulation

The integrity of an asset in the presence of uncertainty is best evaluated through a structural reliability approach. An effective way to achieve this is through a non-intrusive formulation, following several steps as illustrated in Figure 28. This method has been applied in similar research to evaluate the performance of a wave energy converter, mainly focusing on global limit states (Kolios et al., 2018) and the calculation of the reliability of offshore monopile support structures (Wang and Kolios, 2017). The processes in the flow chart are meant to describe the step by step processes of RSM-FORM. The flow chart below summarizes both the FORM and FEA modelling processes.

The benefit of this approach is that it is generic enough to accommodate several problems, and high-fidelity tools can be employed for the individual steps, increasing the analysis's accuracy (Kolios et al., 2018). The algorithm should be adapted for every different failure mode to be formulated in the form of limit states which distinguish safe and failure operational regions.

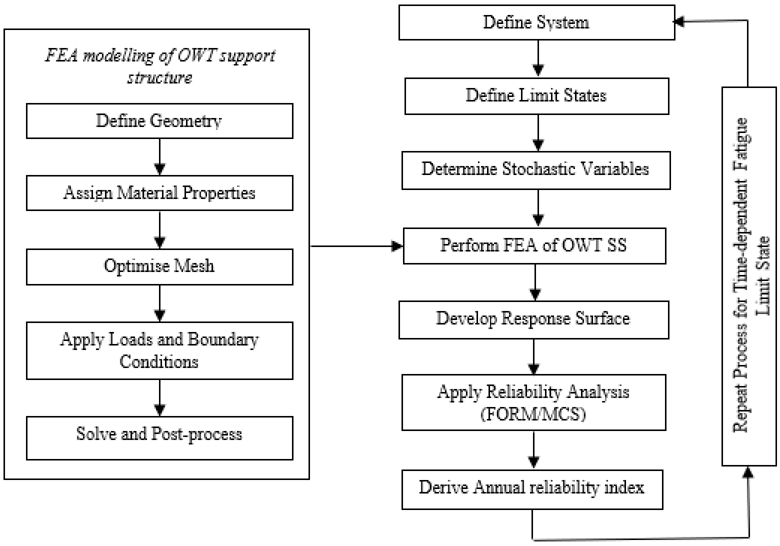

Figure 28: Non-intrusive formulation of structural reliability algorithm

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

i. Define System: Here, the details of the structure are determined, including the layout, material properties, type of analysis etc. This will stand as the basis of the parametric model that will be developed, allowing us to effectively evaluate the response of the structure under varying input conditions.
ii. Define Limit States: Failure modes relevant to the system of reference need to be modelled in performance functions in the form of resistance (allowable) minus load (available). Obtaining positive values for the cases that the structure/component stands in the safe region and negative values when it fails.

iii. Determine stochastic variables: Among the design variables, those with the highest degree of uncertainty should be modelled statistically for further consideration in the analysis. Historical data can be employed, and the fitting of various distributions should be tested to qualify the most appropriate statistical properties alleviating statistical uncertainty.
iv. Perform several simulations: Several simulations need to be executed, mapping the response domain for the combination of inputs and building a design matrix to allow the derivation of analytical expressions through regression.
v. Develop response surface: Employing appropriate approximation methods (response surface or surrogate modelling), an analytical expression can be developed to allow for a probability integral solution. The complexity of the limit state will qualify as the most appropriate approximation method.
vi. Apply reliability analysis methods: Once the response surface has been developed, analytical (FORM/SORM) or stochastic methods (MCS) can be employed to calculate reliability.
vii. Derive reliability index: Depending on the method employed, FORM/SORM derives values directly for reliability index (β) while MCS requires a transformation from the direct calculation of the probability of failure.

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

4.10.1 **Multi-Criteria Design Limit State Assessment**

Recent structural design standards follow a limit state design approach, aiming to derive designs with adequate safety margins to take account of uncertainties that could adversely affect the reliability of the structure. The structure is required to be checked for all categories of limit states to ensure adequate safety margins between the maximum likely loads and minimum resistance of the structure [19].

DNV-OS-J101 design standard [20] suggests three primary limit state considerations in the design of OWT support structures, i.e. (1) ULS (ultimate limit state), which is the load and resistance capacity of the structure (such as buckling and yielding stress); (2) FLS (fatigue limit state), which accounts for failures due to cyclic loading; and (3) SLS (serviceability limit state) which accounts for design tolerance criteria (such as deflection and vibrations). In this study, five design limit states are considered, i.e., buckling, deflection, fatigue, frequency, and ultimate limit states considering two more limit states relevant to slender structures.

4.10.1.1 **Buckling limit state function**

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

$$\boldsymbol{g_b(x) = L_{m,\ allow} - L_m} \qquad \textbf{(4.10)}$$

where subscript b denotes the buckling limit state, L_m is the buckling load multiplier, which is given by the ratio of critical buckling to the load on the jacket support structure and $L_{m,\_\mathrm{allow}}$ is the allowable minimum load multiplier. The above equation implies that the higher the load multiplier, the safer the structure (Lin et al., 2016), as a high load multiplier means the applied loads on the structure is well below the critical load that causes buckling. Also, if the buckling load multiplier L_m is less than the allowable minimum load multiplier, the structure fails. The buckling limit state can be a prevailing design criterion in different types of structures, such as suction buckets. Additional phases of the service life of the asset should be considered, e.g. installation).

According to the DNV standard in DNV-OS-J101 [20], the minimum allowable load multiplier is given as 1.4, and this is therefore used in this study.

4.10.1.2 **Deflection limit state function**

During normal operating and extreme loading of the support structure, the structure may be deflected considerably in the direction of the load. Although deflections are expected, if the total deflection of any member becomes excessive, it can significantly influence the serviceability of the OWT support structures. Thus, the deflection limit state is also a critical factor for consideration in the reliability assessment. The maximum total deflection in this study is measured at the top of the tower. The performance function of deflection limit state design can be expressed as:

$$\boldsymbol{g_d(x) = d_{allow} - d_{max}} \qquad \textbf{(4.11)}$$

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

According to the DNV standard in DNV-OS-J101 [20], the allowable deflection can be obtained based on the following empirical equation:

$$\boldsymbol{d_{allow} = \frac{L}{200}} \qquad \textbf{(4.12)}$$

where L represents the length of the support structure.

4.10.1.3 **Fatigue limit state function**

OWT support structures are exposed to significant cyclic loadings throughout their service life, making fatigue limit state consideration particularly important to this study. Based on the S-N curve method of fatigue analysis, the number of loading cycles to failure is given in (3-4):

$$\boldsymbol{N = \frac{A}{S^m}} \qquad \textbf{(4.13)}$$

and it can also be expressed linearly as given in

$$\boldsymbol{\log N = \log A - m \log \Delta S} \qquad \textbf{(4.14)}$$

where A represents the intercept, m is the slope of the S-N curve in the log-log plot, and ΔS is the stress range.

The values of the intercept (A) and the slope (m) are taken as 13 and 3, respectively, according to the DNV design standard in DNV-OS-J101 [20].

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

$$\boldsymbol{g_f = log(N) - log\,(N_t)} \qquad \textbf{(4.15)}$$

where subscript f denotes the fatigue limit state, N is the number of loading cycles to failure and is obtained from the calculated stress range; N_t is the design life cycle, i.e., the number of loading cycles expected during the service life. This can be estimated based on the rated speed of the rotor and the availability of the turbine at the selected location (Gentils et al., 2017). Therefore, considering a design life number of cycles for 20 years, with a rated speed of 12.1 rpm and an availability of (98.5%), the design life cycle N_t can be calculated by:

$$\boldsymbol{N_t = availability * rotor\ speed * [20\ (yrs) * 365\left(\frac{day}{yr}\right) * 24\ \left(\frac{day}{hour}\right) * 60\left(\frac{min}{hour}\right)]} \quad \textbf{(4.16)}$$

The design life cycle N_t can be computed from equation 4.16, which is then substituted in the performance function of the fatigue limit state. The S-N curve equation in 4.14 is used to calculate the fatigue design stress range$\sigma_{f,design}$, while the maximum fatigue stress range $\sigma_{f,max}$ of the structure is obtained from the FEA simulations result. It is important to reiterate that this study focuses on the fatigue assessment of the most critical details of the structure and does not account for the joints of members. This can, however, be realised by applying appropriate stress concentration factors (SCF) or adoption of the fracture mechanics method and could be suggested as a future work sequel to this study.

For the computation of the fatigue reliability index across the nominal service life of the asset, a quasi-static method was assumed, where the annual reliability index is computed and plotted accordingly for the 20 years under consideration.

4.10.1.4 **Frequency (modal) limit state function**

OWT support structures are prone to vibrations during their service life that can result in resonance. In conscious prevention of such occurrence, the first natural frequency of the jacket support structure needs to be separated from the induced frequency f_{1P} of the rotor and the blade-passing frequencyf_{3P}. The safe, natural frequency is any frequency range between the rotor f_{1P} and f_{3P} frequencies and the stochastic variable considered for this study is Young's modulus. GL standards (Germanischer Lloyds, 2010) suggest the first natural frequency should be separated from rotor induced frequencies with a tolerance of ±5%. This can be expressed as:

$$\boldsymbol{f_{1p+5\%} \leq f_{1st} \leq f_{3p-5\%}} \qquad \textbf{(4.17)}$$

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

$$\boldsymbol{0.212 \leq f_{1st} \leq 0.328 H_z} \qquad \textbf{(4.18)}$$

The performance function of the modal limit state is based on the modal frequency thresholds and can be expressed as:

$$g_{m1} = f_{1st} - 0.212 \qquad (4.19a)$$

$$g_{m2} = 0.328 - f_{1st} \qquad (4.19b)$$

Where the subscript $m1$ and $m2$ are the modal frequency limit state constraints for a safe design, while f_{1st} is the first modal frequency. The computation of the reliability index presented is based on the minimum reliability index obtained from equations 4.19a and 4.19b.

4.10.1.5 **Ultimate limit state function**

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

$$\boldsymbol{g_u(x) = \sigma_{allow} + \sigma_{max}} \qquad \textbf{(4.20)}$$

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

$$\boldsymbol{\sigma_{allow} = \frac{\sigma_y}{\gamma_m}} \qquad \textbf{(4.21)}$$

where σ_y represents the yield strength of the material, with a value of 355 MPa for steel S355; γ_m is the safety factor for the material, with a value of 1.1 suggested by DNV-OS-J101 standards (DNV 2015). Therefore, the allowable stress in this study is 323 MPa. This study has adopted the material factor rather than the conventional modelling of the resistance parameters for structural reliability analysis. This is due to the high computation demand of modelling the material resistance variable.

For accurate results, the samples required by stochastic modelling should be at least 2n+1, where n is the number of stochastic variables. This implies that the computational resources needed in stochastic analysis considerably increase with the number of stochastic variables. For simplicity, the material factor was used in this study to reduce the total number of stochastic variables required for the analysis. Further, it should be noted that the material factor accounts for several uncertainties, including not only the physical uncertainties of the material but also uncertainties related to the manufacturing process. Hence although the main material properties are stochastically modelled, the material factor is retained to account for the latter.

4.10.2 Stochastic Response Surface Analysis

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

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

$$y(x) = a_0 + a_1.x_1 + a_2.x_2 + a_3.x_3 + \cdots + a_n.x_n + \epsilon \quad (4.22)$$

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

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

$$Y = XA + E \quad (4.23)$$

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

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

$$A = \; (X^T \, X)^{-1} \, X^T \, Y \quad (4.24)$$

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

$$y(x) = a_0 + a_1.x_1 + a_2.x_2 + a_3.x_3 + a_4.x_1^2 + a_5.x_2^2 + a_6.x_3^2 + \cdots + \epsilon \quad (4.25)$$

4.10.3 **Selection of Stochastic Variables**

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

After the selection of stochastic variables, the assignment of appropriate statistical distributions to the selected variables should take place to allow for the systematic consideration of uncertainty through reliability analysis. Although the stochastic data are characterized in this application by normal distributions, the framework can accommodate variables of any statistical distribution through appropriate transformations. For this study, the mean value is taken as the base design value. The standard deviation is correlated to the mean value through a coefficient of variation, i.e., the standard deviation for the static structural loads is taken as 10% of the mean and 20% for the time-dependent analysis. Table 5 presents the stochastic design parameters.

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

Table 5: Design Variables and their characteristics (Lanier and Way, 2005)

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

4.11 FORM (First Order Reliability Method)

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

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

$$P_f = \Phi(-\beta) \quad (4.26)$$

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

FORM has been widely used in reliability assessment due to its computational efficiency and relative ease of implementation. FORM has a limitation in analysing non-linear limit state functions, and this limitation can be overcome by using SORM methods. In this study, the FORM, considering the Hasofer and Lind index, is used in the reliability assessment (Lind, 1974). The flowchart of the FORM process for this study is shown in Figure 29.

The flowing flow chart summarises the process and steps for computing the reliability index using FORM, considering the Hasofer and Lind Index.

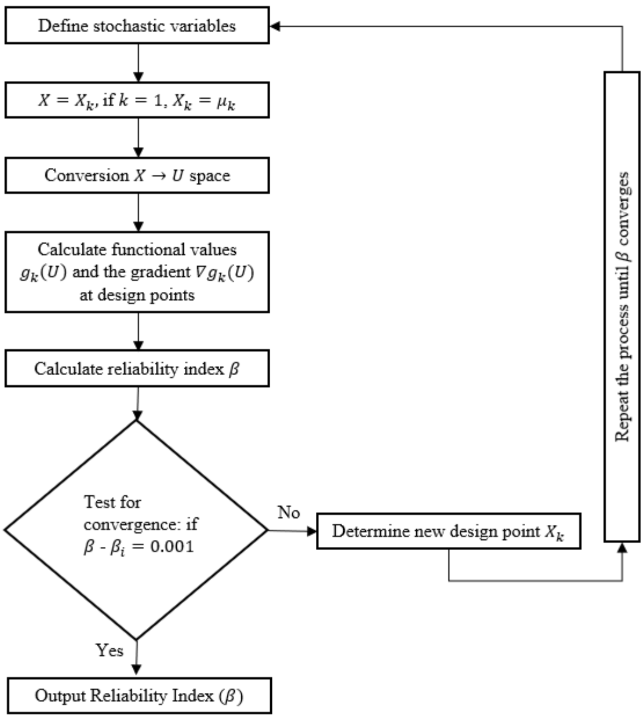

Figure 29: FORM process flowchart

4.11.1 **Validation of FORM**

To validate the FORM, simple hypothetical truss structures, as shown in Figure 30 was used for the analysis. A comparison of the reliability index obtained by Direct Simulation (DS) through MCS and results obtained through a combination of the response surface methods (RSM) and FORM was reported (Kolios et al., 2018). The simple 3-D reference jacket model consists of 40 interconnected beam members in three levels of symmetric geometry in a series of 12 lateral and 4 vertical loads acting at the top of the structure. Four basic stochastic parameters are taking into account. These variables are presented in Table 6 and Figure 30.

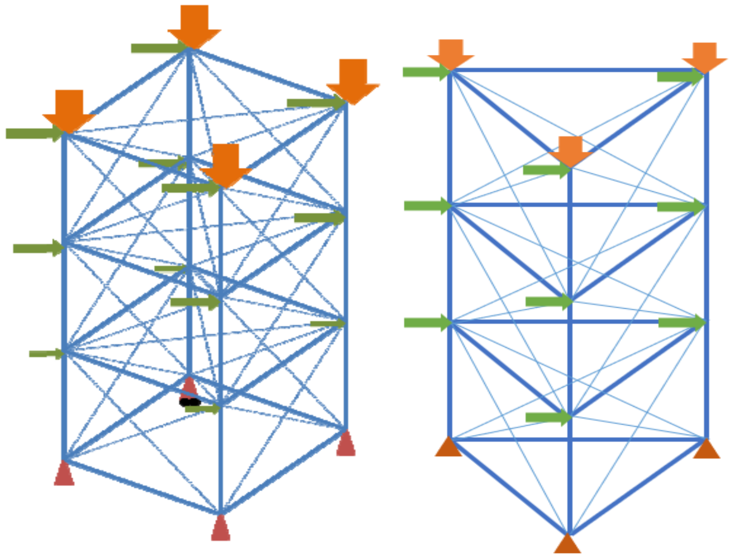

Figure 30: Reference Hypothetical Structures

Table 6: Stochastic loads consideration

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

The results obtained from the Direct simulation and the FORM procedure, executed only on members of the reference structure whose probability of failure is other than zero, are presented in Figure 31. RSM presents a higher reliability index compared to the DS method from the results obtained. However, the difference in the estimated reliability index values from both methods is consistently under 9%. Therefore, with a marginal variance of less than 10% in the results of the reliability index. We can comfortably say that there is an agreement between the methods and that the FORM method can be used for the computation of the reliability index.

Notably, the elapsed time for calculating the reliability index using RSM-FORM was far shorter than that for the conventional MCS. While the reliability index calculation using RSF-FORM took about 20-seconds, the direct simulation using MCS of a sample size of 10,000 took over twice as long. Therefore, in addition to the flexibility, the RSM-FORM method allows users to highlight sections of interest in calculating the reliability index without altering the set-up of the sample space. It is also time-saving, which will be beneficial to the industry.

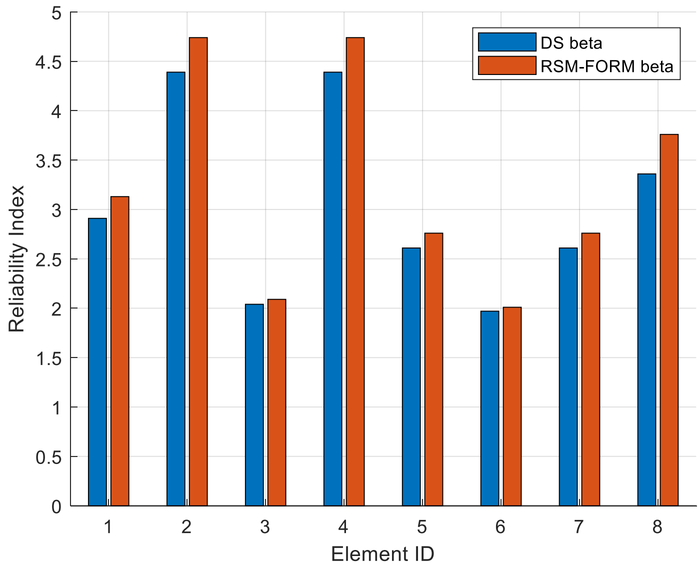

Figure 31: Stochastic Loads Consideration for Validation of FORM

To further ensure the validity of the RSM-FORM method of computing the reliability index, an additional case study with a different type of hypothetical structure was analysed and reported, using the same parameters described in Table 6. The hypothetical structure is a 3-D, 3-legged jacket model consisting of 30 interconnected beam members in three levels of symmetric geometry supporting a series of nine lateral and three vertical loads acting at the top of the structure. A summary of the results is given in

Table 7.

The results obtained from the exercise shows close agreement, between the RSM-FORM and the DS method, with a consistent variance of less than 6%, which falls within the acceptable limits of validation in academic research work. Hence, this justifies the applicability of this method to our model offshore wind turbine jacket support structures.

Table 7: RSM-FORM Vs MCS Reliability Index Results

Sample Size	MCS (10000)	MCS (100000)	RSM-FORM
Time Elapsed	1min	3min	20s
Element ID	Beta	Beta	Beta
1	2.42	2.47	2.69
2	3.94	4	4.85
3	1.73	1.76	1.81
4	3.92	3.99	4.84
5	2.1	2.14	2.29
6	1.65	1.69	1.73
7	2.33	2.36	2.51
8	2.91	2.95	3.32

The results in Figure 31 also shows a 37% improvement in the remaining life of the structure using the RSM-FORM approach. Notably, the statistical distributions used to represent the stochastical variables in this case study are normal distributions. However, the RSM-FORM methodology is also able to accommodate non-normal statistical distribution

The results presented above is a parametric study of the structural response of a hypothetical 3-legged and 4-legged OWT jacket support structures. The analysis was performed using the same structural materials and design loads. The results showed that the 4-legged jacket structure has higher reliability values, and so it is expected to be more durable. This result does make sense given the conservatism in the design and redundances. However, the 3-legged structure also reported overall reliability greater than the required minimum reliability of the structure.

The 3-legged jacket structure is about 17 percent less in structural mass and about 25 percent less in welded joint (Chew et al., 2014), so it might seem more cost-effective. Lower reliability values would also suggest the structure would require more inspections compared to higher reliability values. More so, other factors need to be considered in determining the most cost-effective structure, such as the water depth, cost of inspection, inspection intervals, etc. Therefore, in terms of reliability, the 4-legged structure is more reliable. Concerning cost efficiency, additional field data would be required to ascertain.

4.12 Summary

This chapter has presented a detailed report of the modelling of the loads and capacity of a typical offshore structure. The different classes of loads and their loading patterns were highlighted. Giving the roughness of the wind and wave and some properties of the material such as Young's modulus tends to constitute an uncertainty. Therefore, their parameters are characterized by their statistical distributions under normal and extreme conditions. The process of the development of a parametric FEA model of a typical OWT supported by the jacket type structure on a pile in ANSYS was discussed. To properly depict the stochastic conditions, the distributions of the load and capacity parameters were considered in the model. Nevertheless, the procedure was only for a limited range of conditions, i.e., parking and operating load cases as defined by the IEC61400-3 were considered.

Finally, a validation of the first-order reliability method (FORM) was presented. The hypothetical 4-legged structure was used to compare the results of the reliability index using direct simulation through MCS and to use the probabilistic response surface method (RSM) and FORM. The result shows a close agreement with a variance of about 9%. Therefore, implying that the RSM and FORM method is more sensitive, with reduced computational time compared to MCS. Therefore, RSM-FORM is suitable for calculating the reliability index in the shortest possible time, which could be significant in terms of the cost of project delivery to the industry.

5 STRUCTURAL RELIABILITY ASSESSMENT OF AGEING OWT STRUCTURE FOR LIFE EXTENSION

5.1 Introduction

This chapter aims at applying the proposed framework as described in chapter 3 of this thesis to an ageing jacket support structure to extend the service life. It is imperative to note that OWTs are designed to withstand 20 years of operation with minimum maintenance requirements, and most of the installed OWT jacket support structures are well within their designed service years.

However, Wind Europe projections show that a significant amount of offshore wind turbines would reach their design service life, usually 20 years in the next decade. Consequently, the wind energy industry would have to prepare for the future challenges such as profitably inspecting and maintaining ageing offshore assets, structural integrity assessment, and decommissioning the non-profitable assets. Wind farm owners and service providers would be required to decide to extend the life of the structure or decommission the structure considering economic, safety and legal aspects of the project. Life extension decisions are complex with limited experience to date. Therefore, this thesis seeks to propose a reliability assessment methodology that would serve as very useful to both operators and duty holders in conducting the reliability assessment and making an informed decision.

Therefore, this chapter shall focus on presenting the applicability of the proposed non-intrusive method of the reliability of OWT jacket support structures. This study shall use well-established references and models within the wind energy-related research area, such as the NREL OWT model (for loads) and the OC4 jacket support structure (for geometry and materials). These references will form the basis of the analysis of the jacket type OWT structure, as they have also been used in several highly cited academic research studies. The results obtained from this study shall be compared with standard design criteria. This thesis shall also report the design and reliability assessment of the initial structure as well as the structure for life extension.

Most of this work has already been submitted and published by a reputable journal.

5.2 Reliability Assessment

5.2.1 Project Objective

The objective of this project is to perform a non-intrusive reliability assessment for an ageing jacket type OWT support structure under stochastic environmental loads.

5.2.2 Data Collection, Data Screening and Fatigue Assessment Method Selection

The data for the initial design was obtained from the NREL reference OWT model (for loads) and the OC4 jacket support structure (for geometry and materials). It is assumed that all these data, including any structural modification, design standards, inspection and incidence reports that impact the structure, will be documented and made available during the reliability assessment for life extension. The initial target life with minimal maintenance is 20 years. The reliability assessment for life extension will commence after the 20th year, provided no major accident, as it can risk the structure's continuous operation.

Generally, the selection of the fatigue method is dependent on the available fatigue data. For the initial design where no cracks or dents are expected, the S-N method can be deployed. For the ageing structure, it depends on the available data from the inspection. The fracture mechanics method would be most appropriate if there are details of fatigue crack data and or corrosion degradation. However, given that there are no detailed data on fatigue or corrosion, the S-N method would be used for this study.

5.2.3 Development and Simulation of FEA Models

The data for the initial design and the life extension was obtained from NREL and OC4. The degradation model was based on the initial design and a corrosion model. Figure 32 illustrates the flow chart for performing parametric FEA.

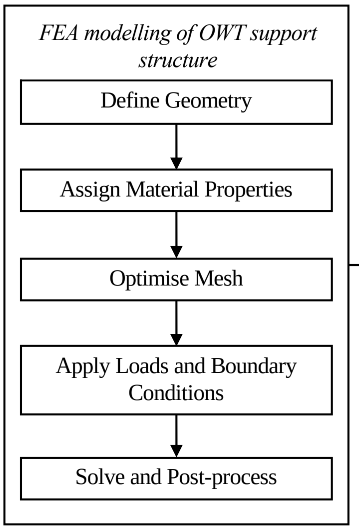

Figure 32: **Parametric FEA model Flow chart**

5.2.3.1 **Define Parameters**

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

5.2.3.2 **Generate Geometry**

The wind turbine model considered in this study is the NREL 5MW OWT (Jonkman J, Butterfield S, Musial W, 2009) which NREL developed. The jacket support structure model used in this study is the OC4 jacket support structure (Vorpahl et al., 2013) which was designed for a reference site with a water depth of 50 m (Fischer T, de Vries W, 2010). The four-legged jacket support structure has four levels of X-bracing, a corresponding mud brace, and four central piles with a penetration of 45 m (Vorpahl et al., 2013). The structure is made up of an interconnecting circular tube frame and is joined together via 64 welded connections. The height of the jacket from the top of the TP (transition piece) to the mudline is 70.15 m, and the top of the hub height to the MSL (mean sea level) is 90.55m. The piles are grouted to the jacket legs, while the transition piece between the tower and the jacket is a concrete block penetrated by the top part of the jacket legs. Table 8 summarizes the properties of the jacket structure

Table 8: Properties of Jacket members (Vorpahl et al., 2013)

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

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

5.2.3.3 **Define and Assign Material Properties**

The primary parts of the jacket support structure are mainly made up of steel materials. The steel material used for the design of the 5MW NREL OWT on OC4 jacket is the S355NL steel plate, which has been widely used for OWT structures due to its high weldability (DNV GL AS, 2016; Igwemezie et al., 2018). This class of steel follows isotropic elastic behaviour, and its physical properties are given in Table 9.

Table 9: Properties of the S355NL structural steel (Vorpahl et al., 2013)

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

The transition piece is a steel-concrete configuration. Grout (of different types and composition) is also used for pilling. The steel material data was obtained from the reference OC4 project (Vorpahl et al., 2013). The grout material data were obtained from the Ducorit data sheet (Ducorit, 2013), a representative for most OWTs. The properties of both the transition piece and grout are summarized in Table 10.

Table 10: Concrete properties for grout and transition piece (Ducorit, 2013; Vorpahl et al., 2013)

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

The soil model consists of three layers of sand, i.e., loose, medium and dense sand, of which material properties are listed in Table 11. The Drucker-Prager model (Drucker DC, 1952) can well describe the soil material, which is mainly dependent on pressure and has been widely used in soil modelling. According to the Drucker-Prager model, the yield strength of the soil, $\sigma_{y,s}$, can be expressed as functions of the internal friction angle φ and the cohesive value of c using the equation below:

$$\sigma_{y,s} = \frac{6c\cos(\varphi)}{\sqrt{3}\left(3 - sin(\varphi)\right)} \tag{5.1}$$

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

$$C_f = tan\left(\frac{2}{3}\varphi\right) \tag{5.2}$$

The properties of the soil used for this study were adapted from (Jung et al., 2015; R. Obrzud, 2010). The depth of the third layer of the soil, the dense region, was assumed to be 36 m to achieve realistic soil conditions. Soil properties per layer are given in Table 11.

Table 11: Properties of the layers of sandy soil (Jung et al., 2015; R. Obrzud, 2010)

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

Depending on the location and the soil properties, the resistance capacity of the soil would vary. This might also affect the overall reliability of the structure. In the next chapter, this thesis will study the sensitivity of the reliability index to different types of soil and soil properties.

5.2.3.4 **Define Element Type and Generate Mesh**

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

Mesh convergence studies were conducted to establish the most appropriate mesh sizes. To well control the mesh, seven meshing parameters are defined for seven parts, i.e. (1) Soil body, (2) grout body, (3) pile body, (4) jacket leg, (5) jacket body, (6) transition piece and (7) tower. The element sizes were refined, and the percentage difference in the maximum von-Mises stress and the total number of nodes were recorded. In this case, a horizontal force is applied to the top of the jacket structure, and the result for mesh refinement performed are presented in Table 12.

The maximum von Mises stress obtained is converged in refinement 2, as it has a relatively low percentage difference of (0.3%) compared to further refinement. Therefore, the meshing parameters used in Refinement 2 are deemed appropriate for this study. The FEA model of the structure based on the created mesh is depicted in Table 12 and Figure 33.

Table 12: Results from mesh convergence study

ID	Element Sizes of the respective bodies (M)							Total number of element	Max Von-Mises stress (MPa)	% Diff
	Soil	Grout	Pile	Jacket Legs	Jacket Body	Transition piece	Tower			
Meshing 1	3.5	1.4	1.4	0.5	0.7	0.7	0.7	38023	3.4537	36
Refinement 1	5	2	2	0.5	1	1	1	5282	1.8836	0.35
Refinement 2	4.5	1.8	1.8	0.5	0.9	0.9	0.9	6501	1.8894	0.31
Refinement 3	4	1.6	1.6	0.5	0.8	0.8	0.8	7735	1.8961	44
Refinement 4	3.5	1.4	1.4	0.5	0.7	0.7	0.7	38023	3.4537	

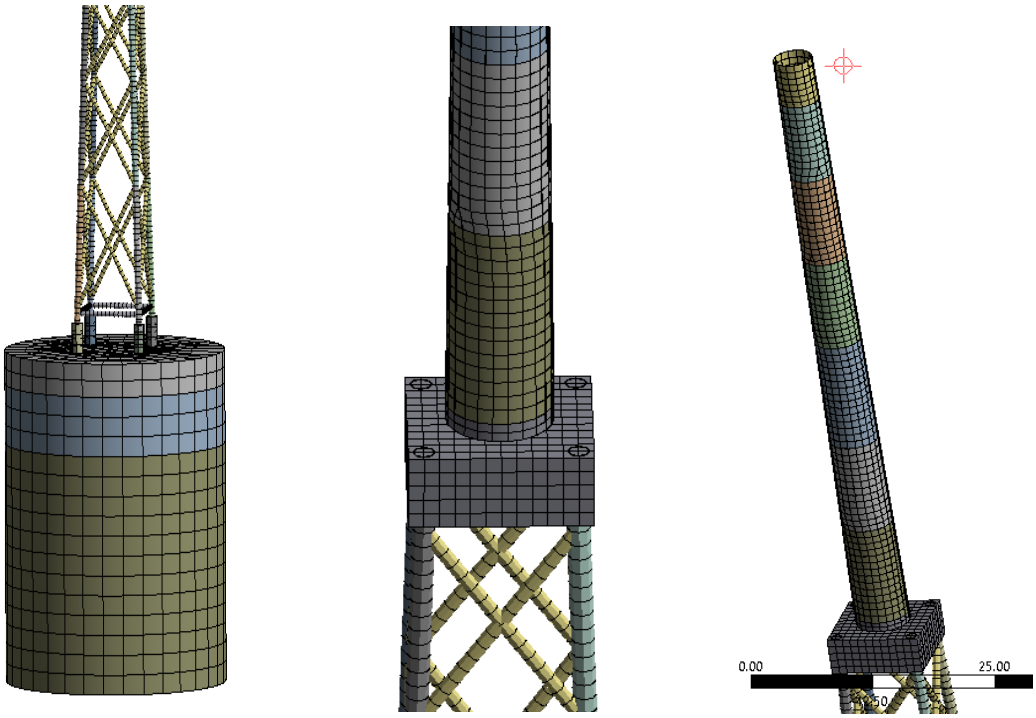

Figure 33: **Meshed FEA Model**

5.2.3.5 **Apply Boundary Conditions**

The main loads acting on an OWT's support structure are mainly the aerodynamic and hydrodynamic loads. The aerodynamic loads are applied to the top of the jacket as force and moment, as seen in Figure 34. The aerodynamic loads used in this study were obtained from the fatigue and extreme load data for 5MW NREL WindPACT design load cases. The wave load is applied as pressure, which enables the automatic update of the loads in case of a modification of the diameter of the steel members.

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

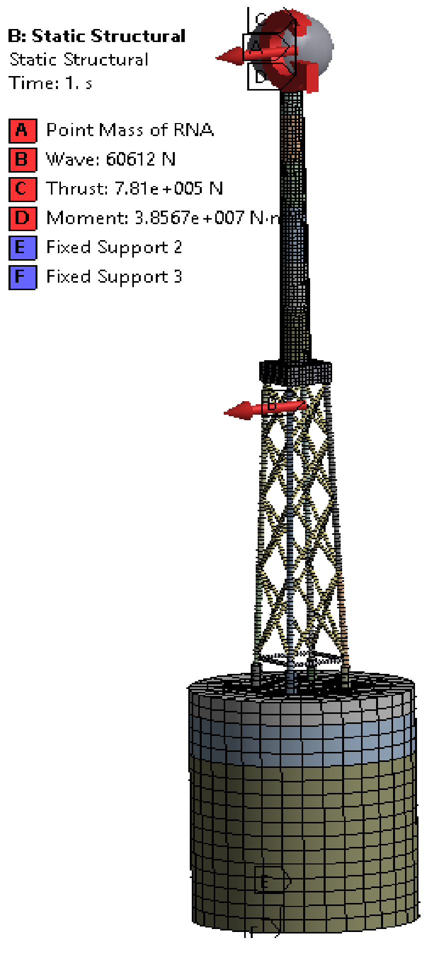

Figure 34: **Boundary conditions of the Model (NREL 5MW OWT)**

5.2.3.6 **Solve and Post-Process Results**

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

5.2.3.7 **FEA Model Validation**

Two base cases that have been reported in the literature (Jonkman et al., 2009), i.e. the deflection analysis and the modal frequency of the NREL 5 MW OWT on OC4 jacket support structures, are considered as benchmarking studies, comparing previously published results to values obtained from this analysis.

5.2.3.7.1 Validation of Deflection analysis

This scenario assesses the total deflection of the OWT support structure in static analysis. In the reference literature Jonkmann (2013), four case studies were performed for the deflection analysis, which consists mainly of an application of a thrust of 2MN and 4MN, with and without the weight of the RNA. The displacements of the RNA and tower base are measured with reference to the location of the RNA and tower base centre under unloaded conditions, respectively.

As can be seen in Table 13, good agreement is achieved, with a maximum relative difference (+6.24%) observed for the displacement at the tower base. This result confirms the validity of the present FEA model.

Table 13: Deformation result of static analysis of 5MW NREL on OC4 jacket structure.

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

5.2.3.7.2 Validation of Modal analysis

This case study also assessed the natural frequencies of NREL 5MW OWT on the OC4 jacket support structure. The modal frequencies calculated from the present FEA model are compared against those reported in (Jonkman et al., 2009), and the comparison results are presented in Table 14. Reasonable agreement is achieved, with a maximum relative difference (12.38%) observed for the 2nd fore-aft mode. This further confirms the validity of the developed parametric FEA model.

Table 14: Modal analysis results, comparing the mode frequencies of structure and the reference values

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

5.2.4 Loading Simulation and Structural Analysis

Design or inspection data for loadings over the service life of the structure are collected and screened before applying it to the FEA model. The desired loads include all dead loads, operational loads, environmental loads, permanent and variable loads. This study has considered dead loads, live load, wave load, wind load, thrust and tilting moments.

Variable and permanent loads were defined in Table 10Table 11, and Table 9. The variable loads such as aerodynamic (wind thrust and tilting moment) load were applied as Uniformly Distributed Loads (UDL), both at the fore and the top of the tower. Dead load such as the weight of the RNA was applied as a point load at the top of the tower. The mass of the tower was applied as UDL at the top of the tower. The wind and wave were applied in the same direction. The load cases considered are those of fatigue and ultimate loads. Tables 4.1 and 4.2 show the typical aerodynamic loads values used for this study.

5.2.5 **Stresses Analysis**

Several linear multistep of static stress, buckling, modal and fatigue analysis was performed. The wind and wase loads were applied in the same direction. The model is fixed on the z-axis. The outcome of the simulation is shown in the isometric diagram in Figure 35.

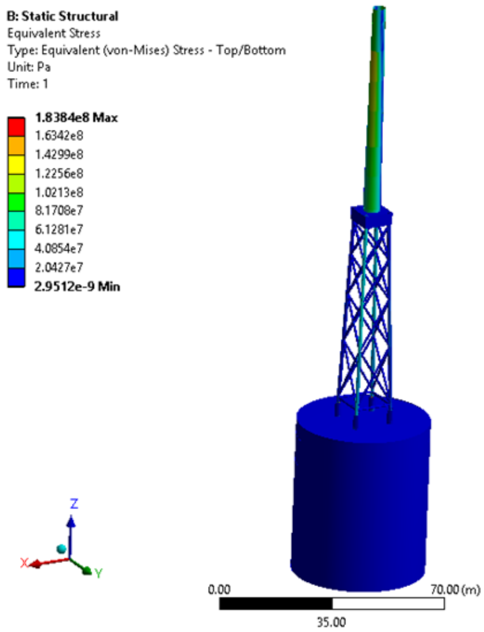

Figure 35: Isometric View of the 3-D Model

5.3 Reliability Assessment Results and Discussion

Previous sections of this thesis have presented a framework for the reliability assessment of complex support structures has been developed. The implementation of the framework from application to the NREL 5MW OWT jacket support structure is presented here for the multiple limit states introduced earlier and accounting for several stochastic input variables. Case studies are performed to validate the main components of this framework, i.e. the FEA model and the FORM has already been presented in sections 5.2.3.7 and 4.11.1. Sequel to the validity of the model and methods, the following case studies were performed.

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

5.3.1 **FEA Analysis Results**

Table 15 presents a summary of FEA results obtained from the multistep simulations performed. The result for the ULS (Stress and buckling) and SLS (Deflection and frequency) suggested that the maximum values of the critical details of the structure all fall between the acceptable design criteria.

Table 15: Summary of FEA Analytics Results

Limit State Analysis	Standards requirement	Safety Criteria	FEA Simulation Results	Computed Safety Index
Buckling	$g_b(x) = L_m - L_{m,\_min}$	$L_m > 1.4$	8.4	5.67
Deflection (m)	$g_d(x) = d_{allow} - d_{max}$	$0.9157 > d_{max}$	0.7338	10.2
Modal frequency (Hz)	$f_{1p+5\%} \leq f_{1st} \leq f_{3p-5\%}$	$0.212 \leq f_{1st} \leq 0.328H_z$	0.25692	11.27
Ultimate (MPa)	$g_u(x) = \sigma_{allow} + \sigma_{max}$	$323\ MPa > \sigma_{max}$	158.65	3.89

5.3.2 **Results of Load Cases**

5.3.2.1 **Ultimate Load Case**

The reliability assessment results obtained from the ultimate load cases, which mainly depend on the buckling, deflection and ultimate stress analysis of the OWT jacket support structure, are presented in Figure 37**.** It should be noted that the framework calculates reliability indices for each component of the structure. Hence different values are obtained for each component. For illustration purposes, only the values of the minimum reliability components are reported in the subsequent sections of this paper. As evident, the multi-attribute reliability assessment exercise performed on the structure shows that the model, as developed, and the stochastic variables considered, satisfies the recommended reliability assessment criteria, as the reliability index for the various limit state are within design thresholds, as they all exceed the target reliability set at 3.71.

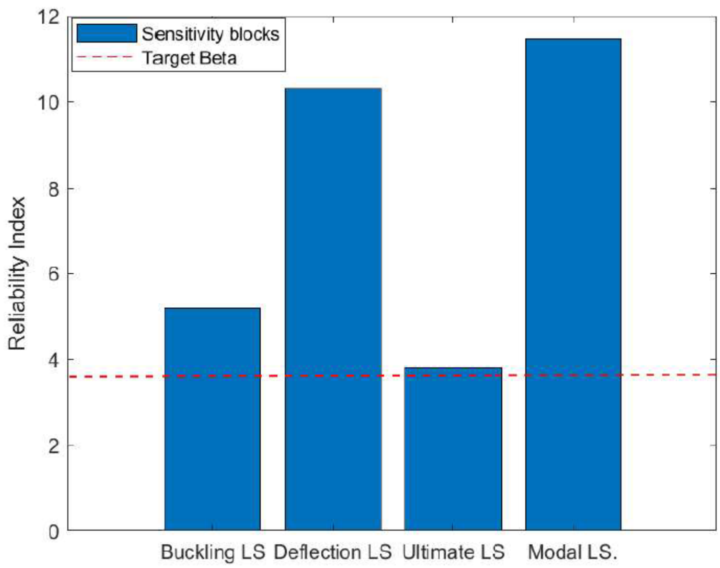

Figure 36: Reliability Index of factored multi-criteria limit state

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

5.3.2.2 **Fatigue Reliability Assessment**

Sequel to the completion of the FEA model validation process, the model developed is applied to assess the fatigue reliability of the NREL 5MW OWT OC4 jacket support structure. The fatigue reliability assessment study performed is based on the well-known fatigue limit state method described earlier, which follows the S-N approach to determine the fatigue life of the structure. The choice of an appropriate S-N curve is crucial. Ideally, depending on the structure detail, an appropriate S-N curve is selected. Nevertheless, given that the most critical components of the structural system were reported to be below the mean sea level, therefore, the SN Curve used for studying the fatigue life of steel structure in seawater for $N > 10^6$ is utilized. According to DNV-OS-J101 (DNV 2014), the parameters of the standard S-N curve, such as the intercept (A) and slope (m), is given as 15.606 and 5, respectively. The result obtained from the fatigue life analysis is presented in Figure 38, showing a comparison of the reliability index deterioration with the target reliability index.

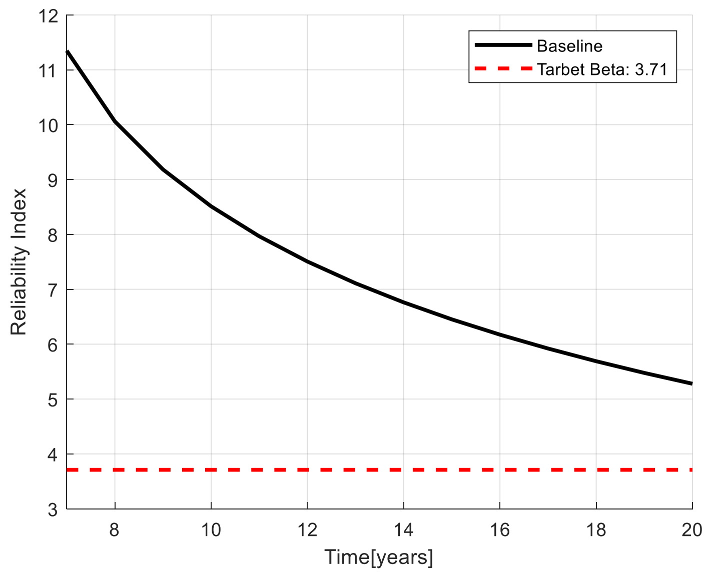

Figure 37: Fatigue reliability assessment

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

Additionally, a case study on CoV's impact of the stochastic variables on the fatigue reliability assessment was performed. To do the CoV case study, two values of CoV are considered, i.e., +5% and -5% of the fatigue life. Figure 38 presents the results of the assessment shows that the calculated fatigue reliability over a 20-year service life of the structure. The result suggests a sensitivity of the reliability index to the value of CoV. Higher values of CoV implies greater uncertainties of the stochastic parameters, which result in a decrease of the reliability index.

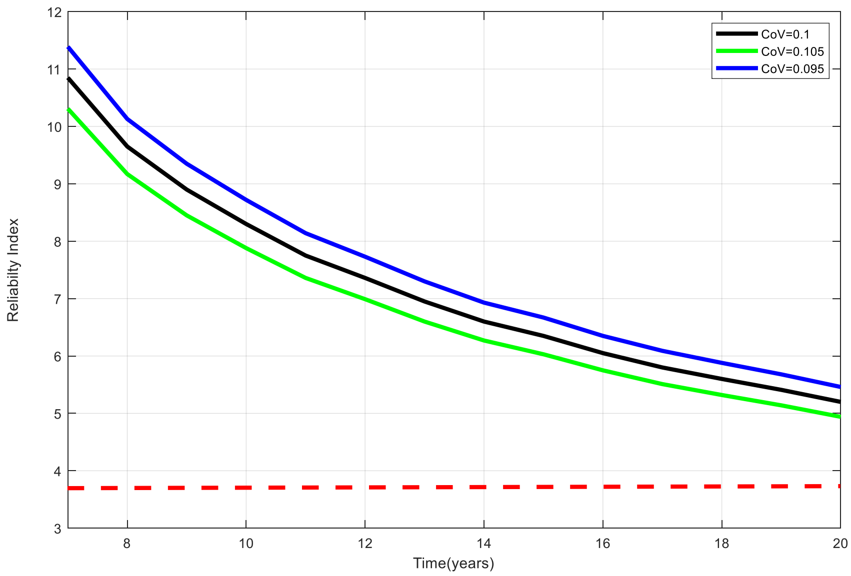

Figure 38: Fatigue reliability assessment with a varying coefficient of variation (CoV)

5.3.3 Sensitivity Analysis

5.3.2.3 **Variation of Mean and Standard deviation of load and resistance parameters**

This study performed general sensitivity analyses on all the variables, and the results were reported. To do this, the stochastic parameters' values gradually vary the mean values and standard deviation of each stochastic parameter, one at a time by 20%, and evaluate the reliability variation for comparison with the defined threshold. The result of the sensitivity analysis is reported in Figure 39. The results obtained from the sensitivity analysis clearly show that the mean values of the tilting moment (M) and the wind thrust (F) are the most influential parameters in the fatigue reliability analysis, with the tilting moment qualifying as the most sensitive parameter.

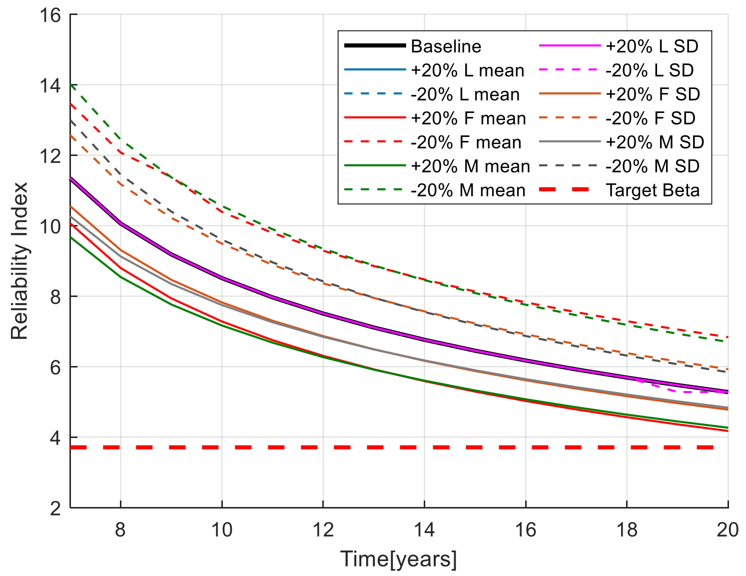

Figure 39: **Sensitivity analysis of statistical parameters**

5.3.2.4 **Soil Structure Analysis**

This study also analysed the effect of the soil on the structure during loading. Jacket structures are normally connected to the pile legs by grouting with concrete. In this study, all the results presented were based on the FEA simulation results obtained from the model that includes the soil. However, the soil was suppressed, and the analysis was repeated. The results obtained from the study are present in Figure 40.

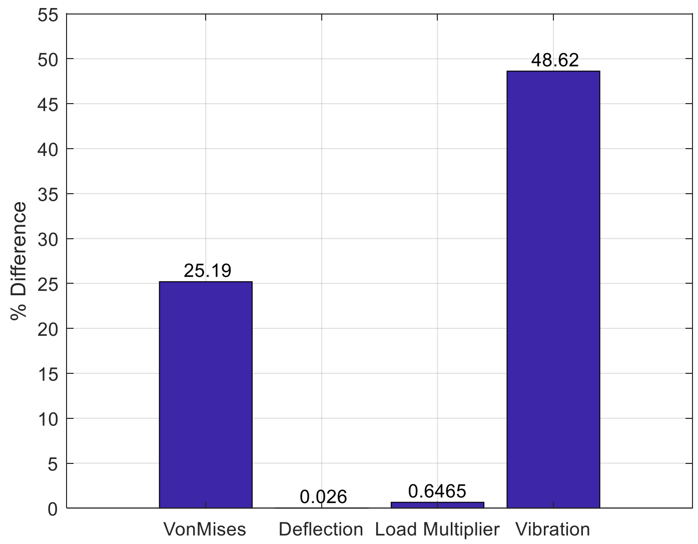

Figure 40: Soil-structure interaction sensitivity analysis

The results in Figure 5.7 clearly show that incorporating the soil model into the jacket model affects the response of the model. The results of the modal frequency tend to be the most impacted by the addition of the soil parameters to the jacket model. The response for the buckling and deflection analysis shows a lower effect on the inclusion of the soil model.

The impact of the soil interaction with the model does not significantly affect the load multiplier and the total deflection. However, the FEA results for deflection at the mudline sections of the jacket model show a higher deflection reading when the soil is incorporated. This also suggests that the soil properties can easily become a significant factor for the reliability of the structure, which further suggests that the reliability of a structure could vary from one location to the other. This section will be studying this effect. But, due to the limitations of obtaining credible named location soil data, soil data from the available literature were used in this analysis. However, this parametric study aims to show the applicability of the methodology rather than the accuracy of the results.

5.3.2.5 **S-N Curves**

This study took cognisance that there are several types of S-N curves dependent on the environment and the joint classification, on which results in the varying values of the negative inverse slope m and an intercept a. According to DNV-RP-C203 (2016), all tubular joints are assumed to be class T. Therefore, the OWT jacket support structure, which is mainly made up of tubular joints, will be treated hence as class T joints, class of joint was not captured in EUROCODE 3 (2016) and so becomes difficult for comparison. However, HSE (HSE, 1999) did due diligence to the welded joint classification presents details of the various joints and the negative slope and intercepts values. Based on the definition of the joints by HSE (1999), joint class C which the most appropriate description of a jacket structure and is mostly used in the experimental study of fatigue. The class C S-N curve will be used for this study

5.4 Life Extension Assessment

5.4.1 Reliability Assessment of Corrosion Fatigue Phenomena

Corrosion has been identified as a significant factor that influences the deterioration of offshore structures. There are several models for modelling used for describing corrosion, as presented in the earlier sections. This section will be studying the sensitivity of fatigue life to corrosion. The corrosion model assumed in this study is uniform corrosion, where the structure loses its thickness uniformly per annum. The rate of corrosion of steel in seawater is within 0.01 to 1.2mm. This study would apply a less conservative approach and use 0.3mm/year as recommended in Melchers (2009). Hence, the structure will be reproduced by reducing the cross-sectional thickness by 0.3mm/year on the structural properties defined in table 5.1, and then the simulation process repeated. The reliability analysis process described in figure 4.6 will be repeated to compute the reliability index.

Some assumptions would be made to further support the theory to buttress the impact of corrosion on the structure. These includes:

1. There is no corrosion on the structure in the first seven(7) years of service in the life of the structure. This is because the structure would have still had its corrosion protection mechanisms in place within this period.
2. The load and capacity models are precise, and there were no extreme loading conditions, accidents or any event that would have impacted the remaining life of the structure.
3. The corrosion protection mechanism, such as coating, cathodic protection, etc., is not renewed, so the corrosion degradation is unhindered.

The result of the remaining life of the structure under corrosion attack after year seven(7) is given in Figure 41. the result shows that the reliability of the structure is sensitive to corrosion. Thus, an increase in the annual rate of corrosion reduces the reliability of the structure

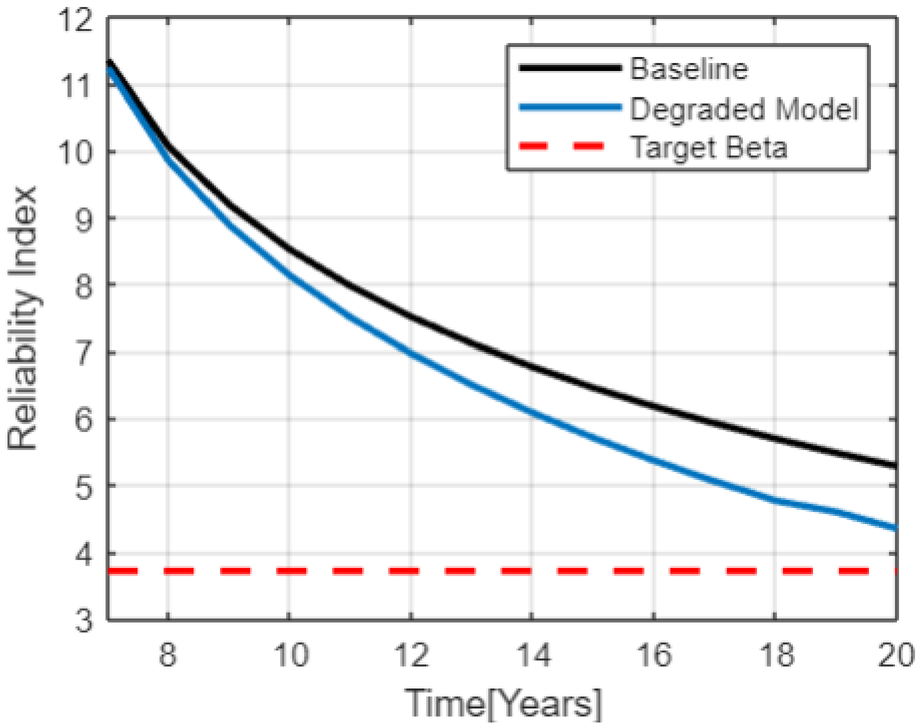

Figure 41: Effect of Corrosion on the Remaining life of the structure

The results obtained suggest that the reliability index of jacket structure under uniform corrosion attack remained higher than the target reliability. This means the structure is still operating within the safe region and thus can be considered for a life extension, with appropriate inspections and mitigation of flaws through either repair and or replacement wearied out critical details of the structure. Although the results suggest that the reliability index is still greater than the target minimum target reliability for the first 20 years without any intervention, it is more susceptible to failure giving that its reliability index is now very close to the minimum target reliability.

Depending on the quality of maintenance and, or repair, the reliability of the structure is expected to improve thereby, allowing the operators of the structure to continue putting the structure in service.

5.5 Summary

The present chapter has presented the non-intrusive methodology of reliability assessment of a jacket type OWT structure under highly stochastic variables. The assessment outcome shows that the structure's reliability is based on the load and resistance ratio. Therefore, uncertainties in either the load or resistance of the structure would impact the outcome of the reliability. The results of the reliability index calculated for all limit states indicates the structure maintains a higher reliability index than the minimum acceptable reliability index of 3.71 according to DNV-OS-J101 (DNV 2015). The fatigue assessment also presents sufficiently high reliability at the end of the first 20 years in service, which implies the structure is safe and can undergo life extension with a high probability of success.

The life extension is computed based on the reliability index of the most safety-critical elements of the structural details, in this case, submerged in water. It is important to note that each structural member's reliability index may differ, so it is advisable to repeat the computation for every structural member. Depending on the outcome of the reliability assessment of other structural details, which may require further strengthening to increase their fatigue life by employing fatigue improvement techniques such as grinding, hammer peening or TIG dressing of joints and weld, and addition of stiffeners, braces, and grout for the strengthening of the structure.

The service life of the jacket can be extended if there are a robust inspection and mitigation plan. Mitigations should be executed in liaison with the on-site inspection to verify the feasibility offshore, and the strengthened structure satisfies all limit state criteria. It is crucial to have an inspection plan that accounts for regular maintenance intervention. The intervals for inspection should consider a high probability of fatigue cracks or material wastage, given the structure has spent its initial design life. The next chapter shall provide details of the inspection and mitigation of the jacket type structure.

6 INSPECTION AND MITIGATION OF AGEING OWT JACKET SUPPORT STRUCTURES

6.1 Introduction

Offshore structures are usually designed for economic operation throughout their design life, in strict compliance with recognized industry standards and must comply with relevant shelf state requirements. Given that structural deterioration is due to corrosion, fatigue is very likely in offshore operations. So in some cases, they are considered in the design phases with a certain level of informed assumption and uncertainty modelling. In principle, the installed structure is assumed to be operated in line with the hypotheses considered in the design phase. Otherwise, the deterioration phenomenon would result in the reduction of the performance of the structure beyond its limit states (Jean et al., 2003).

It is important to control the deterioration process to ensure the structure operates within the appropriate limit states throughout its service life. Suppose any structural detail requires mitigation, implementation of such recommended mitigative actions in the form of re-enforcement, repair, or replacement of critical structural details. In the offshore industry, inspection has been the most effective means of controlling deterioration. The primary purpose of an inspection is to analyse the risk and probability of failure and put adequate controls to ensure the consequence of failure on the environment, assets, personnel, or reputation of the organization is as low as reasonably practicable (ALARP).

One significant advantage of inspection is that it helps reduce the uncertainty about the current state of the structure. For instance, inspection results can be used to verify fatigue analysis results during design. Fatigue analysis is usually performed with uncertainties, such as identification of cracks and sizing of the cracks. However, inspection results can provide such data to validate the design and narrow the gap of uncertainties. It should also be mentioned that inspection results also have their associated uncertainties. This is principally due to the ability of the inspection equipment or technique not able to capture a defect that is less than a certain range. In a broader sense, an inspection can be said to be an act of surveying the structure and collecting necessary data ISO 19901-9 (ISO, 2017)

Additionally, monitoring the structural condition of offshore assets by planned inspections will provide decision-makers with enough information to ascertain the structure's reliability and confidence in the structure living out its service life. Accordingly, inspections are to be planned such that every visit to the offshore asset is maximized. Various inspection policies depend on the specifications and the composition of the wind farm. Inspection policies are developed by implementing different inspection techniques, considering their efficiency, associated efficacies, and inspection time. Therefore, this thesis aims to propose an optimal risk-based inspection plan that maximizes the information obtained from the inspection throughout the service life of the structure.

State-of-the-art offshore structure inspection policies as contained in design standards provide guidelines for the inspection, detection, maintenance, and repair of flaws observed during the inspection. While the repaired or replaced sections are checked for integrity, their impact on the reliability of the overall structure is not usually considered. This thesis sought to extend the inspection methodology to include an on-the-spot assessment of the impact of a repair intervention on the reliability of the overall structure by developing a non-intrusive, reliability assessment methodology that can be used to create and simulate the reliability of the overall structure based on the outcome of the intervention. This extra step of assessing the reliability during and after Inspection Maintenance and Repair (IMR) is necessary. It provides an update of the residual life of the structure as a result of the repair or maintenance activity and provides data for planning future maintenance or repair works.

Most recently installed OWTs are lighter and smaller in size but larger in power generation capacity than their predecessors. There is also growing speculation on the possibility of retrieving existing turbines and replacing them with more advanced turbines using the same support structure. In these circumstances, the outputs from this thesis will provide a framework for the assessment of the reliability of an aged structure, which can also be used to assess the impact of any prospective repairs or improvements on the global reliability of that structure.

Inspections are an effective means of controlling the deterioration of an offshore asset. They may also impact the operation considerably, giving the time and resources required to execute the inspection. Therefore, it is important to plan the inspections to ensure a balance between the pros and cons of inspection, such as downtime and other economic consequences. Generally, inspection planning would require answering some basic questions such as when to perform the inspection? Where to perform the inspection (detailing the exact location)? How to perform the inspection (methodology)? What remedial actions are required after the inspection?

Several design standards have stipulated their requirements for structural integrity management, which includes the inspection process cycle, as shown in Figure 42. The process cycle comprises data collection and retention from previous and present inspection, analysis of the data based on findings, updating the long-term inspection plan, and developing the scope of work for the inspection. DNVGL (2015) and ISO19902 (2007) also defined some types of inspection, such as Baseline inspection, which is usually done just after installation. Based on the design inspection plan, periodic inspection requires a special inspection to monitor a known defect. Finally, unscheduled inspection is needed after a major environmental event such as hurricane or Tsunami.

Figure 42: **Inspection Cycle Based on ISO 19909 (2017) Model**

6.2 Inspection Philosophy

Inspection of offshore structures is expensive with its associated safety implications, especially if divers are used. Thus, it is not possible to inspect every structural component at every inspection. Therefore, the choice of inspection method and the inspection frequency are crucial in planning an efficient, cost-effective, and robust inspection plan.

Operators and regulators of offshore exploration initially relied on time in service for inspection, which requires an annual inspection of the structure for fatigue cracks. This was considered too prescriptive in some instances and insufficient in others (Ersdal et al.,2020). The annual inspection was able to detect some defects but had minimal impact on the reliability of the structure, especially for relatively new structures. As such, the need for a more effective plan led to condition-based inspection. Condition-based inspection uses the reports from the previous inspection to determine inspection intervals, however, prioritizing inspection became difficult due to criticalities and complexities of structural details, e.g., welded joints. With the introduction of probabilistic methods, the structural details are ranked according to their criticality and inspection is focused on high-risk areas only.

According to DNV-OS-J101 (DNV 2015), depending on the design philosophy adopted for the OWT, the annual inspection of a fatigue crack is required for critical structural details may be waived if safety factors were the basis of the design. It corresponds to an assumption of no access for inspection, which implies there may be no need for fatigue crack inspection. However, if smaller safety factors were applied for the fatigue design, inspections may be required, and the surveyor is expected to inspect the structure through its service life

Generally, the larger the safety factor, the longer the inspection interval. Depending on the quality of the inspection performed, DNV-OS-J101 (DNV 2015) recommends an inspection by eddy current or a magnetic particle inspection, is performed. The interval between inspections can be computed from the safety level expressed in terms of design fatigue factor as follows.

$$\text{Inspection interval} = \text{Calculated fatigue life} \cdot \frac{DFF}{3.0} \qquad (6.1)$$

So, for a calculated fatigue life of 20 years, if the DFF is 1, it implies checks for fatigue is required every seven (7) years, and if the DFF is 2, checks would be required in 13 years of the service life of the structure. However, if the DFF is 3, no inspection is required. The inspection is usually carried out at the site by different surveyors.

The inspection is expected to give feedback information about the jacket support structure's structural conditions, and obtained information is to be processed based on the FORM model presented in Figure 43. The reliability index is thus computed. If the value is below the acceptable limit, mitigative actions are to be performed either by repairing or replacing, which improves the reliability index above the acceptable minimum limits.

It should be noted that the higher the precision of the inspection data would imply a higher cost of the inspection. Nevertheless, higher inspection quality is more likely to ensure better performance and optimal output during operations. This scenario results in a trade-off between the operational and capital expenditures, which is still being researched to develop an optimum inspection time interval to ensure a minimum level of energy cost (Yeter et al., 2018).

Determination of the optimal inspection plan can be based on the decision tree. Figure 43 is an illustration of a typical decision tree. The decision tree considers all possible events that lead to the decision problem. The decision tree method considers every event's outcome and describes the associated cost of failure based on the probability of failure and the consequence of failure (Yeter et al., 2020). The decision tree method has been applied in several high impact research work in the area of minimising the expected cost, which considers every event in the decision tree diagram, including the cost of the inspection, failure, repair or maintenance to provide optimal inspection planning. Although this thesis does not focus on the life-cycle cost, the method applied in this study can be extended for the life-cycle cost of an OWT jacket support project.

For this study, three possible outcomes from an inspection. These shall include no detection of any damage due to corrosion fatigue, mild damage detected due to corrosion fatigue, but no repairs or replacements required and finally, severe damage detected due to corrosion fatigue and require repair. An illustration of a generic decision tree is presented in Figure 43. It shows the evolution of events and decisions corresponding to potential inspection schemes. The ability to identify an inspection plan with minimal projected cost would allow for planning for inspection with the specification of the schedule frequencies and method of inspection.

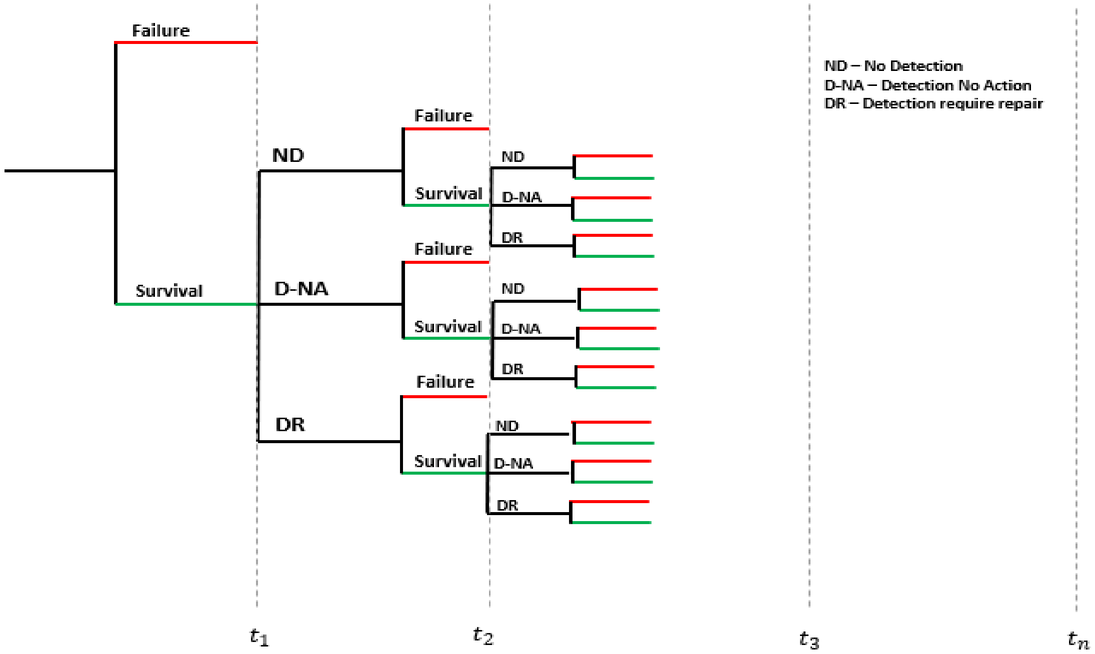

Figure 43: **Generic Decision Tree for Inspection Plan Formulation**

It should be mentioned that irrespective of the inspection quality, the structure may still fail, which is why every event is followed by either failure or survival conditions. The probability of occurrence of each event is dependent is evaluated based on the probability of detection and the probability of failure of the OWT structure. Furthermore, most offshore support structures are designed to withstand the rough load and loading pattern during the service life. Suppose the design of the support structure follows a less conservative method than the general methods provided in the standard design procedure. In that case, a lower reliability index is expected and thus a higher probability of detection of any defect during an inspection. Therefore, structures with a lower reliability index benefit more from inspection and have a higher probability of failure, risk premium, and capital cost, especially in the earlier stages of their service life (Yeter et al., 2020).

6.3 Probabilistic and Risk-Based Inspection Planning

There are several methods of conducting inspection such as General Visual Inspection (GVI), Close Visual Inspection (CVI), Non-Destructive Testing (NDT) and inline structural monitoring equipment and so on. All of these methods can produce results with varying degrees of accuracy, which can be due to the adaptability and compatibility of these methods to the structure they are deployed to inspect. For instance, some NDT methods are more suitable for surface-breaking defects and then ineffective for sub-surface defects. In the same vein, different equipment may be more precise in detecting and measuring defects of a particular orientation and size than others (Brennan, 2013). Therefore, it is recommended to conduct trial inspections to compare the performance between techniques and the most appropriate technic deployed for optimal inspection.

This section of this study aims at analysing inspection methods and propose an optimization of the data collected from inspection to evaluate the structural reliability of an offshore structure. In previous chapters, the development of an FEA model of a typical offshore wind turbine jacket support structure and the reliability assessment has been presented. The outcome of the results has all maintained a reliability index greater than the required target minimum reliability index without IMR. In this chapter, the structural reliability assessment will consider some scenarios, such as the effect of free corrosion and the impact of IMR on the system reliability of the OWT jacket support structure. These results make up the overall profile of the reliability of the structure. The overall structure's structural reliability profile is an important document. It provides stakeholders (such as service providers and owners of the wind turbine) in making investment decisions such as life extension or decommissioning.

Maintaining reliability above the target reliability index of 3.71 throughout the structure's design life is a major requirement of the scope of the inspection and the required intervals between inspections. In some instances, sensitivity analysis is used to explore the effect of different levels of reliability on the overall structural integrity, or the already achieved structural reliability to date is assumed as the acceptable level irrespective of ageing in the structure. This remains a grey area in inspection planning (Ersdal et al., 2020). For a complex structure like the jacket structure, the setting of target reliability levels must consider the effect of redundancy.

The risk-based method of inspection planning has been seen to provide optimized structural integrity. However, the intervals between inspections tend to get even longer for ageing structures compared to the commonly used condition-based or calendar methods. To some extent, it may be cost-effective, but it contradicts the common knowledge that as the structure gets older, more inspections are required. The structure would be more susceptible to failure due to degradation caused by environmental and operational loads. This is a notable drawback in the use of risk-based inspection planning for ageing structures. However, there is a need for expert elicitation and application of common knowledge in planning inspection for ageing structures to mitigate this.

Several risk-based inspection models have been developed using either qualitative or quantitative analysis of the probability and consequences of failure of a critical member of the structure. Structural reliability assessment (SRA) have in the past been a helpful tool in determining the probability of fatigue failure (Erdsal et al., 2020). Probabilistic inspection planning methods based on SRA enable optimization of inspection by prioritising the critical structural elements of the structure that are most susceptible to failure and can lead to catastrophic damage.

Optimizing the inspection schedules for structural elements with calculated fatigue lives will require the quantification of certain probabilities such as:

- The probability of detecting a given crack
- The probability of accurately sizing the given crack
- The target reliability (i.e., the minimum acceptable reliability index)

6.3.1.1 **Probability of Detection (PoD)**

Inspection is usually surrounded by uncertainty, mainly due to the probabilities of detection and sizing. PoD arises due to the high level of uncertainty of accurately detecting flaws in the structure. Conducting inspection trials have been one means of improving the PoD. However, giving the huge cost implication of performing an inspection even for small sample sizes, it becomes uneconomical to consider the inspection of every member of the structure. Therefore, only critical members of the structure are inspected (Brennan 2013). NDT is the most common type of inspection method used.

Despite the technological advancements, inspection and measuring of flaws, still has uncertainty regarding their success. The measure of this uncertainty emanates from the blind trials conducted, which are expressed as PoD and is usually associated with a confidence level.

6.3.1.2 **Probability of Sizing (PoS)**

PoS is the ability of a deployed inspection technic to size the defect or flaws on the structure accurately. PoS is less known compared to PoD but is as vital as PoD for damage assessment. Brennan (2013) presented a distribution result of a blind inspection trial conducted on a range of representative defects. The outcome shows that the approach is skewed towards underestimating the actual flaw size and only overestimate the size in a minority test. This kind of report is important to the integrity engineer, as it ensures the provision of damage estimates considering the inspection method deployed is likely to be unconservative.

6.4 Updating Reliability

The outcome of the inspection is usually subjected to uncertainty quantification due to the limitations relating to flaws detection and sizing. Thus, the quality of the inspection is measured by the PoD. The formulation of a typical PoD can be expressed as a function of the quality of the inspection, current crack size and the minimum detectable crack size. This study shall deploy the S-N method for the fatigue reliability assessment, details in section 5, to account for the whole fatigue life of the structure. The PoD curve can also be derived in terms of the reliability index rather than crack size. Therefore, the parameters of the PoD are replaced with the reliability index, associated with the detectable corrosion fatigue damage with 100% certainty, as well as the quality of the inspection (Yeter et al., 2020)

The criteria for repair can thus be deduced with respect to the reliability index, which implies that if the reliability index detected is lower than that of the current reliability index, the structure will require repairs, which should be within the cost threshold of the repair criterion of reliability index concerning fatigue damage that is detectable with 100% certainty (Yeter et al., 2020).

Therefore, the POD expressed as a function of reliability index is given as:

$$POD = e^{\frac{-(e^{\beta}-e^{\beta_{detect}})}{e^{\lambda}}} \quad ; \beta > \beta_{detect}$$

$$POD = \quad 1 \quad ; \beta \leq \beta_{detect}$$

where β is the present reliability index and β_{detect} is the detected reliability index, and it is a function fatigue damage that is detectable with 100% certainty and is assumed to be 0. Whilst λ is a function of the inspection quality.

Non-destructive inspection techniques generally perform planned inspection, for example, vibroacoustic, magnetic, ultrasonic, infra-red thermography, X-ray, or radiography computed tomographic testing. The POD increases with the increasing quality of the inspection. However, there is also an annual general visual inspection conducted for OWT and its support structure to look out for defects on critical details of the structure, including the RNA and blades of the turbine. It must be mentioned that the quality of the kind of inspection is lower compared to the planned inspection conducted explicitly for the support structure.

6.5 Inspection of a Jacket structure

Over the years, several inspection techniques have been developed for the underwater inspection of offshore assets. In the early years, divers were commonly used. However, the rising emphasis on safety and the safety of the safety challenges of operating with divers in Deepwater led to the deployment of remotely operated vehicles (ROVs) and the reduction of the use of divers. NORSOK N-005 Annex B (Standard Norge 2017) has provided a comprehensive overview of inspection techniques.

General visual inspection (GVI), close visual inspection (CVI) and flooded member detection (FMD) are the standard methods of conducting an underwater inspection. These methods are relatively cost-effective and fast. They are also able to cover most of the installation underwater, using an ROV. These techniques also do have their drawbacks. The GVI is limited to only detecting severance in members such as severe dents but are unable to detect cracks. While the CVI is limited by the concept of PoD of crack less than certain crack sizes typically, 350 – 400mm. FMD is limited to the detection of cracks through the thickness, as it relies on the detection of water penetrating the member of the structure been inspected. The remaining life of members detected by FMD has been seen to be short after detecting flooding (Ersdal et al., 2020). So there are also concerns in using this technique for ageing structures with a higher probability of cracks. Hence, it can only be efficient when if there is a thorough understanding of the joint. Therefore, further investigation is necessary for cracks detected by CVI and FMD.

Most jacket structures are designed with redundancy, and as such, failure in a member would not lead to structural collapse due to the multiple load paths, and as such, GVI, CVI and FMD techniques are acceptable. Nevertheless, this might not be exactly the same for ageing jacket structures because as the structure gets older and more susceptible to failure, there may be widespread deterioration. These techniques may not be appropriate.

The inspection of a jacket support structure is carried out by dividing the jacket into sections include the auxiliary support systems such as the corrosion protection system, to ensure the local potentials of the standard electrodes is within acceptable limits. Inspection of the components in the splash zone using the ROV is usually a challenge, except for ROVs that can look up. There has been an introduction of aerial remote operated vehicles (AROVs) or drones, which is expected to mitigate the limitations of the ROV.

The ROV has inspected the underwater structure, which will then inspect piles, scour around the foundations, and other build-ups of seabed material on the structure, which could hinder any future inspection. Inspection of the guide frames, transition pieces and members above water is also important as most of the part has been a significant concern due to corrosion (Ersdal et al., 2020). ISO 19902 (2007) has recommended a detailed inspection process to inspect a fixed offshore platform, which includes both regimes of risk-based inspection and calendar-based inspection. The standards also recommend four levels of inspection, as shown in Table 16.

Table 16 Summary of ISO 19902 Inspection process

Levels	Scope	Inspection Intervals
Level I	Above water visual inspection, below water checks on the Cathodic protection CP system	Annually
Level II	General underwater visual inspection, checking for scouring, accidental damages, cracks, and corrosion	Every 3 – 5 years
Level III	Under CVI of pre-determined areas and FMD	Every 5 years
Level IV	Underwater NDE for selected areas, detailed CVI etc.	As required

For ageing jacket structures, inspections would require a more arduous approach giving that the structure is more susceptible to corrosion and fatigue. Therefore, a greater probability of damage and widespread deterioration. However, most of the available standards do not provide specific guidelines for inspecting an ageing structure. This may be a considerable challenge in the coming years giving more structure would be reaching their design life, and with appropriate inspection, there could be several future failures. Consequently, the urgent inclusion of specific guidelines for the inspection of ageing offshore structures, including the inspection intervals, for the various ageing mechanisms to avoid widespread degradation and the sudden collapse of ageing structures.

Although NORSOK N-006 (2015) has included adjustments in the inspection intervals for ageing structures due to the likelihood of fatigue cracks, the inspection interval should be determined based on higher PoD of potential cracks before the cracks become a threat to the structural integrity. It also recommended that safety-critical elements (SCEs) that have reached the design life should have a closer inspection by an appropriate NDT and be inspected at most every 5 years.

Also, HSE (2009) provided recommendations on the management of ageing offshore structures. The recommendations identify factors connected to inspection planning, especially safety-critical elements, including the need to determine the dependency of the performance of the SCEs on the structural degradation. Also, lists of some aspects of ageing and degradation relevant to inspection planning for offshore structures are shown in Table 17.

Table 17. HSE (2009) Guidance on the management of an ageing offshore structure

Ageing Indicator	Examples specifically to offshore structures
External corrosion indicator	Rust streaks, Paint blistering, corroded screwed joist or bolts.
External incomplete reinstatement indications	ill-fitting enclosures, Loose covers, missing equipment, loose bolts, incomplete systems
Lack of incompatibility	Replacement equipment of a subsequent design or an alternative supplier. Problems of interface problems between old and modern control systems
Structural performance deterioration	Fatigue cracks initiation and propagation in structural members
Un-inspectable SCEs degradation	e.g. foundations, single-sided joint and ring stiffened
Increasing maintenance actions	Increased instances of repairs, in most cases left unresolved, is an ageing indicator. When this subsists, the growing pile of maintenance can become increasingly challenging to get back on track

Inspection result	Inspection results can reveal the actual status of the equipment and any defect. Trends can be developed based on repeat inspection data.
Experience of ageing of similar members	Unless active measures have been used to prevent the ageing of similar components, it will be likely that the same problems can occur at other locations
Previous repairs	May indicate that ageing problems are already occurring, and since repairs have been needed during the life of the structure, the necessity of the repair will indicate the potential for further problems

Some structural elements are also difficult or even impossible to inspect due to their location. They may also be vulnerable to ageing processes, making it challenging to account for when considering ageing and life extension assessment. This further raises the question of design for maintenance. Modern designs should consider design for maintenance in that component with design fatigue factor ranging from 2 for non-critical members to 10 for critical members are provided for in order to reduce the probability of failure during their service life (Ersdal et al., 2020). This factor needs to be considered in the fatigue analysis of ageing structures. NORSOK N-006 (2015) has recommended a process for the inspection of adjacent joints and members to obtain data of difficult to access members.

6.6 Mitigation

6.6.1 Inspection and Mitigation Analysis

The inspection policy for an OWT or, by extension, an offshore wind farm can be guided by several parameters. Parameters such as inspection intervals (inspection frequency), the technology deployed, number of critical details to inspect or, in the case of a wind farm, number of offshore wind turbines to inspect, inspection duration (in days) and the number of hotspots that require inspection.

Other factors such as the inspection technique deployed would also determine the number of hotspots inspected and effectively spent on the inspection. It is almost impossible and not cost-efficient to inspect every joint or, in the case of OWT farm, every turbine and its support structure, given the nature of the offshore environment. The quality of inspection could also influence the time spent per inspection. Yeter et al. (2020) has suggested the characteristics of various categories.

Combining all inspection policy parameters would mean a huge number of simulations and sometimes a significant amount of computational effort of a complex system. Therefore, it is necessary to simplify the number of parameters to be considered based on anticipated criticality, mainly with the help of expert elicitation. This study shall adopt the inspection interval recommended by DNV-OS-J101 (DNV, 2015), as shown in Equation 6.1. but would apply a less conservative approach by assuming the DFF to be 1 (DNV, 2015); therefore, for an OWT with projected 20-year service life, it implies there will be a total of three (3) planned inspections for the first 20 years period. If extended for another 20 years, the inspection interval is expected to be more frequent, based on the findings in the last inspection can be reviewed and updated. The inspection intervals may vary. For this study, the inspection years shall be years 7, 14 and 20.

The quality of inspection may vary for each run. Therefore, for optimal inspection quality, the combination of parameters for the inspection policy should be constrained on the assumption that 50% of the OWT support structure will be inspected during the service life of the structure (Yeter et al., 2020). Figure 44 shows the impact of performing inspection and mitigation during the service life of the structure.

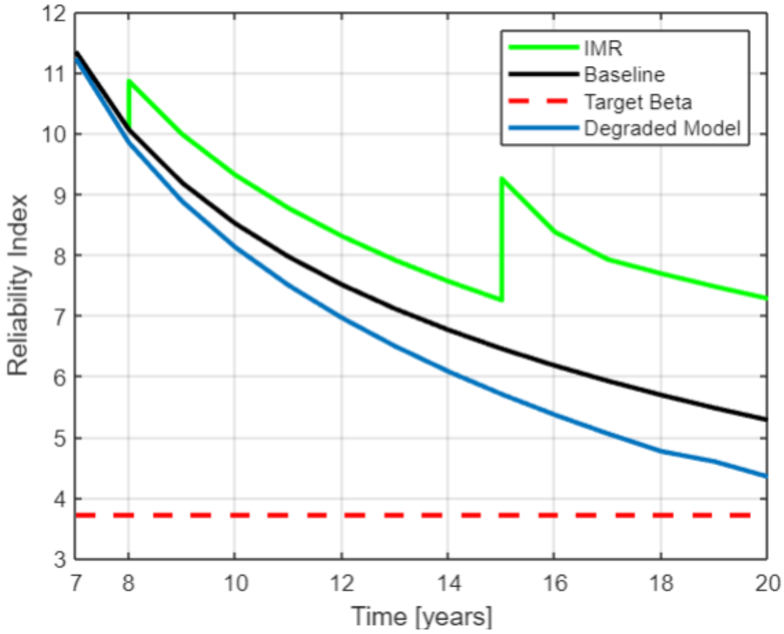

Figure 44: Effect of Inspection on the reliability assessment of an OWT

The results obtained in Figure 44 suggests that conducting inspection and mitigation improves the reliability of the structure. This is a potential cost-saving mechanism, as minor flaws and defects are arrested at their incipient phases before they pile up into severe defects. Some mitigation actions may include re-painting surfaces, removing and cleaning fouling and debris, peening, filling etc. However, most of these flaws would require sophisticated technology to detect them for immediate intervention. Close monitoring, analysis and mitigative interventions can contribute to the elongation of the remaining life of the structure.

7 Discussion

Historically, research and technological innovations have continued to play an important role in developing industry standards and guidelines for design, operation, inspection, and integrity assessment. The offshore wind industry is no exception. Although benefiting from lessons learned from a well-established industry like oil and gas, offshore wind energy still requires evidence-based research that is targeted at addressing the specific needs of the industry. Presently, offshore wind energy is thriving and has proven to be an effective alternative form of clean energy to further reduce the dependence on fossil fuels. However, the present high volumes of manufacture and marginal profitability indicate the need for more research to further drive down the cost of wind power to enable it to compete favourably with other sources of energy. Therefore, the development of economical and reliable designs is crucial for the sustainability of the sector.

This study has focused on an improved method of structural reliability assessment, which is an extension of the existing conventional methods. The proposed methodology can be applied to both new designs and ageing structures for life extension. The methodology takes advantage of the RBDO principles that have shown evidence to provide reliable and cost-efficient designs. Contrary to the conventional methods, the proposed method makes use of the response surface method (RSM) sampling method to obtain data of the structural response during loading, which is an improvement of the random sampling method used by the conventional methods. The method also required reduced computational time when compared to the conventional methods.

Sections 3.5 and 4.11.1 presents a detailed analysis of the comparison of the proposed RSM-FORM method and state of the art methods. While section 3.5 details the pros and cons of both methods, section 4.11.1 presents a detailed analysis using a hypothetical structure to validate the proposed methodology. Amongst others, the study results show that the RSM-FORM method required less computational time than the conventional Direct simulation method. This difference in computational can be of enormous benefit to both academic and industry practice.

A further study was also performed to test the methodology on other OWT support jacket structures, such as the 3-legged jacket structure under similar conditions. This additional case study shows that both structures maintained an average reliability index above 3.71 in the first 20 years. However, given that the 3-legged jacket structure requires a lesser CAPEX than the 4-legged jacket structure, it might suggest that the 3-legged jacket is more economical.

This conclusion is hasty because the 4-legged jacket structure maintained a higher reliability index, and it means at the end of the first 20 years of the structure, the 4-legged structure would require no or minimal repair compared to the 3-legged under similar environmental and operating conditions. Therefore, in making any economic conclusion, a balance between reliability and cost must be made considering the whole life cycle of the structure.

The outcome of this study shows the applicability of the proposed methodology. A parametric finite element analysis (FEA) model of a typical OWT jacket support structure was developed, incorporating operational and environmental load and soil-structure interactions to properly map its response under varying input conditions. The results from several FEA simulations have been analysed to formulate the performance function. The developed framework was applied to an NREL 5MW OC4 reference jacket support structure. The reliability assessment considered five limit states: deflection, buckling, vibration, ultimate and fatigue limit states calculated using the first-order reliability method (FORM). The results of this reliability assessment show that, for the given stochastic conditions, the structural components of the OWT jacket support structure are found to be within acceptable reliability levels as defined in DNV-OS-J101 design standards.

Appropriate quantification of model uncertainty is important in the study of reliability assessment, as it could potentially lead to an optimized design or reduction in the safety factors. The reliability assessment of the structure was based on partial safety factor or limit state function. This principle is highly dependent on the parameters describing uncertainties in the load modelling.

Therefore, the ratio between the partial safety factors for the loads and materials in this study are those recommended in the DNV-OS-J101 (2015) design standard. Also, models used in this study account only for the physical variations in the materials of construction. The materials are otherwise assumed to be defects free without discontinuities at the microscopic level. The uncertainties associated with the construction material has been accounted for by a material factor. This factor is meant to only account for additional safety required to make up for any variability in the properties of the material, including any defect in the material or unstable geometry. DNV-OS-J102 (2016) recommended the use of an additional partial safety factor if there is any material-related degradation due to ageing, casting defects, welding defects etc.

The main aim of this study was to develop a reliability framework for ageing assessment and life extension that can be used to analyse the reliability of new design and ageing structures due for life extension. To achieve this aim, certain objectives were outlined, such as a state-of-the-art review of OWT evolution of design and methods, development, and validation of typical OWT jacket support structure, development, and validation of the proposed methodology. Finally, the application of the proposed proposed proposed methodology on a reference structure. The state-of-the-art review considered the different offshore structures and their applications. A monopile support structure is commonly applied, although, for water depths greater than 50m, it becomes uneconomical, and so, jacket support structures are preferable for such water depths. The second chapter of this study also contains reviews of recent research and different design philosophies, their advantages and drawbacks.

The development of a typical offshore wind turbine model supported by a jacket structure was limited by the unavailability of field data that would have been most desirable. To mitigate this, the reference structure made available in the NREL OC4 report, which has been the basis for numerous research works, was used to develop the model. The resultant model was analysed, and the deflection and vibration results obtained from this study were compared to the results given in the NREL report by Jonkmann et al. (2013). This showed a reasonable alignment of the results with a minor variance of about 6%, which falls within the acceptable limits of validation for academic research works. This implied that the structure was valid and good enough to be used as a typical representation of a jacket structure. Details of the development of the proposed framework were reported in chapter 3 of this thesis. The modelling process and the validation are recorded in sections 5.2.3 and 5.2.3.7, respectively. While the application of the framework was reported in chapter 5. Much of the research reported in this thesis has been summarized in a published paper by Ivanhoe et al. (2020) in a reputable journal.

This study started by asking some pertinent questions, such as how long should a structure last? What is the difference between design life and remaining life? What is the best measure of the remaining life of an OWT jacket support structure? And what is the impact of uncertainty in the environmental and operational loads in determining the reliability of a structure? There have been several historic structures that are still in use and some recently installed ones that have failed already. It is, therefore, safe to say that how long a structure last is dependent on multiple factors such as the design, material strength, fabrication methods, maintenance procedures, environmental loads, incidence/accident encountered during service, and whether the structure has been designed for maintenance (i.e. to allow the possibility of repairing or replacing damaged components of the structure). Of these, the most critical factors are environmental loads and material strength due to their associated levels of uncertainties involved. The impact of all other factors on the reliability of a structure can be minimized systematically by following design guidelines. However, the magnitude of a natural disaster (such as a hurricane, Tsunami, etc.) that are all products of environmental loads cannot be estimated. Neither is it possible to predict future effects of undetected flaws or flaw sizes in critical elements of the structure.

On the question of how the remaining life of an ageing offshore structure is computed, it is worthy of note that the remaining life is dependent on the outcome of the fatigue reliability assessment. At this point, it is also important to highlight the difference between the concept of ageing due to time in service and ageing due to structural integrity. While ageing due to time in service uses the elapsed time of operation, it does not consider the reliability of the structure due to the impact of loads. Therefore, we can have a structure in operation for five (5) years, but due to its loading experience, it might have aged by over ten (10) years in this period. This also works the other way if proper inspection procedures are performed and any flaws found are repaired. More details of this can be found in chapter six of this thesis. Guidelines for the reliability criteria is provided in relevant design standards.

The term ‘design life’ has been used differently in different standards. Nevertheless, in the context of structural reliability, the design life is defined based on the HSE Design and Construction Regulations (HSE 1996), which defines design life as the working period in which the structure needs to maintain its integrity during its life cycle. This implies that the design life is the period where the structure is expected to maintain its integrity with little or no intervention, usually at the first 20 years after installation and commissioning. This should not be confused with the minimum return period (MRP). MRP considers the whole life cycle of the structure, including inspection, maintenance and repair, until the point when the structure is no longer economically viable or safe to use. MRP is usually followed by de-commissioning. So, while the design life is generally in the first 20 years of the structure’s existence, the MRP is the design life plus the years of extensions, usually about 50 – 100 years.

Depending on the operating location of the OWT, the requirements of life extension may differ. For OWTs operating on the UK Continental Shelf, an operator planning to extend the life of a structure is required to submit an assessment report of safety-critical elements (SCEs) as part of the safety case legislation for a verification plan, which is to be conducted by an independent competent person (ICP) to the duty holder. The ICP is expected to review safety-critical elements (SCEs), including the support structure, and highlight any reservations. The verification plan must be reviewed regularly and be up to date. The reviews should consider mainly functionality, availability and reliability, dependency, and survivability. For life extension purposes, availability, reliability and survivability are most crucial, as these are directly impacted by ageing mechanisms such as fatigue and corrosion. Further details of this can be found in section 2.5

Chapter 6 of this study extensively applies the proposed reliability method to an ageing offshore structure for life extension. Although most installed OWT support structures are still within their design life, most of them are approaching the end of their design life and likely remain in service. Therefore, owners and end-users need to have a robust inspection and mitigation plan. Assessing the reliability of an ageing structure for life extension would require historical data of every recordable event in the life of the structure. These include all data from the design phase through to fabrication, installation, operation of events, incidents (extreme loads), modifications, repairs, and any maintenance carried out during the life of the structure.

Important data of interest are fatigue cracking, dents, and corrosion (local or uniform). The mode of corrosion or fatigue cracks identified from the inspection report would determine the model to be used to define the deterioration mechanism. While the finite element model developed should, as far as is possible, depict the structure in its present state. Modern structures are designed with structural monitoring devices such as probes and sensors with NDT capabilities at critical and hard to reach sections, such as hotspot areas, to monitor and report any structural defects.

Data obtained from these sensors or other inspection methods like divers or unmanned inspection vehicles are often associated with uncertainty as most measuring devices are calibrated to detect and measure defects within a certain range. Therefore, if the flaws in the structure do not fall within the range, the flaws may go unnoticed. There are also concerns about the ability of some of the devices to report the actual size, classification of various defects of the defect, especially when it occurs subsurface. Also, given that defects in different materials will have a different impact, there is also a need to understand the effects of the different defects on different materials on the overall structural reliability of the structure. In this study, uncertainty regarding measurement was assumed to have been accounted for in the material factor. However, an optimization in the capability of unmanned inspection vehicles to capture additional structural data such as tensile and compressive stress of critical structural elements would be beneficial to the industry.

Due to the unavailability of data, the study assumed no fatigue cracks or dents, only uniform corrosion, and an average rate of 0.3 mm/year (Melchers, 2009). The S-N method was used to analyse the fatigue life since there were no cracks reported. The fatigue life was analysed by applying the fatigue limit state function and FORM with an initial scenario of no inspection carried out until year 20. The outcome of the results shows a reliability index of 5.2 at year 20, as shown in Figure 37. A second scenario was also reported where the structure was inspected based on the defined inspection interval. It was also assumed that minor repair actions such as grinding, hammer peening, painting, etc., were performed, which is also expected to improve the reliability of the structure. The outcome of the reliability assessment after year 20 shows a reliability index of 7.3, implying the regular inspection, maintenance and or repair (IMR) can improve the reliability of the structure and extend the life of the structure. A comparison of these two case studies shows that inspection and repairs have extended the life of the structure by additional 7.5 years within the design life period.

However, it is important to consider the economic implications in carrying out inspections, given that they are expensive and the cost increases with time. So, as the structure ages, with a potential decrease in the revenue generated from the structure (depending on the maintenance), there is also a possible scenario of an increased number of interventions. This implies a corresponding increase cost of IMR. Therefore, there should be an analysis of the revenue and cost of IMR with time and the associated uncertainty. Studying the revenue trend and cost of IMR of an offshore structure leads to a period that should be the end of life, based on economic considerations. When the continued operation of the asset is no more profitable, it should be either decommissioned or converted for use in a different application or used for tie-ins.

Therefore, regarding how long the life of an offshore structure is extended, it is simply until it is no more economically viable or meets the minimum safety requirement of the operating location. Therefore, further studies on the whole life cycle cost would be required to determine the point when the margin between the revenue and the cost of IMR becomes unprofitable. Moreover, due to the durability of some aged structures, there are reasonable speculations of using them for the deployment of new generation offshore wind turbines that are lighter in weight but with greater power generation capacity.

This study also encountered some challenges. The main challenge was the unavailability of field data, primarily due to commercial sensitivity, and so the required data could not have been made available. The risk of erroneous conclusions from applying models based on publicly available data has been mitigated by using well-established reference models for offshore wind energy-related research. In this case, the NREL wind turbine model (for loads) and the OC4 reference jacket support structure (for geometry and materials) were used, both of which have in the past been used as the basis for multiple highly cited research works. However, for the next steps of this study, a collaboration with the wind energy industry allowing access to real-life data would be required to test the model and methods before full deployment by the industry.

This emphasizes the need for collaboration with stakeholders in the wind energy industry such as owners of wind farms, service providers and wind turbine parts and equipment manufacturers. Most of the tests required for the validation of this framework may take over 20 years, which is far more than the time required for PhD research. However, a long-term collaboration would allow the process of testing the framework to be broken down into phases, which would give time for test and re-test if needed. This will also enable both the wind energy industry and academics the opportunity to build experience and contribute to the development of wind power to the point wherein it can compete fairly in the energy market, which is the overall goal.

All the results obtained from this study align with the minimum target defined by relevant standards. However, concerning the inspection, a risk-based inspection technique has been seen to provide an optimized inspection. In applying the method recommended by DNV-OS-J101 (2015), in equation 6.1, the inspection interval seems to be constant. This is a clear departure from the natural understanding that, as the structure gets older, it becomes susceptible to corrosion and fatigue and should have shorter inspection intervals.

In addition to the fact that most standards for inspection of offshore structures do not have specific guidance for inspecting ageing offshore structures. This could cause a huge concern in the future, as more structures reach their design life and require life extension assessments. Deploying the available techniques without a thorough knowledge of the degradation might lead to failures after inspection. Therefore, further research is needed to improve the method.

This study has presented a generic framework for the reliability assessment of an offshore wind turbine jacket support structure. The framework has demonstrated the applicability of the non-intrusive stochastic expansion and FORM for the calculation of structural reliability of offshore structures in a cost-effective manner and without the need for a fully integrated SRA code.

8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusion

In this study, a generic framework for assessing the reliability of ageing OWT (offshore wind turbine) jacket support structures have been developed. The framework starts with data collection and screening of both the structure and the environmental condition, including the loads and loading patterns. Data such as cracks, defects, initial design documents, modifications etc., of the structure are critical, as well as historical and present environmental loads such as wind and wave. These data sets would govern the selection of an appropriate fatigue analysis method and the development of an FEA model. A parametric FEA model for the simulation of the OWT jacket support structure has been developed, taking into account all stochastic load and capacity variables and SSI (soil-structure interaction). The model developed was first based on the original design specification and then degraded due to uniform corrosion to reflect the actual state of the OWT at the time of inspection.

After several FEA simulations were performed, results were post-processed through response surface models, deriving performance functions expressed in global stochastic parameters. FORM was then employed to calculate the reliability index, evaluating the reliability of the structure based on a set of pre-defined limit states, which include the buckling, fatigue, ultimate stress, vibration, and deflection design criteria. The proposed framework has been applied to the NREL 5MW OWT OC4 jacket support structure to assess its reliability.

In conclusion, it is important to state that the purpose of this thesis is to document a generic framework for assessing the reliability of an ageing OWT jacket structure that can be applied to different structural configurations and environmental conditions, as demonstrated in section 4.11.1. Rather than analyse an actual structure as this would require data that (due to commercial sensitivity) would not be available to an academic organisation. The analysis, however, is based on established references data and models that are widely accepted by the academic researcher community.

The following conclusions can be drawn from the present study:

- Good agreement has been achieved when comparing the results from the present FEA model against those reported in other published literature, which confirms the validity of the present FEA model.
- The model considered in this present work, when subjected to a multi-variate reliability assessment and for a valid set of stochastic variables, shows that the results obtained fall within the recommended design limits for OWT support structure design for all limit states examined.
- The results from the sensitivity analysis performed to analyse the effect of the soil properties in the model clearly shows that adding the soil element to the model had a considerable impact on the reliability assessment. The vibration consideration presents the highest impact in this instance. This highlights the importance of a proper feasibility study of the site before deploying an offshore structure.
- The sensitivity analysis performed on the statistical parameters of the stochastic variables considered, specifically for the fatigue reliability assessment, indicates that the most sensitive parameters were the mean value of the tilting moment (M) and wind thrust (F). This implies that the tilting moment and the wind thrust are the main drivers of the reliability of the structure. This agrees with Gentils et al. (2017) findings.
- The fatigue reliability analysis performed shows that the reliability index of the model was 5.2 at year 20 of this analysis, which is above 3.71. Thus the structure can be said to be safe and potentially useable even after its original design life. However, this would need to be supported by a thorough structural assessment of the structure.
- With increasing uncertainty of the structural integrity with time as the structure ages, this framework will be a viable tool for assessing the structural integrity and establishing a level of confidence in a structure’s serviceability.
- The application of this methodology on the reference structure shows a 37% improvement of the remaining fatigue reliability life of the structure.

- Stochastic variables considered have been assumed normal. However, the proposed method can also accommodate non-normal distributions through appropriate statistical transformations.
- With increasing expertise in life extension of ageing structures and an increase in the number of ageing structure's reliability assessment method would require standardization for life extension
- Despite increasing expertise in life extension, obsolesces should also be considered, especially for older structures.

This study has thus far demonstrated a non-intrusive stochastic expansion method of reliability assessment that can be used for the calculation of reliability in a cost-effective manner and without the need for a fully integrated SRA code. Collaboration with an operator in the wind energy industry could further be tested and applied in real scenarios, as the mode is suitable for industrial application. The results obtained aligns with what is industry trends. In practice, wind turbine foundations are designed to withstand the 20 years of operation with a minimum requirement for maintenance, so it is expected that their reliability should be above the threshold across the 20 years of operation.

This work has highlighted the applicability of the proposed non-intrusive formulation for the reliability analysis of offshore wind jacket support structures. It has considered several assumptions that may inspire future work. Fatigue analysis focuses on the response of the members, excluding the effect of joints and other geometrical details; this can be addressed through the incorporation of appropriate SCFs or the fracture mechanics approach. Further, the stochastic variables considered have been assumed normal, while the proposed method can accommodate non-normal distributions through appropriate statistical transformations. Finally, this work has focused on the reliability analysis of structural components, while specifically for complex structures, the system performance should also be considered.

8.2 Recommendations

This study has presented a generic framework for the reliability assessment of an offshore structure. However, there are several aspects that the author considers worthy of further study.

These include:

- Further investigation of the increased inspection interval after life extension.
- Further investigation of possible additional factors that may influence the deterioration of the strength of the material used for construction, e.g., geometric discontinuity.
- Study the whole life cycle of an Offshore Wind Turbine to determine the end of life for an offshore structure.
- Study the techno-economic viability of the 3-legged and 4-legged jackets structures under similar environmental and operating conditions.
- Further studies of the classification of various defects, material selection and analysis of the effects of the different defects on the structural reliability.
- Further studies to update the standards for the inspection of an ageing offshore structure to include specific guidance for ageing structures
- Further research on enhancing the ability of unmanned inspection vehicles to precisely capture additional information on the integrity of the structure, especially in the hard-to-reach sections.

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