Master of Science

FATIGUE ASSESSMENT OF EXISTING OFFSHORE WIND TURBINE SUPPORT STRUCTURES FOR LIFETIME EXTENSION

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Fatigue assessment of existing offshore wind turbine support structures for lifetime extension

FOR OBTAINING THE DEGREE OF M.Sc. IN OFFSHORE ENGINEERING AT DELFT UNIVERSITY OF TECHNOLOGY

by
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Preface

This thesis report is my final exercise to complete the master Offshore and Dredging Engineering at the faculty of Mechanical Engineering at Delft University of Technology. This is the result of one year of hard work that has been executed at the office of DNVGL in Arnhem and Barendrecht.

First of all, I would like to express my gratitude and sincere thanks to the members of my assessment committee for their support and guidance during this graduation project. Ir. Frank Sliggers for proposing me this subject and for his critical feedback and guidance. Ir. Pim van der male for his guidance and explanations about designing a support structure. My DNVGL supervisor Ir. Joris Truijens who has provided me with advice and encouragement during my project. Special thanks to Ir. Dominik Fallais for his valuable guidance and knowledge which was of great value for this thesis and for helping me whenever I got stuck.

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Abstract

Wind energy has become an important topic nowadays and has made a remarkable growth during the last decades, especially in Europe. Offshore wind turbines (OWTs), together with their support structures, are designed for an operational period of 20 years. The first generation of these offshore wind turbines has already reached or is approaching their designed lifetime of 20 years. Depending on the legislation and the governmental subsidies, a decision needs to be made about their future. One option is to keep the OWTs in operation after exceeding the design lifetime while the safety levels are not compromised. Operating after the design life, which is called lifetime extension has become more and more interesting in the current market conditions. To find out whether the safety levels, which are determined by the design standards are not compromised when the lifetime is extended, the OWT support structures should be reassessed when the end of design life is insight. Reassessing the support structure can take out the uncertainties of the parameters that are monitored and can make lifetime extension possible.

The objective of this study is to propose a framework for reassessing existing OWT support structures for lifetime extension since there is not a clear detailed methodology describing the assessment and extension which can be applied for OWT support structures. Because of the complexity of the problem and the limited time, only the governing limit state is studied which is the fatigue limit state. The proposed framework consists of two phases. The first phase is the reassessment phase in which the available documentation and measurements of the (operational) history are taken into account to determine the fatigue damage with more certainty from the installation of the OWT till the point when the reassessment takes place. The second phase is the remaining useful lifetime (RUL) prediction phase, which aims at determining the remaining operational lifetime of an OWT without exceeding the safety limits. For both phases, different methods can be used that can be classified in deterministic methods and probabilistic methods.

Finally, the suggested framework is demonstrated in a simplified case study. First, the fatigue lifetime of the simplified structure is calculated with wave conditions of the Gemini wind farm. This calculated lifetime resembles the initial design lifetime and serves as a comparative measure for the following reassessment and RUL prediction phases. Then the simplified structure is reassessed with updated data, using a deterministic method. Subsequently, the RUL is predicted by using probabilistic fatigue calculations in which different uncertainty distributions are taken into account. From this case study, it can be concluded that the proposed framework is applicable for different amounts and types of measurement data as well as assessment methods. The deterministic reassessment shows different outcomes of fatigue life of the structure even with a small change in the input parameters. The probabilistic fatigue calculations used for the RUL are computational more complicated but very promising since site-specific uncertainty distributions replace the generalized partial safety factors. The suggestion is, therefore, to use probabilistic models to achieve a longer lifetime for the OWT support structure without compromising the safety levels.

Contents

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List of Symbols

a	Water particle acceleration	$[m/s^2]$
α	Sensitivity factor	$[-]$
β	Reliability index	$[-]$
C_D	Drag coefficient	$[-]$
C_m	Inertia coefficient	$[-]$
D	Diameter	$[m]$
d	Deflection at the top of the structure	$[m]$
$\mathrm{d_a}$	Deflection above sea surface	$[m]$
$\mathrm{d_b}$	Deflection below sea surface	$[m]$
D_c	Miner's sum	$[-]$
D_d	Total damage	$[-]$
D_{in}	Inner diameter of the tower	$[m]$
D_{out}	Outer diameter of the tower	$[m]$
D_{tower}	Diameter of the tower	$[m]$
D_{max}	Maximum water depth	$[m]$
$F(z)$	Hydrodynamic load	$[N/m]$
f_{drag}	Drag force	$[N/m]$
$f_{inertia}$	Inertia force	$[N/m]$
F_{shear}	Shear force	$[N/m]$
G	Fatigue limit state function	$[-]$
I_{tower}	Moment of inertia of the tower	$[kg.m^2]$
k	Thickness exponent	$[-]$
K	Spring coefficient	$[N/m]$
$\mathrm{L_a}$	Length of the structure above the sea surface	$[m]$
$\mathrm{L_b}$	Length of the structure below the sea surface	$[m]$
$log_{10}\, a$	Intercept of log N axis	$[-]$
m	The negative inverse slope of the S-N	$[-]$
M	Mass	$[kg]$
$M_{bending}$	Bending moment	$[Nm]$
M_{max}	The maximum bending moment	$[Nm]$
n	Number of stress cycles	$[-]$
N	The allowable stress cycles	$[-]$
ρ_w	Water density	$[kg/m^3]$
p_f	Probability of failure	[-]
$\mathrm{q_0}$	Maximum value of the distributed Morison force	$[N/m]$
R	Resistance term	$[-]$

S	Load side	$[-]$
t	Thickness through which a crack will probably grow	$[m]$
t_{ref}	The reference thickness for welded connections	$[m]$
v_{tot}	Total water particle velocity	$[m/s]$
ω_n	Natural frequency	$[rad/s]$
χ_n	Uncertainty distribution n	$[-]$
Υ_m	Material safety factor	$[-]$
σ	Mean value	$[-]$
$\Delta\sigma$	The stress ranges	$[MPa]$
σ_{max}	The maximum stress	$[MPa]$
$\theta_{sea,surface}$	Inclination angle at the sea surface	[deg]
ζ	Zeta	$[-]$
ζ_s	Structural damping	$[-]$
ζ_a	Aerodynamic damping	$[-]$
ζ_{soil}	Soil damping	$[-]$
ζ_h	Hydrodynamic damping	$[-]$
µ	Mean value	$[-]$
λ	Lambda	$[-]$

List of Abbreviations

ALS	Accidental Limit State
DAF	Dynamic Amplification Factor
DFF	Design Fatigue Factor
DNV	Det Norske Veritas
EU	European Union
FL	Fatigue Life
FLS	Fatigue Limit State
FORM	First Order Reliability Method
GL	Germanischer Lloyd
IEC	International Electrotechnical Commission
LTE	Lifetime Extension
OWF	Offshore Wind Farm
OWTs	Offshore Wind Turbines
RNA	Rotor-Nacelle Assembly
RSR	Structural Reserve Strength Ratio
RUL	Remaining Useful Life
SCADA	Supervisory Control and Data Acquisition
SCF	Stress Concentration Factor
SLS	Serviceability Limit State
SORM	Second Order Reliability Method
SRA	Structural Reliability Analysis
ULS	Ultimate Limit State

Chapter 1

1 Introduction

1.1 Wind energy

Renewable energy is an important topic nowadays and it has made remarkable growth during the last decades, especially in Europe [1]. One reason for the transition from fossil fuels to renewable energy is the ongoing high CO2 emissions which have been caused by increasing demand for energy consumption [2]. Furthermore, the European Union (EU) has targeted in 2007 to generate 20% of the consumption through renewable sources by 2020, to reduce the global warming effect [3]. Currently, wind is one of the most essential and popular renewable source to produce electricity in the world.

Wind energy has been used for a long time for transforming wind energy into mechanical work for operating windmills, wind pumps and later to generate electricity in the 19th century by the first onshore wind turbine [4]. The share of wind energy of the total installed power capacity has increased from 6% in 2005 to 18% in 2017 [5]. The increase in electricity production with wind energy is quite remarkable for both the onshore and offshore industry, as shown in Figure 1-1. A transition from onshore to offshore technology allows the turbine to benefit from many advantages. Due to fewer obstacles in offshore, the wind is more stable, which leads to less turbulence for the offshore wind turbines and finally to a longer lifetime. Furthermore, the visual impact and the noise nuisance for the population are less in comparison with onshore wind turbines [6].

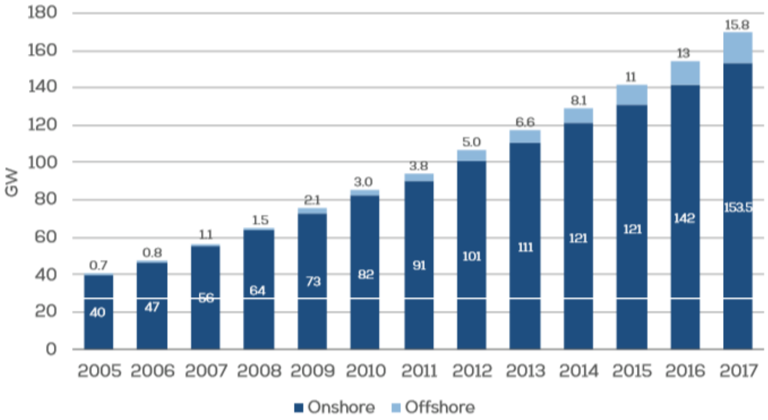


Figure 1-1: Cumulative onshore and offshore installations in the EU [5]

1.1.1. Offshore wind turbines

Wind turbines are grouped at the same location into a wind farm to convert wind power into a large amount of electrical power. Grouping wind turbines, especially in the offshore industry, makes maintenance and managing the network easier. The offshore wind turbines (OWTs) are usually located in areas where the wind conditions are favorable for producing electrical energy, as mentioned in section 1.1. The amount of generated energy depends on the size of the offshore wind farm (OWF) and the size of the OWTs.

The OWTs consists of different visible components, as presented in Figure 1-2. The combination of the rotor and the nacelle is called the *Rotor Nacelle Assembly (RNA)* and the support structure is the entire structure from the RNA into the soil, which includes the tower, the transition piece, the substructure, and the foundation. There are many combinations of substructures and foundations available in the offshore wind industry, as shown in Figure 1-2, each with their characteristics, depending on the location, the environmental conditions and the size of the wind turbine. In this thesis, the focus will be on the monopile, since the monopile substructure represents almost 81% of the installed substructures in Europe and is still the most used substructure in offshore wind [7].

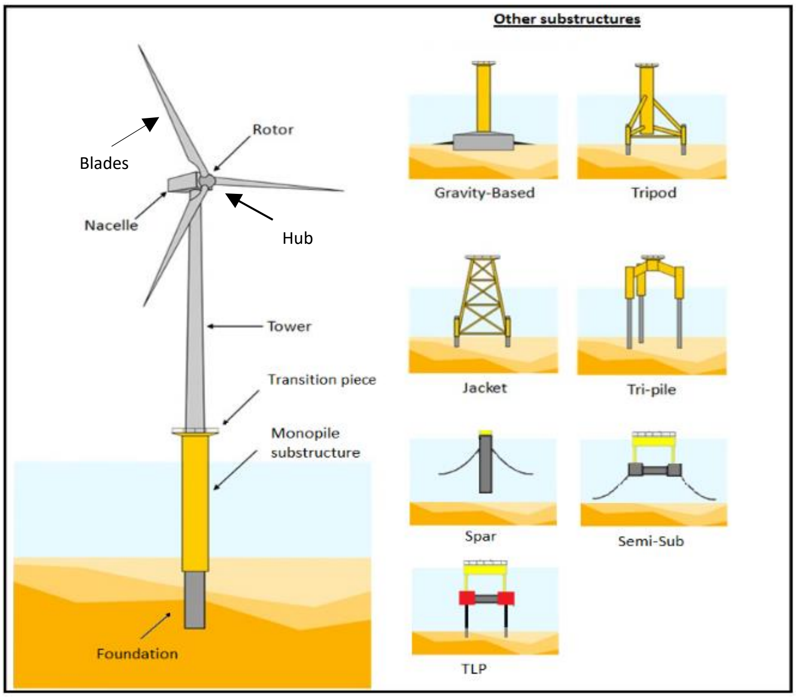


Figure 1-2: Offshore wind turbine components [63]

1.1.2. Offshore wind turbine support structure lifespan

The design of an OWT together with the support structure can be divided into several phases. The two subsequent phases are the preliminary design phase and the detailed design phase. An overview of the design stage is presented in Figure 1-3. In the pre-feasibility phase, the desktop studies will be done about the site conditions before the preliminary design phase can start. The primary objective of the preliminary design phase is the feasibility of the different support structure concepts and turbine types. In the early phase of the project, limited data is available, which makes the uncertainties high. During the planning phase, more site investigations are done to decrease the uncertainties in order to prepare the detailed design phase. During the detailed design phase, the design will be optimized, and the final support structure will be chosen. After the final design, the construction and installation will take place, and the OWT can finally start producing energy [8] [9].

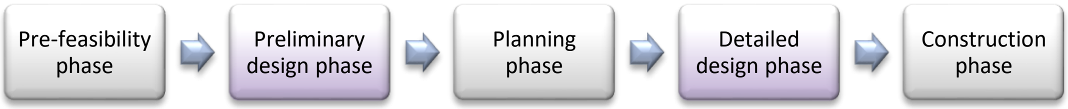


Figure 1-3: OWT support structure design stage [8]

Producing energy during the operating phase should be possible for at least 20 years according to the International Electrotechnical Commission (IEC) [10]. Until this typical 20-year design life, the OWT has an appropriate safety level. Ongoing operating after 20 years of design life could be possible depending on the different loads that the OWT will experience during the operating life. The possible loads acting on OWTs are explained in Appendix C. An overview of the lifetime of offshore wind turbines is presented in Figure 1-4.

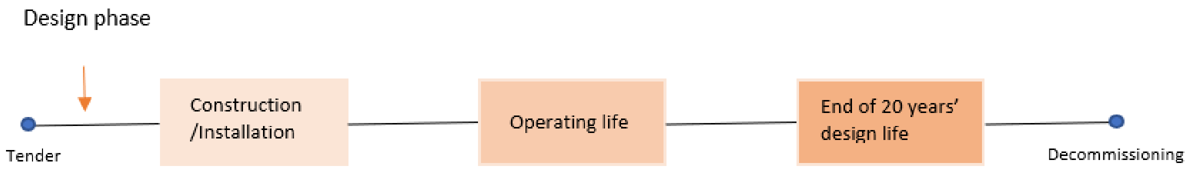


Figure 1-4: Lifetime overview

1.2 Problem description

As described in section 1.1.2, OWTs, together with their support structures are designed for an operational period of 20 years according to the design standards. Since there are a lot of uncertainties in designing an OWT, the question is if this 20-years design life is also the end of life of the OWTs. Considering the early generation offshore wind farms are reaching the end of their design lifetime in the near future [11], a decision needs to be taken about the future of the wind farms.

The decision about the future of a wind farm, that is at the end of their design life, differs per country and depends on the legislation and the governmental subsidies. For instance, only small capacities have been installed in Spain in comparison with past because of the removal of subsidies and incentives [12]. That makes it harder to install new offshore wind farms due to the higher investments and makes also decommissioning less interesting. A possible solution is to investigate if operation after exceeding the design lifetime is possible while the safety levels are not compromised. Operation after the design lifetime, which is called lifetime extension, has become more and more interesting in the current market conditions.

Lifetime extension (LTE) is a complex process due to limited data, limited research and available literature, particularly in the offshore industry. That proves that there is not a clear detailed methodology describing the lifetime extension process which can be applied for all offshore wind farms worldwide [12].

1.3 Research Description

The objective of this thesis is to introduce a framework and a methodology to reassess existing OWT support structures and to predict their remaining useful life (RUL). Reassessing offshore wind turbine support structures gives the opportunity to decide if extending the design life is possible in such a way that the safety level is not compromised. The methods described in this thesis could be a complement for the DNVGL-ST-0262 standard. This study aims to answer the following main research question:

How to reassess an offshore wind turbine support structure to predict the remaining lifetime with the aim of lifetime extension?

In order to answer the main research question, the following sub-questions should be investigated:

1. What are the uncertainties in the design of OWT support structures?
2. Why is reassessment of an offshore wind turbine essential?
3. When should reassessment of an offshore wind turbine take place?

4. It is possible to develop a framework to determine the remaining lifetime of an offshore wind turbine?
5. What are the methods in that can be used inside that framework?
    a. Which method can be used in which case?
    b. How to combine the different methods?
6. Is it possible to develop a logic diagram/decision tree for the key choices?

1.4 Structure of the report

This report is divided into six chapters, starting with an introduction about wind energy, offshore wind turbines, and the lifetime overview of OWT support structures that are designed for 20 years. To find out if operating beyond the 20 years of design life is possible and which possible approaches are feasible to reassess the OWTs, the literature will be studied in chapter 2. In chapter 3, the background for the fatigue analysis will be studied, which is the focus of this thesis. In chapter 4, a framework will be suggested with different methods to reassess existing OWT support structures and to predict the RUL. The methods suggested and discussed in chapter 4 will be applied in a simple case study in chapter 5 to illustrate the approach. The final chapter summarizes the conclusions that were made in this thesis and recommendations for future research. Reference is made to the appendices if some subject needs more clarifications for the reader. An overview of the chapters with the important keywords is presented in Figure 1-5.

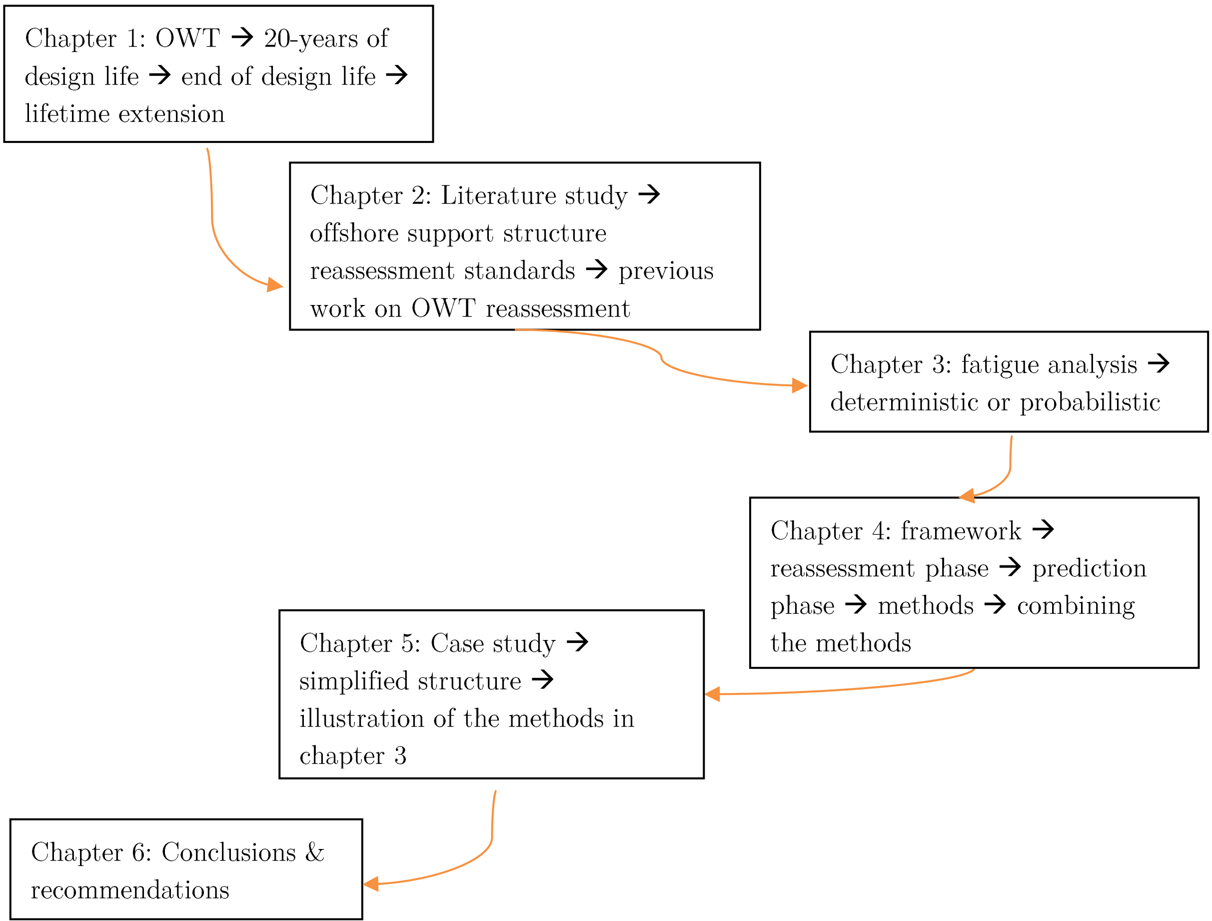


Figure 1-5: Thesis overview

1.5 Limitations

The focus of this thesis is based on FLS of the monopile support structure of offshore wind turbines. The support structure is analyzed as a whole and there is no division made in the tower, transition piece and foundation. Other parts of the wind turbine such as the blades and the RNA are not evaluated because of the limited time and the complexity of the problem. Moreover, the ULS, ALS, and SLS are out of the scope since these limit states are less critical for reassessment and remaining lifetime prediction when assuming that deterioration stays within the design limits. Therefore, the corrosion, scour, and marine growth are taken into account only through inspections to check if they are still in line with the design assumptions. It is also assumed that the loadings stay within the design limits and that there is no (accidental) damage.

Chapter 2

2 Background for Assessing Existing Offshore Support Structures

From chapter 1 it is clear that operating beyond the 20 years is possible depending on the loading the OWT experience during the operating life. It is also mentioned that a reassessment is essential to say something useful about the remaining life of the OWT. In this chapter, the literature will be studied to find out the existing procedures for reassessing an OWT support structure. Since there is not a generally standardized methodology describing the reassessment and the lifetime extension process for OWT support structures, a more general approach is studied, starting with the design standards for offshore structures in general. In section 2.2, the field will be narrowed and previous works on OWT on this topic are studied. In section 2.3, the essential subjects for this thesis, obtained from the literature study will be discussed.

2.1 Assessment according to the design standards

The literature and the guidelines in the offshore industry are mainly focused on designing new structures instead of reassessing existing structures since the market is relatively young. Nowadays, it is not sufficient anymore for an offshore engineer to know only the rules and guidelines for designing new structures, considering that the offshore structures are becoming more mature. There are several guidelines for assessing an existing structure. The experience level of these guidelines is not at the same level as for designing new structures, in particular for the offshore industry, but are still very useful for the reassessment process. In this section, only the guidelines and procedures will be discussed, starting with the more general guidelines and ending with the DNVGL guidelines which are specific for the onshore and offshore wind turbines. The following standards are consulted for assessment:

- ISO 13822 [13]
- ISO 19900 [14]
- ISO 19902 [15]
- NORSOK N-006 [16]
- UL 4143 [17]
- DNVGL standards

Assessment proposed in ISO 13822

This standard provides the requirements and procedures for assessing an existing structure in general and is not specific for the offshore industry. ISO 13822 is mainly based on the principles of structural reliability and the consequences of failures. According to this standard, an assessment can be initiated under the following circumstances:

- A change in use of the structure or when the design life is extended.
- A reliability check, as required by the stakeholders due to unexpected actions like earthquakes.
- Structural deterioration due to time-dependent actions as a result of fatigue and/or corrosion
- Structural damage due to accidental loading.

The standard suggested two approaches for the assessment, namely a preliminary assessment, and if necessary, a detailed assessment. Both assessment approaches are based on new information about the actual conditions of the structure. The preliminary assessment consists of verification of documents, inspection for possible damage of the structure, preliminary checks to identify any deficiencies related to the future safety and serviceability of the structure and actions to reduce the probability of failure. The ISO 13822 states that a detailed assessment is required if there are uncertainties in the actions or the properties of the structure.

The detailed assessment approach as described in clause 4.6 in ISO 13822 consists of a detailed documentary search and review, a detail inspections and material testing, determination of actions, determination of properties for the structure, structural analysis, and verification. The documentary search includes design documents, soil conditions, and regulation or design codes. The structural analysis is to determine the effect of the actions on the structure and the verification is to ensure a target reliability level that represents the required level of structural performance. Figure 2-1 presents the flowchart for the general assessment of existing structures as suggested by ISO 13822.

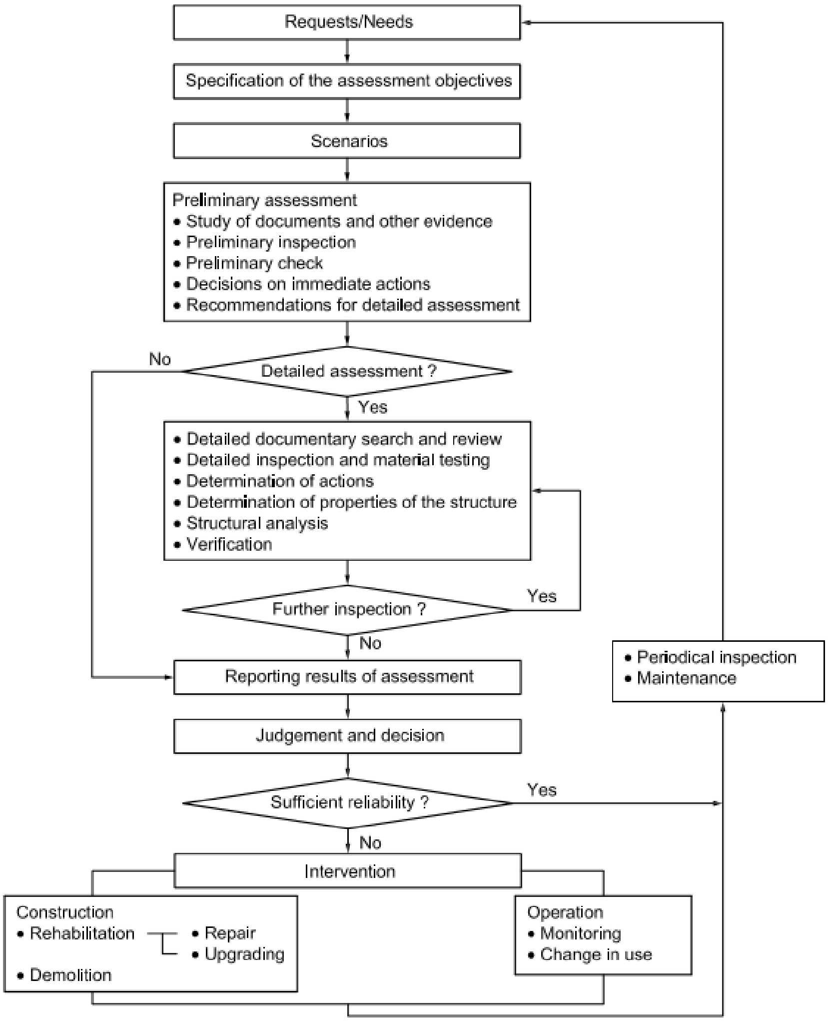


Figure 2-1: General assessment of existing structures [13]

Assessment proposed in ISO 19900

Another standard which is describing the assessment of existing structures is the ISO 19900. This standard gives the general requirements for petroleum and natural gas of offshore structures. In clause 12, the assessment is defined for existing structures. This standard is referring to ISO 19902 for a detailed assessment description but gives some essential steps regarding the assessment. It indicates that when a structure requires an assessment, it cannot be assumed that the structural conditions and actions originally used for design remains valid and suggests using updated data if available. It is not indicated which data and which specific parameters should be updated and the methodology how to do it.

Assessment proposed in ISO 19902

A detailed assessment procedure can be found in the ISO 19902 standard. Clause 24 gives the procedures for assessing existing fixed steel offshore structures to demonstrate their fitness-for-purpose. The standard states that the owner should maintain and demonstrate the fitness-for-purpose of the structure for its specific site conditions. The structure is fit-for-purpose when the annual probability of failure is in line with the design standards. If the structure does not comply with the standards, the owner should seek how to reduce the risk of failure by risk prevention and measurements. The standard states that any deviations from the design standards should be reviewed and approved by the regulator. The approach to decide if an assessment is required is illustrated in Figure 2-2.

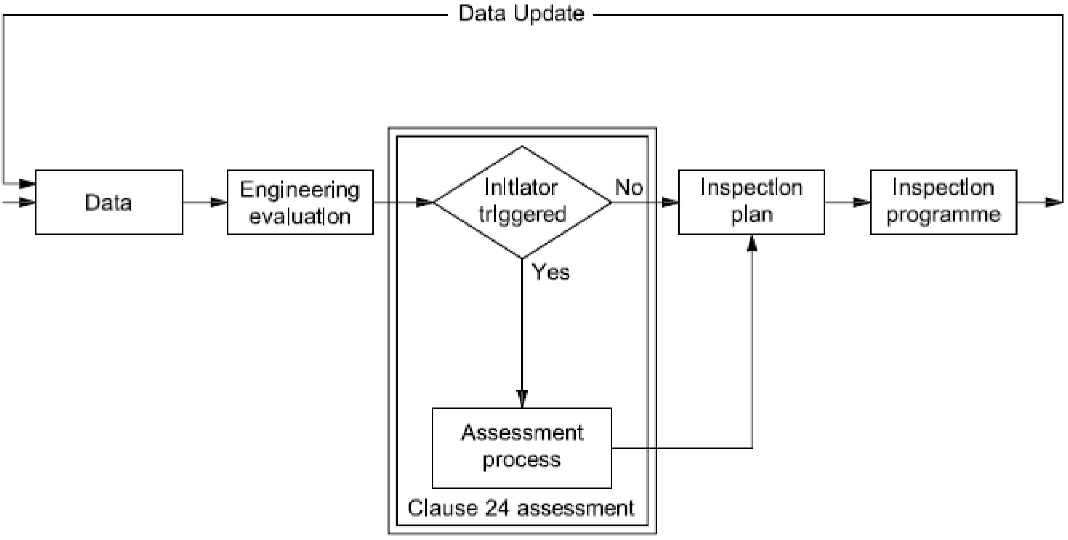


Figure 2-2: Flowchart to decide if assessment is required [15]

When an initiator is triggered, an assessment should be studied according to this standard and the main elements of the assessment consists of the following stages:

1. Assemble data, history and exposure level.
    - General information on structure/configuration
    - Design documents
    - Construction information
    - Information on structure history
    - As-is conditions
    - The exposure level depends on the risk and consequences
2. Determine if any assessment initiators are triggered
    - Changes in the structure
    - Damage or deterioration
    - Exceedance of the design life. This standard states that an extension of the design service life can be accepted without a full assessment if inspection of the structure of the structure show that time dependent degradation (i.e. fatigue and corrosion) has not become significant and there have been no changes to the design criteria (original design).
3. Determine the acceptance criteria.
    - Probability of failure
    - Structural reserve strength ratio (RSR)
    - By comparison to a similar structure which is considered as fit-for-purpose
4. Assess the condition of the structure.

    To assess the condition of the structure, sufficient information should be collected to allow an engineering assessment, consisting of the information of the platform's structural condition and facilities. Inspection of the topside, underwater, splash zone and foundation should be performed.
5. Assess the actions

    The loads acting on the structure should be recalculated with the updated environmental data.
6. Screen the structure in comparison with similar structures.
    - The exposure level of the structure to be assessed is not L1
    - If the structure to be assessed has the exposure level L2, the nearby structure that has been assessed has an exposure level L2 or L1

- The distance of the two structures is not more than 25 km
- The structures are in the same waterdepth
- The environmental and seismic conditions at the site of the structure to be assessed are not more severe than those at the location of the structure that has been assessed.
- The topside arrangements of the two structures are similar
- The structures has the same support structure
- The structure to be assessed has not suffered from any accidental damage
- The materials and welding strengths and ductility on the structure to be assessed are greater than or equal to those on the structure that has been assessed
- The component dimensions on the structure to be assessed are equal to those on the structure that has been assessed
- The soil conditions at the location of the structure to be assessed are not less competent than those at the location of the structure that has been assessed
- The age of the structures are within 5 years of each other

7. Perform a resistance assessment.

During the resistance assessment, the structure should be evaluated based on its current conditions or future intended conditions, taking into account any damage, repair, scour, modifications or other factors which can affect the structural performance or integrity. A design level analysis should be performed if the above stages are not sufficient. During the design level analysis, the resistance can be checked following the same approach as for a new design. If the reassessment is still not sufficient, probabilistic methods can be used for the assessment. Acceptance is highly dependent on the knowledge and skill of the analyst and the data upon which the analysis is based. Figure 2-3 gives an overview of the assessment procedure according to ISO 19902.

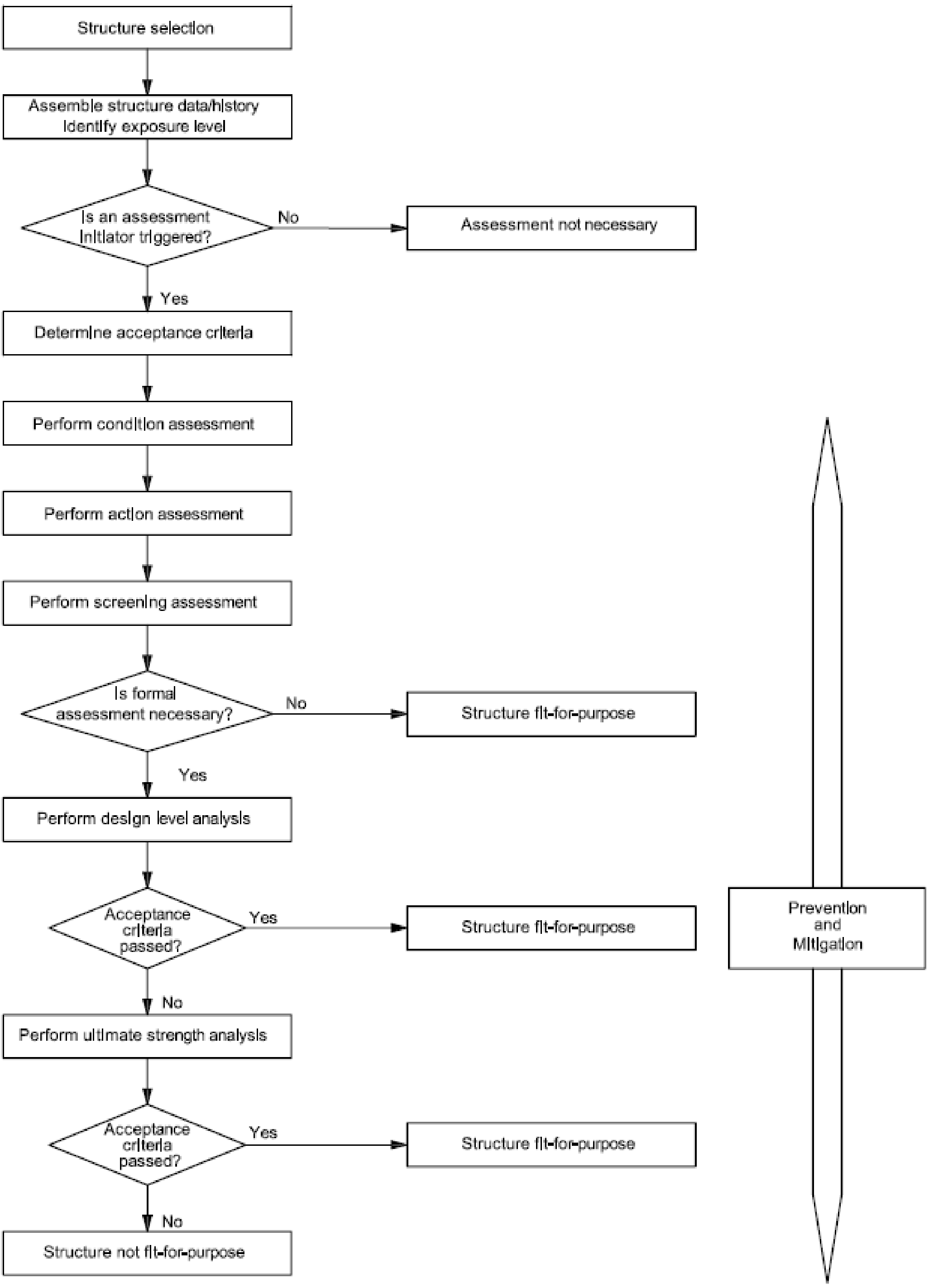


Figure 2-3: Assessment procedure proposed in ISO 19902 [15]

Assessment proposed in NORSOK N-006

Another standard is the Norwegian regulation NORSOK N-006, which gives some additional requirements for assessment of the structural integrity of offshore structures, but it refers again to ISO 19900 and ISO 19902. According to NORSOK N-006, existing structures should be assessed if any of the initiators specified in the ISO standards are triggered. The purpose of the assessment is to demonstrate that the structure is capable of carrying out its intended functions in all phases of their life cycle. This standard also indicates that data collection is an important part of the assessment. In case of lack of data or insufficient information, assumption to the safe side may be made according to this standard. The data that should be collected are listed in more detail in comparison with the ISO standards and consists of the following information:

- As built drawings of the structure
- New information on environmental data, if relevant
- Permanent actions and variable actions
- Previous and future planned functions requirements
- Design and fabrication specifications
- Original corrosion management philosophy
- Original design assumptions
- Design, fabrication, transportation and installation reports
  - Material properties
  - Information about the weld specifications during installation and repairs
  - Non-destructive testing
  - Pile driving records
- Weight report of the structure that is updated during the service life
- Inspections reports
  - Marine growth
  - Corrosion
  - Cracks
  - Dents and deflections
  - Scour
  - Damages due to frost
- Information on in-place behavior including dynamic response
- Information on soil condition and seabed subsidence

The standard states that for steel structures it is important to control degradation mechanics related to corrosion and fatigue. The assessment with respect to fatigue according to NORSOK N-006 can be summarized as shown in Figure 2-4.

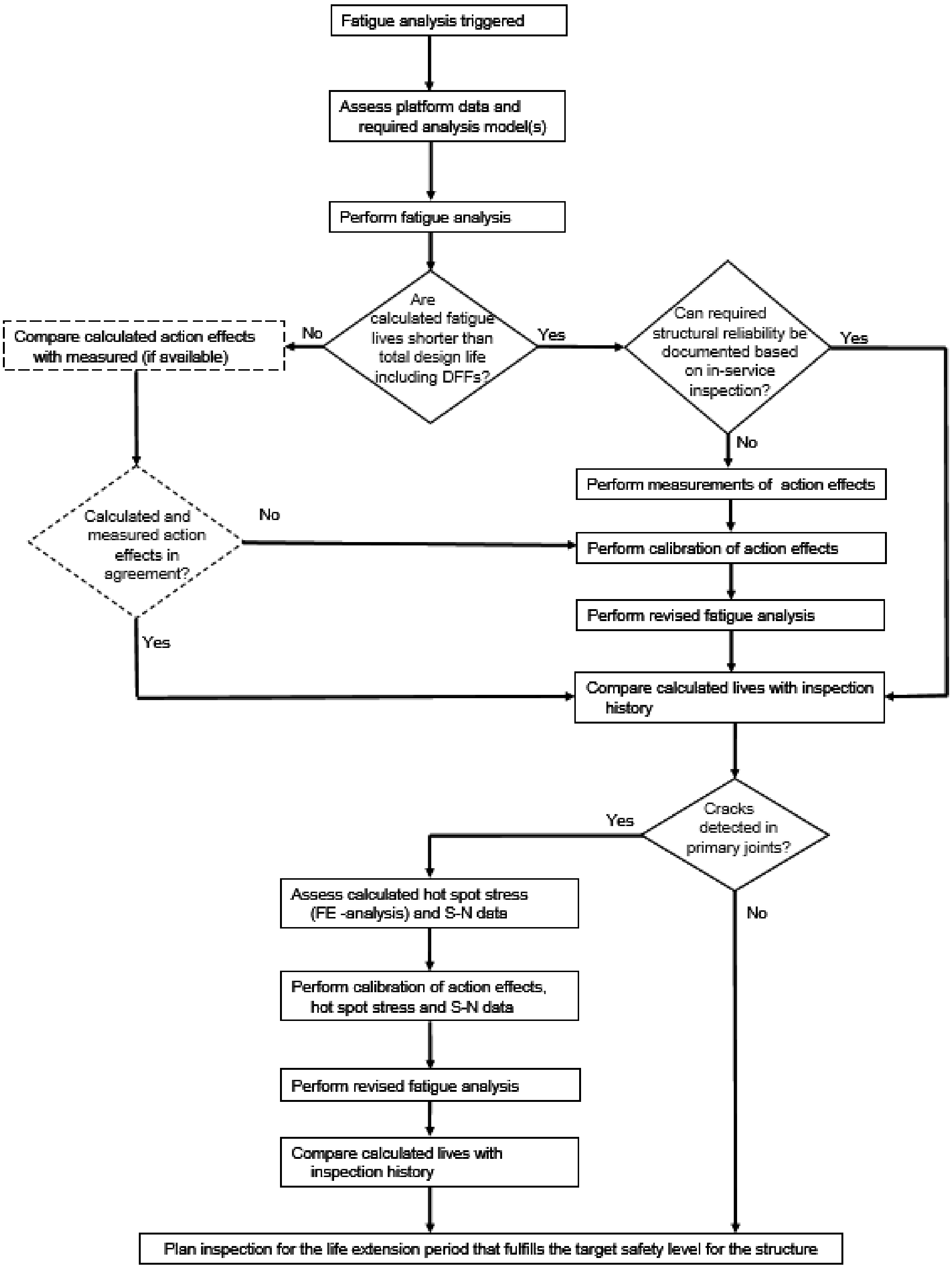


Figure 2-4: Fatigue assessment process according NORSOK N-006 [16]

Assessment proposed in UL 4143

The first edition of UL 4143 published in 2016 provides with guidance on lifetime extension of wind turbines. Although the focus of this standard is on onshore wind turbines, it gives good insight for the requirements needed into the assessment. The first step according to this standard is collecting the external conditions data from SCADA data, met mast data, and/or public databases. The external data that should be collected consists of the following information:

- Site topography
- Measurement equipment
- Wind speed and wind direction
    - At least 2 m/s bins
    - At least 30-degree sectors
    - 10 minutes wind speed average
- Wind speed distribution
- Turbulence
- Wind shear
    - The wind shear should be derived for each wind turbine location based on flow modeling on two anemometers with a minimum vertical spacing of 20 m.
- Density, temperature, pressure and humidity
    - In complex terrains, more than two sensors per measurements parameters should be used according to this standard.
- Flow inclination
- Electrical conditions
    - The grid conditions of the site should be evaluated for possible impact on the loads and the performance of the wind turbine.

The second step according to UL 4143 is to determine and to update the operational conditions from SCADA data and the OEM records. The following types of data are required for the assessment:

- Load relevant transient events
- Safety stops
- Control system stops
- Start ups
- Yaw misalignments
- Power production data
- Parked/idling data
- Maintenance and repair data
- Grid related data on site
- Earth quake, lightning, storm etc.

The third step is related to the wind turbine model. According to the standard, the simulation model should include the changes during the operational life and the external environmental conditions. The model validation should comply with the IEC 61400-13 standard.

The fourth step is the requirements for the remaining useful life. For the RUL, the next criteria should hold according to this standard:

$$FL_{design} \geq FL_{consumed} + FL_{RUL}$$

The fifth and sixth steps are the requirements for the inspection and the risk analysis, which will not be listed in this thesis.

Assessment proposed in DNVGL Standards

DNV GL has published a guideline for the continued operating of wind turbines in 2009 [18]. The objective of this guideline is to allow the assessment of wind turbines, in order to ascertain whether they are fit for continued operation. This guideline suggests two approaches, namely an analytical approach and/or a practical approach. The steps for both approaches are presented in Figure 2-5.

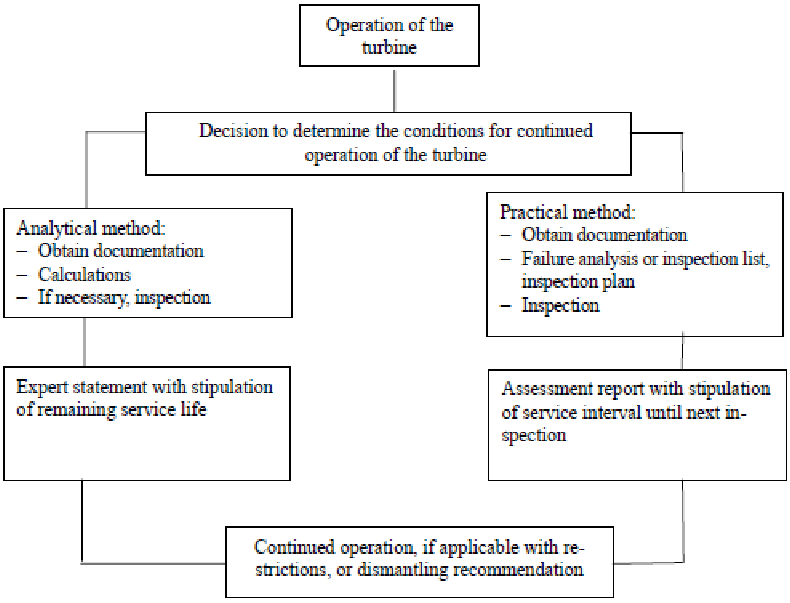


Figure 2-5: Flowchart for continued operation of wind turbines according GL [18]

Further developments based on experiences and customer needs have led to a publication of a standard and a service specification report about lifetime extension of onshore and offshore wind turbines by DNV GL in 2016. The standard allows the assessment of wind turbines in order to ascertain whether they are fit for lifetime extension and defines the assessment methods to extend the operating life of a wind turbine [19]. The service specification report serves as a publicly available description of DNV GL's services related to the certificate of wind turbine lifetime extension and it specifies the relevant tasks and requirements to be fulfilled for the certificate [20]. The standard divides the approach again in an analytical method and a practical method. The practical method is to support the analytical part by inspections. The analytical method is divided into three approaches, namely a simplified approach, a detailed approach, and a probabilistic approach.

The simplified approach can be used when the original documentation of a turbine is not available. It compares the original design conditions with the environmental conditions at the site. This is done by load simulations applying both a set of environmental conditions. The focus of the assessment is on the fatigue limit state. An assessment of the extreme loads is not required, if the environmental conditions are lower than the original design conditions, according to this standard. The fatigue calculations should be based on the current state of the art. This standard indicated that a generic load simulation model can be used for the load calculations for the simplified approach. The uncertainty of using a generic model should be considered in the assessment results according to this standard.

The detailed approach is a deterministic approach whereby the original design documentation is required for the assessment. The scope of this approach is new load calculations based on specific turbine model in which the site-specific environmental conditions are taken into account, consisting of the following data:

- Wind data
- Wave data
- Soil conditions
- Influence of the wind farm configuration
- Other environmental conditions, if applicable (e.g. temperature, humidity, ice aggregation, salt content of the air)
- Load measurements, if available

The focus of the detailed assessment is on the fatigue limit state. An assessment of the extreme loads is not required, as soon as the environmental conditions are lower than the original design conditions, according to this standard.

The final approach is a probabilistic approach where the parameters can be described by appropriate probability distributions. The probabilistic method described in this standard is the structural reliability method (SRA) which consists of a number of steps that will be explained in more detail in chapter 3. The standard states that the uncertainties in the mathematic models and the input parameters can be described as probability distribution. By defining the limit state and choosing the probabilistic calculation method, the probability of failure can be calculated. The probability of failure depends on the uncertainties in the design, which can be divided into aleatoric and epistemic uncertainties. More information about the uncertainties of an OWT can be found in Appendix A and an overview of the methods for continued operation assessment according to DNV GL - ST - 0262 is listed in Appendix B.

Conclusions based on comparison of these standards

- No principle differences between the standards.
- The UL and the DNVGL standards are wind related and the other standards more oil and gas related.
- For this study, the DNVGL standards will be followed as a guideline.

2.2 Previous work on lifetime extension of offshore wind turbine structures

Besides the design standards, as described in section 2.1, many researchers have worked on this topic. In this section, the work by other researchers will be discussed, starting with Megavind [21]. Megavind has published a report describing the strategy for determining the lifetime extension strategy based on component-by-component analysis, which means that each component of the turbine will be assessed. The report states that the remaining lifetime must be assessed, based on the available data for the design parameters and operational history. A summary of the available data scenarios and the approaches for lifetime extension according to Megavind are as follows:

1. No design basis or operational measurements are available
   In this case, it is recommended to develop software tools that use information from maintenance and inspection to predict the critical points of the wind turbine and to predict the remaining lifetime.
2. Turbine parameters used in the initial design are available
   The initial conditions can be used to determine the IEC class for the site and the remaining lifetime can be predicted by computational tools by understanding the annual reliability of the critical structural components.
3. Besides the design-basis information, SCADA data is available
   When the design basis information and the operational data is available, Megavind recommends using a probabilistic design approach to predict the remaining life and the frequency of inspection.
4. Load measurements are available along with SCADA data and design basis
   This is the ideal case where a lot of information about the wind turbine is available. The RUL for the substructure can be estimated using a deterministic and/or a semi-probabilistic approach according to Megavind. This paper states that the estimated RUL accuracy depends strongly on the available information. The deterministic approach will give a characteristic estimate of the RUL and the probabilistic approach will provide more detailed information about the RUL.

Another researcher is Gerhard Ersdal [22] who has investigated the possible life extension of existing offshore jacket structures. The focus of his thesis is based on the ultimate limit state and the fatigue limit, recommending the assessment of existing structure based on linear analysis, non-linear system strength analysis, structural reliability analysis for the ULS and the FLS. Ersdal recommended as a part of the assessment procedure, a structure specific evaluation of hazards and the failure modes. He concludes that life extension of a structure is feasible with respect to the FLS without compromising the safety of the structure. However, this requires that sufficient maintenance, inspection, and repair are performed at an acceptable level. The most promising method according to Ersdal is the structural reliability analysis (SRA). This method takes the uncertainties into account and can predict when failure can occur. In 2008, Ersdal et al. [23] published a paper giving a general approach of assessment of aging facilities for life extension. He states that the as-is condition, consisting of the operational experience, accidents, degradation, performed maintenance, and new knowledge is essential for the assessment. A schematic overview of his proposed assessment process is presented in Figure 2-6. This overview is applicable for all the limit states and not specific for FLS.

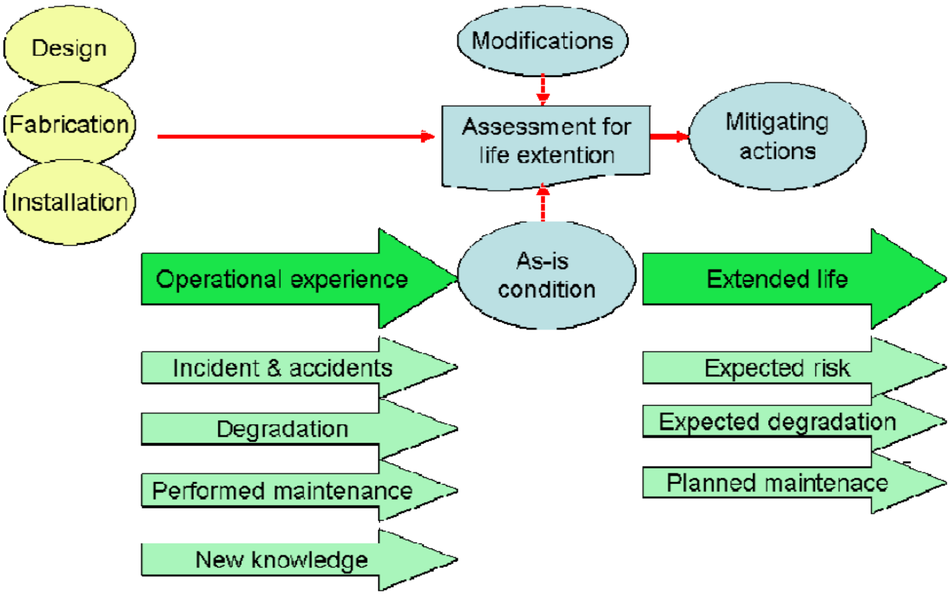


Figure 2-6: Schematic overview of assessment process according to Ersdal [23]

Veldkamp [24] [25], investigated the input parameters used in designing a wind turbine and the uncertainties in the design. He concludes that the best approach is to determine the site-specific damage and predict the remaining useful life (RUL) by using probabilistic methods. Because of the complexity of the problem, it is hard to implement inspection into the models.

According to Dalsgaard Sorensen [26], a probabilistic model should be formulated for fatigue damage accumulation. If inspections are included, then a fracture mechanics approach should be used. The model for fatigue damage is the basis for the limit state equation and will be a function of a number of stochastic variables in the fracture mechanics model and parameters modeling the uncertainty of the stress ranges.

Other researchers on this topic are Lisa Ziegler and Michael Muskulus. In 2010 they published a paper for fatigue reassessment of offshore wind monopile substructures to identify the important parameters to monitor during the operational phase [27]. They conclude that corrosion, turbine availability, and turbulence intensity are the most influential parameters. They also conclude that those parameters vary strongly for other settings and that case-specific assessment is necessary. Muskulus et al. [28], conclude that uncertainties in the design of an OWT can dramatically influence the system reliability, especially for the monopile substructure. Jacket structure is more robust to uncertainties and modeling error according to Muskulus. Ziegler and Muskulus [29] conclude in a paper of 2016 that inspection for fatigue cracks is essential to take out the gross errors but numerical fatigue reassessment and monitoring are still needed for the RUL prediction. Another paper of Ziegler and Muskulus [30] conclude that the biggest difficulty in a probabilistic approach is to determine what uncertainties have to be considered and how to parameterize them properly. There are also unsolved problems of deterministic analysis, such as how to determine the real loads at all the points of the structure form a limited number of measurements. In her last paper [12], she concludes that the market for end-of-life solutions is still in its infancy and that life extension is getting more and more important in the next years because of the number of onshore wind farms that are reaching the end of their design life. It is also mentioned that the lifetime extension assessment differs between the countries, which means that there is not a clear methodology that can be used internationally.

2.3 Proposed framework for assessing offshore wind turbine support structures

In this section, the essential information obtained from the previous sections that is relevant for assessment and RUL prediction of an OWT support structure will be discussed. As mentioned in the oil and gas design standards, the first step is to determine if reassessment is needed depending on the initiators. The reassessment is to demonstrate the structures fitness-for-purpose for continued operation. The conditions for a reassessment of an OWT are slightly different in comparison with an offshore oil and gas platform and shall be assessed if one of the next conditions exists.

1. Changes in the environmental conditions
   The environmental conditions such as wind speed, wave height and current may change during the design lifetime of an OWT. A reassessment is only needed when the environmental conditions are more onerous then where the OWT is designed for. The possible changes and the way of how to update this data are explained in more detail in chapter 4.2.
2. Degradation of the OWT
   There are many factors that lead to degradation of the OWT during their design lifetime, e.g. fatigue, corrosion, scour, marine growth or damage due to boot impact, as illustrated in Figure 2-7. If the degradation rate is higher than the as-designed for rate, a reassessment shall take place to demonstrate the fit-for-purpose of the turbine and mitigation shall take place if needed. The degradation factors shall be checked during inspections and registered in the as-is documents.
3. Changing the wind turbine components
   This is the case when repowering takes place and a new turbine is installed or when another component is installed as a replacement for the initial equipment of the turbine. The upgraded component is generally more onerous and a reassessment is needed to demonstrate if the support structure is still fit-for-purpose. This is because wind turbines are getting bigger and bigger, for example in the case of repowering. A reassessment is not necessary if the replaced component is the same as the initial component, e.g. in the case of maintenance.

4. <u>Exceedance of the design life</u>
   Another condition in which the reassessment is essential is when an exceedance of the design life takes place. The reassessment is to determine the cumulative fatigue damage of the existing structure and to judge if the structure is suitable for continued operation after the 20 years' design life. The focus of this thesis is on this type of condition. The methodology of lifetime extension, which included the reassessment methods and the methods for predicting the remaining useful life is described in chapter 4.

Figure 2-7: Degradation of an OWT monopile structure [64]

From the literature study, it can be concluded that the available information about the structure during the design phase, the construction and installation phase, and the operational phase are essential for the meaningful reassessment and the remaining lifetime prediction. Since the focus of this thesis is based on existing offshore wind turbines, the reassessment takes place after some years of operating. During the operational phase of the turbine, more information can be gained to reduce the uncertainties in the design and the assumptions. To determine the lifetime of an OWT support structure, a framework is suggested in this thesis, consisting of two phases. As presented in Figure 2-8, one phase is the reassessment phase to identify the current health of the OWT support structure and the prediction phase to determine the remaining lifetime and taking into account the future uncertainties. The methods for each phase in the framework and applying the methods will be discussed in chapter 4. Since the focus of this thesis is based on the fatigue limit state and some background information is needed to understand the methodology in chapter 4, the fatigue calculation process will be explained in chapter 3.

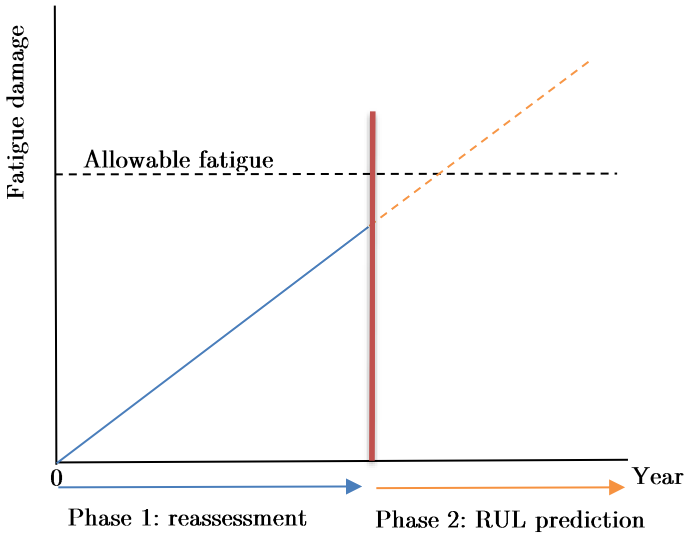


Figure 2-8: Proposed framework for reassessment and RUL prediction

Chapter 3

3 Fatigue Analysis

In this section, the fatigue calculation process will be explained for the offshore wind turbine support structure.

First, the general approach of fatigue analysis is studied, which can be applied for the deterministic fatigue approach based on the S-N curves and the Miner's rule. In addition, it will be explained how the uncertainty distribution can be applied instead of a partial safety factor to demonstrate the probability fatigue approach. Both the deterministic and the probabilistic approach will be studied for a simplified OWT as a case study in chapter 4.

3.1 The general fatigue approach for an offshore wind turbine

The general fatigue approach overview is presented in Figure 3-1 and it starts with the offshore wind turbine model that can be modeled in a simulation program like Bladed. The structural parameters together with the fatigue load cases are the input for the simulations. With the load cases that are listed in Appendix C, the different loads are simulated for the critical member or place of the structure which experiences the highest loads.

The inputs for the load simulations are the wind and wave data, which are described in chapter 4. The stress response can now be determined through simulations. The simulations can be skipped if strain measurements are available. In both cases, the stress should be multiplied by the stress concentration factor (SCF) if there is a local stress increase in stress with respect to the nominal stress due to welds, wall thickness jumps, welded attachments or any changes in geometry [DNV-RP-C203]. If the strain gauges are placed at the critical points to measure the stresses, multiplication by the SCF is not needed.

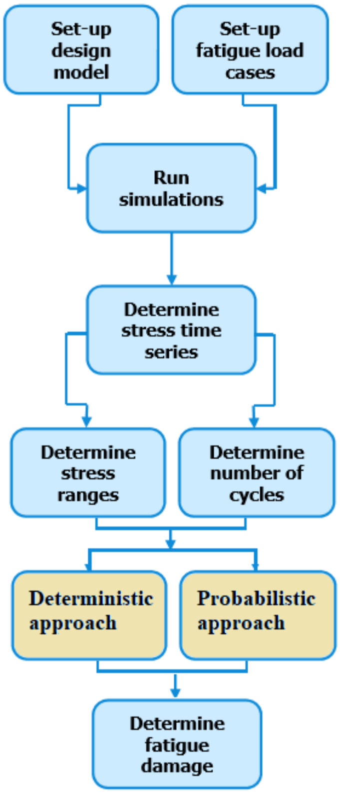


Figure 3-1: Fatigue analysis approaches

Since fatigue depends on stress ranges and number of cycles, both should be determined. The stress ranges can be determined by rainflow counting or by peak counting, see Appendix D for more explanation about these two methods. The final step is to determine the fatigue damage using a deterministic approach or a probabilistic approach. Both methods will be explained in section 3.2 and 3.3

3.2 Deterministic fatigue process

As described in section 3.1, the stress response for the critical location is multiplied by the SCF and the stress ranges are determined through the peak counting method or the rainflow counting method. The next steps to determine the fatigue damage is by following a deterministic approach, as presented in Figure 3-2.

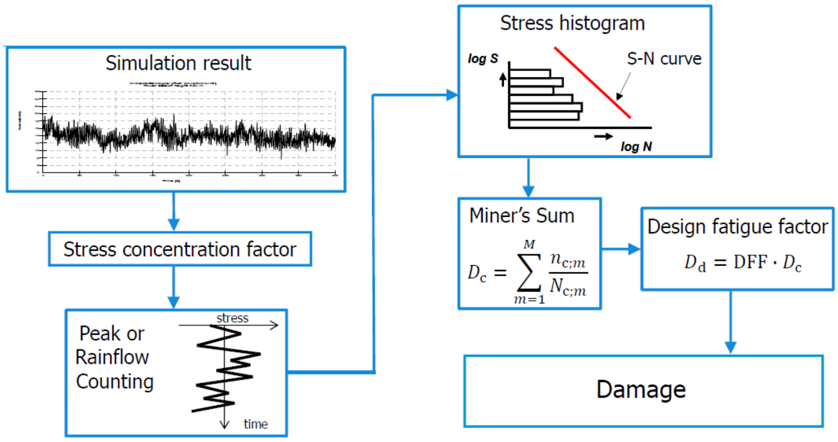


Figure 3-2: Deterministic fatigue approach flowchart

After determining the stress ranges, the allowable stress cycles should be determined from the experimental S-N line approach. Many material specimens are fixed in a testing frame and subjected to a series of constant load cycles until failure occurs. The number N of cycles that leads to failure with the corresponding stress ranges gives the S-N line plotted in a graph. The S-N line is the mean of the number of cycles N minus two times the standard deviation, as illustrated in Figure 3-3.

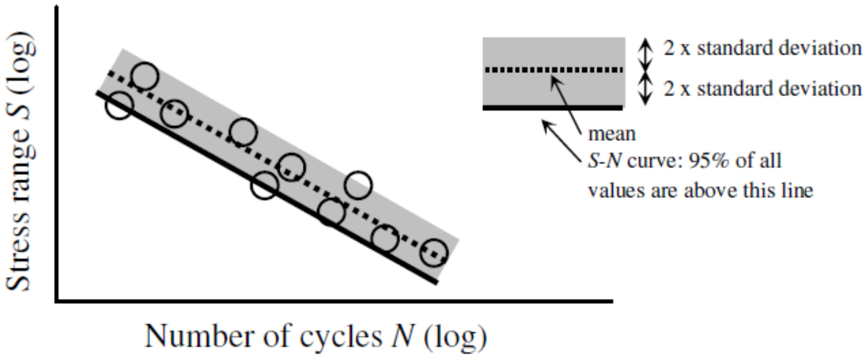


Figure 3-3: The failure point at stress range S in N number of cycles [9]

It is essential that each construction detail at which fatigue cracks may potentially develop is placed in the right class according to the DNVGL design standard [31]. In the design standard DNV-RP-C203, the different S-N curves and specifications are described. The basic design S-N curve for joint constructions is described by Equation (1).

$$\log_{10} N = \log_{10} a - m.\log_{10}\left(\Delta\sigma.\left(\frac{t}{t_{ref}}\right)^{k}\right) \tag{1}$$

where:

N	The allowable stress cycles
$\log_{10} a$	Intercept of log N axis
m	The negative inverse slope of the S-N
$\Delta\sigma$	The stress ranges
t	Thickness through which a crack will probably grow
t_{ref}	The reference thickness for welded connections
k	Thickness exponent

A typical S-N curve is presented in Figure 3-4 for different construction details. There are three important regions in the S-N curve namely, the category region, the constant amplitude region and the cut-off region:

- The detail category, defined as the region N ≤ 10^6 cycles. A logarithmic relation between stress range and cycles is valid here, with slope m = 3.
- The constant amplitude fatigue limit, defined as N > 10^6. After this point, a slope of m = 5 describes the stress range – cycle relation.
- The cut-off limit is defined at 10^8 cycles. Below this stress range, the fatigue damage is not considered.

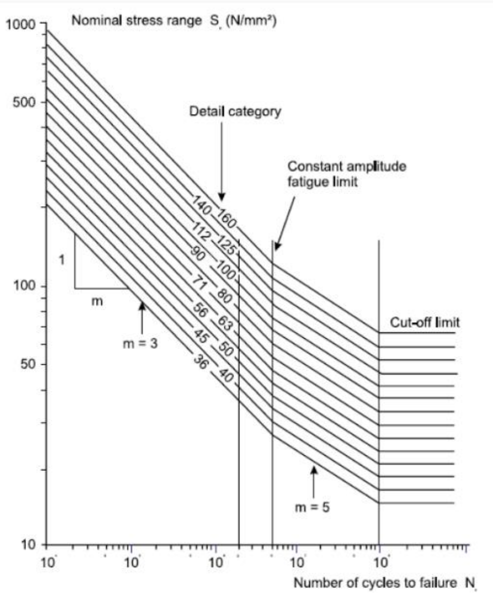


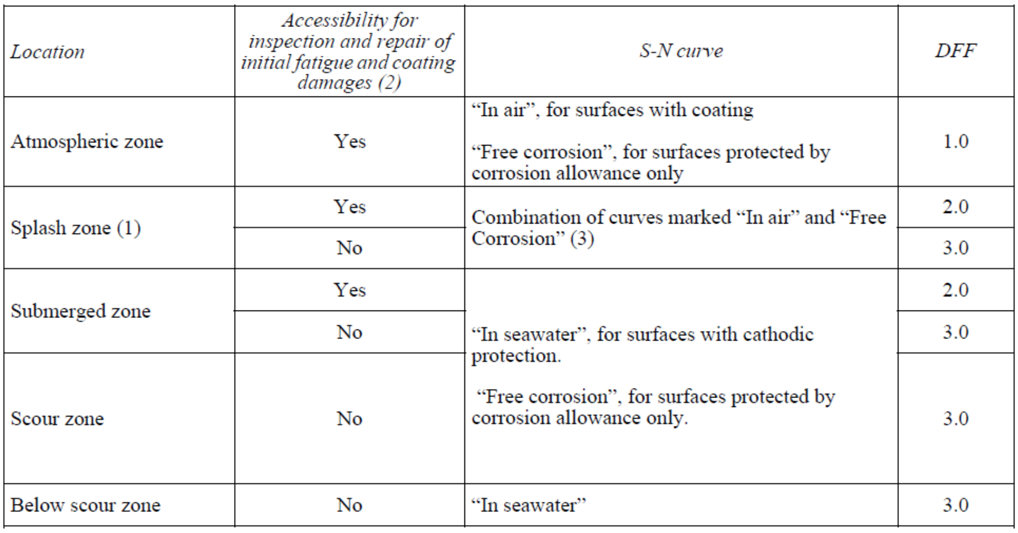
Figure 3-4: Fatigue strength curves [65]

After determining the allowable stress cycles from S-N curve, the cumulative wave damage can be calculated with the Miner's rule. This rule does not consider any influence of the sequence of wave loads might have on the structure, but it gives a good indication of the fatigue life of the structure. The Miner's rule is given in Equation (2):

$$D_c = \sum_{m=1}^{M} \frac{n_{c;m}}{N_{s;m}} \tag{2}$$

A safety factor DFF is applied to determine the total design fatigue damage. The safety factor applied is set by DNV-OS-J101 [32] and depends on the location of the structure, as shown in Table 1.

Table 1: Fatigue safety factor DFF [32]



Location	*Accessibility for inspection and repair of initial fatigue and coating damages (2)*	*S-N curve*	*DFF*
Atmospheric zone	Yes	"In air", for surfaces with coating "Free corrosion", for surfaces protected by corrosion allowance only	1.0
Splash zone (1)	Yes	Combination of curves marked "In air" and "Free Corrosion" (3)	2.0
	No		3.0
Submerged zone	Yes	"In seawater", for surfaces with cathodic protection. "Free corrosion", for surfaces protected by corrosion allowance only.	2.0
	No		3.0
Scour zone	No		3.0
Below scour zone	No	"In seawater"	3.0

The total damage can now be calculated according to Equation (3).

$$D_d = DFF.D_c \tag{3}$$

Another method is to multiply all the stress ranges by the material factor Υ_m instead of the fatigue safety factor [31]. The relation of the DFF and the material factor is presented in Table 2.

Table 2: Relation between fatigue safety factor and material partial safety factor [31]

$\boldsymbol{DFF}$	$\boldsymbol{\Upsilon_m}$
1.0	1.0
2.0	1.15
3.0	1.25

If the total design fatigue damage is below 1, then the structure is not susceptible to fatigue damage in the predefined lifetime.

3.3 Probabilistic fatigue process

The deterministic fatigue assessment as described in section 3.2, is based on the concept of limit states in combination with partial safety factors. This deterministic method is described by the design standards, which means that designing according to the design standards does not deal directly with uncertainties and the probability distributions. The uncertainties are covered by partial safety factors and this may lead to conservative results, as they are calibrated for general offshore applications [33]. In this section, the fatigue damage approach will be explained again, but now taking the uncertainties into account. The aim of this section is to show the way of working for probabilistic fatigue calculation. The background of the probabilistic design will not be explained in section. Any information about the probabilistic background can be found in Appendix E.

For the probabilistic fatigue damage of an offshore wind turbine, all the structural and environmental parameters should be taken as stochastic parameters. However, relevant simplifications have to be made to perform the analysis. Due to the simplifications, most of the steps in the probabilistic fatigue approach will be the same as the deterministic fatigue approach, as shown in Figure 3-5. The difference is that the partial safety factors will be replaced by the uncertainty distributions. The limit state function, consisting of the resistance term and the load term can be calculated with different probability methods like, FORM, SORM, Monte Carlo as described in Appendix E. Instead of getting a fatigue damage with the Miner's rule, the result of the probabilistic approach will be in probability of yearly failure or as the cumulative probability of failure. The relation between the probability of failure and the cumulative probability of failure is given in Equation (4). The steps needed for the probabilistic calculations of a structure are combined to a method, which is called the structural reliability method (SRA). This method will be explained in detail in chapter 4.

$$P_{f,annual\_n} = P_{f,cummulative\_year\_n} - P_{f,cummulative\_year\_n-1} \qquad (4)$$

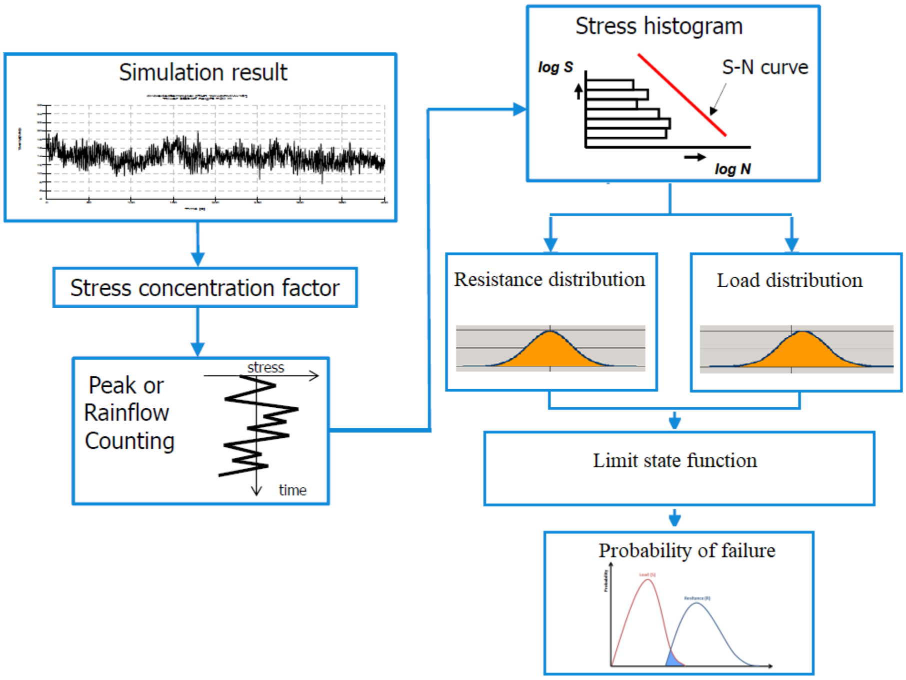


Figure 3-5: Probabilistic fatigue analysis

Chapter 4

4 Assessment of Existing Offshore Wind Turbines

As it is concluded from the literature study of chapter 2, the most promising approach to determine the remaining operating life of an OWT support structure is to divide the process into two phases. The procedure for these two phases, the reassessment, and the calculation of RUL will be discussed in this chapter. In the first section, an overview of the OWT lifetime will be presented to explain the framework for the reassessment and the calculation of RUL. The framework consists of different methods for the reassessment and the RUL. Since the methods are dependent on the available data, an overview of the possible available data and the different scenarios of available data will be discussed in section 4.2. Depending on the different scenarios of available data, the methods for reassessment and RUL will be presented in sections 4.3, 4.4 and 4.5. Most of the reassessment methods are based on a deterministic approach while the RUL prediction is based on a probabilistic approach. Combining these two approaches is essential to predict the remaining lifetime and is studied in section 4.6. All these subjects come together as a methodology to predict the remaining useful lifetime of an OWT support structure.

4.1 Explanation of the proposed framework

In the last section of chapter 2, the proposed framework has been explained shortly and presented in Figure 2-8. In this section, the framework will be studied in more detail to get a better understanding of the procedure. The starting point of the framework is the lifecycle of OWTs as presented in Figure 1-4 in chapter 1. That figure needs to be expanded to make it useful as a building block for the framework. Figure 4-1 is the same figure but now with the relevant terminology, which will make this chapter more clear.

As presented in Figure 4-1, after the design phase, construction and installation, the operating phase starts where the wind turbine finally starts producing energy. During this phase, the inspection of the structure will take place in intervals as described by the design standard. The inspection reports provide a good picture of the as-is condition of the OWT support structure. Next to the inspections, monitoring the OWT in this phase is a key element for collecting environmental and operational data of the turbine. More information about data acquisition will be given in the next section.

After some years of operation, the monitored parameters together with the inspection data can be compared with the parameters assumed during the design phase. The new data can then be used to reassess the OWT support structure and to calculate the site-specific fatigue damage. After the reassessment, the RUL can be predicted. The RUL is a prediction since the environmental conditions are random which makes it hard to say something with certainty about the future.

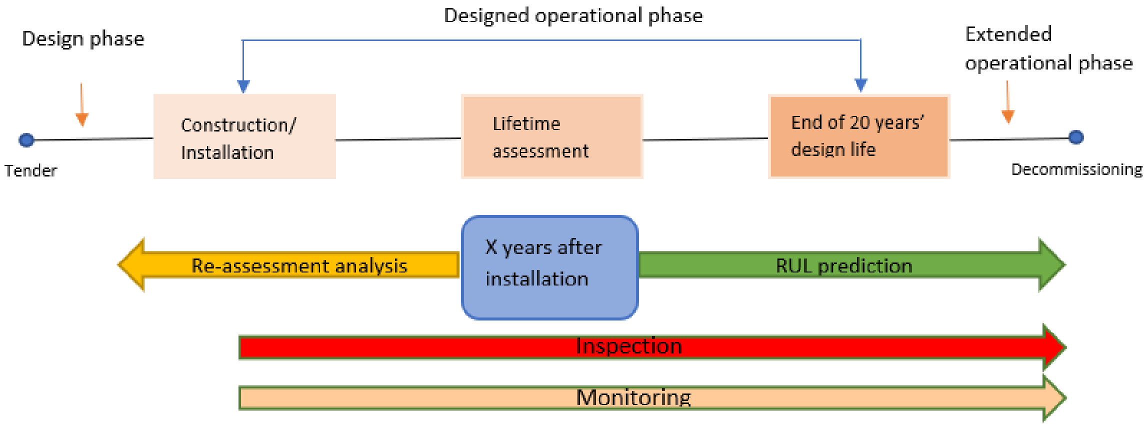


Figure 4-1: Lifecycle overview of an OWT

From Figure 4-1 it can be concluded that two actions are essential, one is data gathering and recalculating of the fatigue damage, and the other one is to determine how long the OWT can still operate. That leads to a framework consisting of two phases as shown in Figure 4-2. The first phase is the reassessment phase in which the history of the OWT is taken into account to determine the fatigue damage with more certainty from the installation of the OWT till the point when the reassessment takes place. The results of the re-assessment may be higher or lower than the initial calculated fatigue damage in the design phase. The second phase is the remaining useful lifetime (RUL) phase which defines until when the OWT can keep operating without exceeding the safety limits. For both phases, different methods can be used that can be classified in deterministic methods and probabilistic methods. The methods that can be used are depending on the available data. Before the different analysis methods can be described for each phase, the possible available data needs to be studied, which will be done in the next section. The aim of the reassessment is to see if the OWT support structure is fit-for-purpose. The criteria of fit-for-purpose is according to the design standards, which is the Miner's sum for the deterministic reassessment [34]. The structure is still fit-for-purpose when the next criteria hold:

$$D_c\ [34]= \begin{cases} 1, & when\ the\ material\ safety\ factor\ is\ applied \\ \frac{1}{\eta}, & when\ the\ DFF\ is\ applied, with\ \eta\ is\ equal\ to\ the\ DFF\ factor \end{cases}$$

For the probabilistic methods, the criteria for fit-for-purpose is defined when the annual probability of failure is below the target reliability level of 10^{-4} [35].

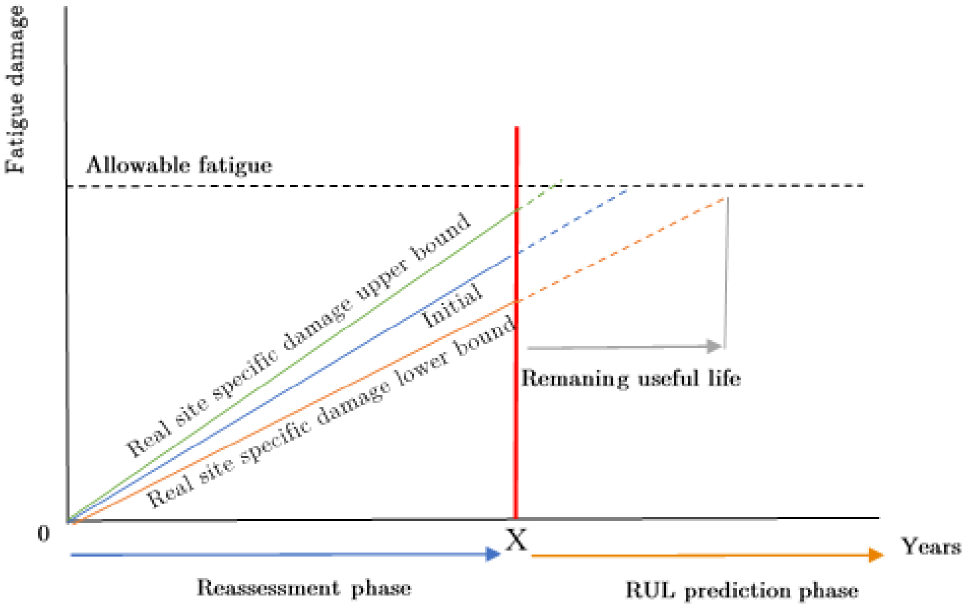


Figure 4-2: Proposed framework with two phases

4.2 Scenarios of available data for assessment

As mentioned in the previous section, information and data acquisition are not only key elements for the design of the OWT but also essential for a meaningful assessment with reliable results of the OWT. The information and data acquisition for each phase of the lifespan of the OWT is shown in Figure 4-3. The ideal scenario is that all data listed in the figure is available, which is not always the case and there can be all kind of reasons for the missing data. As mentioned before, which methods can be used for the reassessment depends on the available data and they will be classified into three levels of available data. The three levels of available data are proposed and presented in Table 3. For each level of available data different methods can be used for the reassessment and the RUL prediction as presented in Figure 4-4. Each level will be explained in more detail in the next section together with the possible methods that can be used for each level.

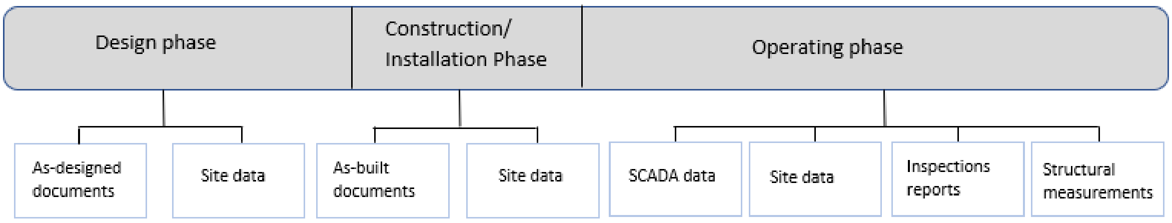


Figure 4-3: OWT data gathering during the different phases

Table 3: Levels of available data

Level of available data	Kind of data
Level 1	As-designed + as-built data
Level 2	Level 1 data + SCADA data + updated site data + inspection reports
Level 3	Level 2 data + structural measurements

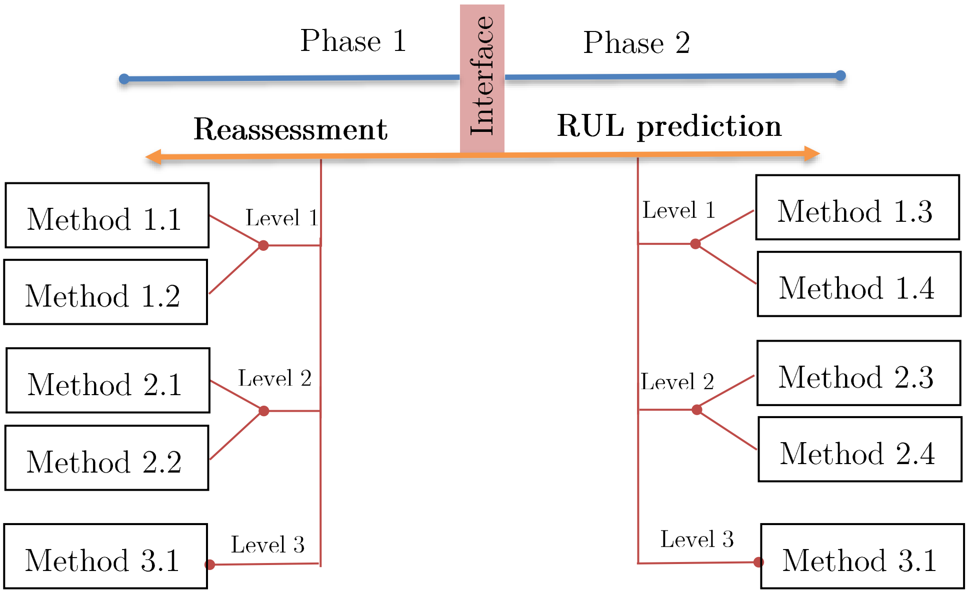


Figure 4-4: The methodology depending on the available data

4.3 Methodology for level 1 data

In this section, the methodology will be studied that can be used when level 1 data is available. This level is the worst-case scenario whereby limit information and new data are available. As shown in Figure 4-5, the information and data can be divided into two parts. For the first part, new information and knowledge can be gained by doing a regulation gap analysis. The rules and regulations for designing an OWT may have been slightly altered during the design life of the structure. The existing OWTs which are at the end of their design life are built according to old standards. Since the reassessment shall be performed based on the current state of the art, a regulation gap analysis is essential to identify the gaps against current regulations [36]. The second part of information can be gained from a comparison between the as-built and the as-designed documents as shown in the green boxes of Figure 4-5 . The green boxes will be explained in more detail.

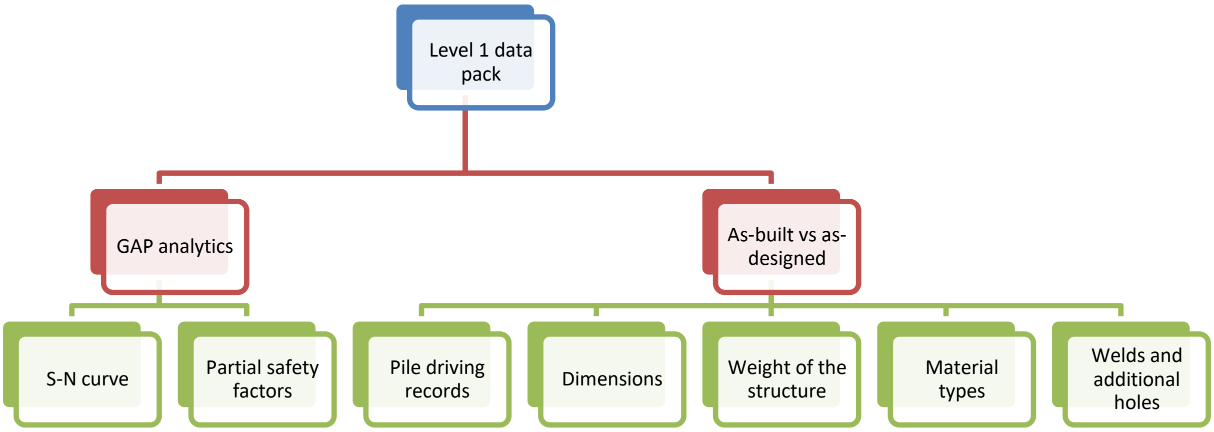


Figure 4-5: Level 1 data pack

S-N curves

For offshore applications, there are three different categories of S-N curves in the design standards for welded joints and it depends on the environment the steel is placed in, that can be air, water or free corrosion. The S-N curves for the air environment have the highest fatigue life, the water environmental (splash zone) has lower fatigue lives and the free corrosion environment (under the water level) is predicted to have the shortest fatigue life. This is because of the corrosive environment which fosters fatigue damage. In total, there are 14 different S-N curves per environment and curve B1 represents the maximum stress range.

For the fatigue assessment, the appropriate S-N curve has to be picked. It depends on the structural detail which is going to be analyzed. Monopile and transition piece are classified as hollow sections according to the Appendix A.9 in DNVGL RP-C203 standard. The joint type is typically a circumferential two-sided butt weld. From this information, the type of S-N curve can be determined. The two S-N curves applying to hollow sections are type C1 and D. The difference between the two is that for type C1 to be applicable the weld had to be machined or ground flush during the fabrication. It is allowed to use the more advantageous S-N curve in this case as the treatment after the welding improves the fatigue life (DNV GL, 2016a)

Safety Factors

The safety factor is another essential part of the fatigue assessment and the last version of this safety factors can be picked for the reassessment. The two safety factors for the fatigue assessment are the material safety factor and the design fatigue factor, as described in chapter 3 and presented in Table 2.

As-built and as-design documents

The as-built and the as-design documents can be compared to get new information for the reassessment. There are a few parameters that may differ from the as-designed documents [37]. The most important parameter is the thickness of the plates. Due to the tolerance, there is a small extra margin on the thickness. However, this is just a fabrication tolerance. It is also possible that different plate thicknesses are used or stiffeners at some locations. While reviewing the reassessment for the foundation structure, the as-built drawings shall be used in the calculation. However, this is just to add the extra thickness on the as-built structure. So, some plates will have exactly the same thickness as the approved drawing, some plates may have 1-2 mm thicker plates. The other parameters that could be different as mentioned in the as-design documents are the following:

- *Pile driving records*

  For fatigue analysis of a foundation pile, the characteristic load effect distribution shall include the history of stress ranges associated with the driving of the pile prior to installing the wind turbine and putting it to service. The pile driving damage is lower than assumed in reality [38].

- *Dimensions of the support structure* (diameter/ length/ thickness/ out-of-roundness/ straightness)

  The dimensions of the support structure is directly related to the environmental forces acted on the structure e.g wave force by Morison equation or wind force, and the stress range e.g. the effects of thickness and diameter on stress calculations. Eccentricity is the misalignment of plates at welded connections measured transverse to the plates. For welded pipes, it is out of roundness that normally will govern the resulting eccentricity. The more the out-of roughness, the more stress concentration factor (DNVGL-RP-C203)

- *The weight of the support structure*
- *Material types.*

  Sometimes the quality is better than used for the calculations (free issued materials).

- *Wide welding due to repairs*
- *Additional holes in the structures*

  Presence of holes localizes the stress at the section and the local stress should be calculated which equals to nominal stress multiple by stress concentration factor (SCF) of the hole.

Taking the described information and data into account, the reassessment and the RUL can now be performed. Since level 1 data is very limited, only simplified methods can be used with assumptions and that will be at the expense of the accuracy. The next methods can be used for the reassessment and the RUL prediction when only level 1 data is available:

Reassessment method 1.1

The first method that can be used for the reassessment is to run load simulations again to perform the deterministic fatigue load calculations according to the current state of the art (gap analytics). The as-design documents can be consulted for the next information:

- The wind farm location and layout
- The wind turbine type and specifications
- The support structure specifications

This information initially needs to be compared with the as-built documents to get the updated specifications. The external condition parameters, as listed in Table 14 in Appendix A can also be taken from the as-design documents since no updated data is available for a level 1 reassessment. An attempt can be done to take real life environmental data from the site of National Oceanic and Atmospheric Administration [39]. The new calculations can be done by a generic load simulation model if the specific turbine model is not available. This is allowed according to DNVGL-ST-0262 standard [19].

Reassessment method 1.2

Another method to reassess the structure is using a probabilistic method. The inputs for this method are the turbine and support structure specifications that can be taken from the as-designed or the as-built documents, the environmental input parameters and the uncertainty distributions of the parameters. Since the as-built documents are available, the uncertainties of the dimensions are small. The uncertainty distributions regarding the environmental conditions can be taken from the literature. The probabilistic calculation procedure is explained in section 3.3. The most difficult part of this method is to define the probabilistic fatigue limit state in the proper way. Different probabilistic methods can be used for the calculations like Monte Carlo, FORM, SORM, etc. The methods are listed in Appendix E.

RUL prediction method 1.3

For the remaining useful life prediction, method 1.1 can be extended until the fit-for-purpose criteria described in section 4.1 is reached. With reassessment, the yearly damage can be calculated. To determine the RUL, the yearly probability of failure can be counted further until the limit is reached.

RUL prediction method 1.4

This method is also an extension of method 1.2. The calculated yearly probability of failure calculated for the reassessment can be counted further until the yearly reliability target of 10^{-4} is reached.

An overview of the level 1 methodology is presented in Figure 4-6. As shown, two methods can be used for the reassessment and two methods for the RUL prediction. The interface is for combining the reassessment methods with the RUL prediction methods. If the combined methods are of the same type (deterministic - deterministic or probabilistic - probabilistic), then there is no problem with combining the outputs. When the combined methods are not of the same type (deterministic - probabilistic), a translation is needed which is explained in section 4.5.

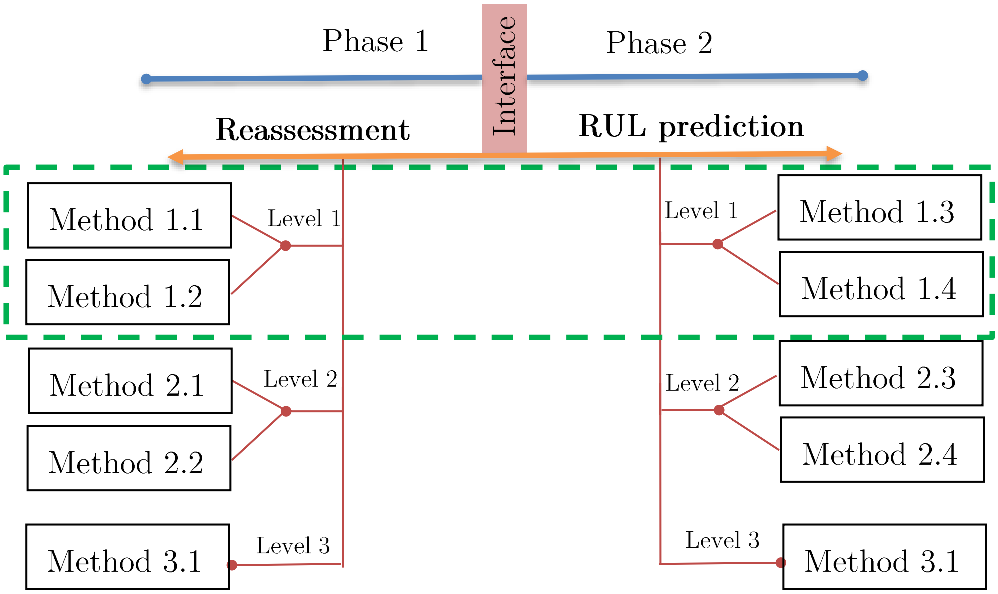


Figure 4-6: Level 1 methodology

4.4 Methodology for level 2 data

In this section, the methodology will be studied that can be used when level 2 data is available. This level is the most common scenario in which next to the level 1 data pack also updated site data and inspection data is available as presented in Figure 4-7. The environmental data measurement period should be at least 12 months to cover seasonal variations [17]. The green boxes under the updated site data and the inspection data will be explained in more detail.

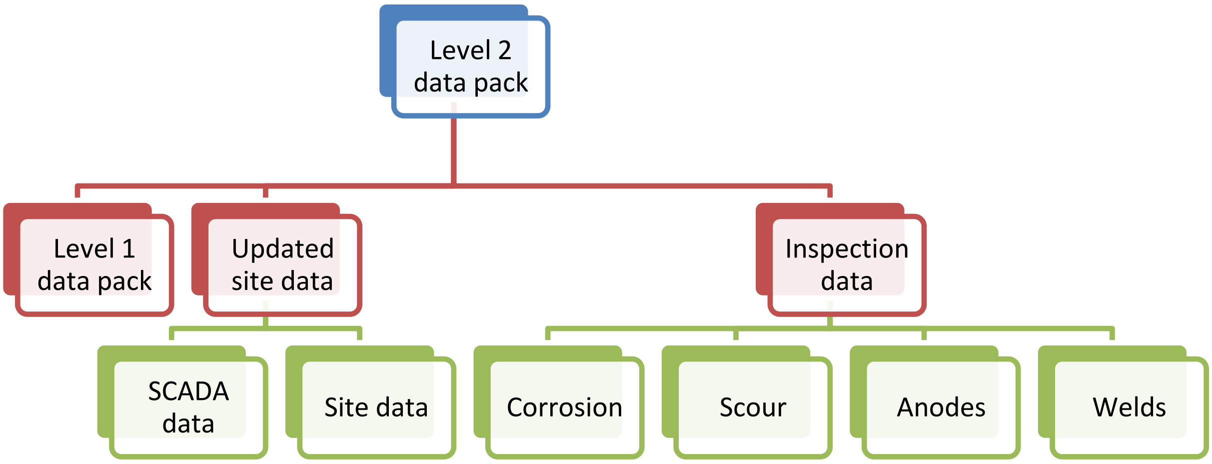


Figure 4-7: Level 2 data pack

SCADA data

The supervisory control system and data acquisition (SCADA) is the control system of the wind turbine that measures the operational data of the wind turbine and the environmental data. The operational data consists of nacelle angle, pitch angle, rotor speed, production, and availability. The environmental data consists of wind speed and the orientation of the turbine. This data is exported via a FTP server and is collected during the lifetime of the turbine [40]. The wind speed data on top of the nacelle of the turbine is measured behind the blades, therefore a correction is needed to get the actual wind speed. The updated parameters can be used again for the fatigue calculations.

Updated site data

Beside the SCADA data, the environmental parameters can be measured by an offshore met mast, lidar system or a wave buoy. The next parameters can be monitored:

Aerodynamics

- Turbulence spectrum
- Mean wind speed & direction
- Wind shear
- Air density

Hydrodynamics

- Water depths
- Significant wave height
- Mean zero-up crossing period
- Wave period
- Current speed

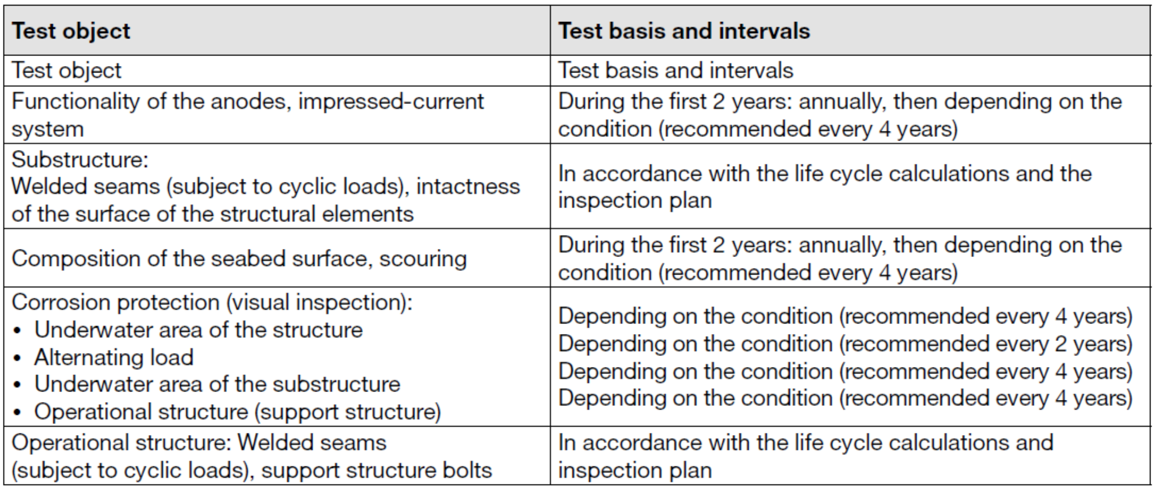
Inspection reports [41]

During the operational phase, regular periodic inspections must be carried out. The appointed inspector shall issue the respective certificate of conformity in line with the test and inspection plan for periodic inspections.

Differentiations are made between two types of offshore structure inspections: Periodic and event-driven inspections. A periodic inspection specifically serves the purpose of examining any changes that become apparent with respect to the status of the support structure. An event-driven inspection must be conducted whenever damage is to be expected as a result of a specific event. If components and structural elements are subjected to a detailed inspection as the result of an event-driven inspection, the periodic inspections start afresh. The intervals specified for testing the various elements are merely guidelines. These must be adapted according to the location and type of offshore structure in question and must be accordingly noted in the test and inspection plan.

The operator shall submit the inspection reports resulting from the periodic inspections to the appointed inspector. The appointed inspector shall then assess the results of the periodic inspections with respect to the structural integrity of the overall structure and then compile a comprehensive report. The minimum requirements pertaining to the periodic inspections of the support structures are listed in Table 4.

Table 4: Minimum requirements with regards to the periodic inspections of support structures [41]



Test object	**Test basis and intervals**
Test object	Test basis and intervals
Functionality of the anodes, impressed-current system	During the first 2 years: annually, then depending on the condition (recommended every 4 years)
Substructure: Welded seams (subject to cyclic loads), intactness of the surface of the structural elements	In accordance with the life cycle calculations and the inspection plan
Composition of the seabed surface, scouring	During the first 2 years: annually, then depending on the condition (recommended every 4 years)
Corrosion protection (visual inspection): • Underwater area of the structure • Alternating load • Underwater area of the substructure • Operational structure (support structure)	Depending on the condition (recommended every 4 years) Depending on the condition (recommended every 2 years) Depending on the condition (recommended every 4 years) Depending on the condition (recommended every 4 years)
Operational structure: Welded seams (subject to cyclic loads), support structure bolts	In accordance with the life cycle calculations and inspection plan

Taking the new information and data into account, the reassessment and the RUL can now be performed. Since the level 2 data is more comprehensive than the level 1 data, the reassessment and the RUL prediction is more accurate. The methods that can be applied for the two phases are as follows:

Reassessment method 2.1

The first simple reassessment method is using a lookup table approach to estimate the fatigue load damage of OWTs. For this method, it is assumed that the simulations from the design phase are known which are called the load simulations. The load simulations are used to determine the internal loads during certain operational and external conditions. In the design phase, a large number of different simulations are run by engineers for a different combination of parameters such as wave height, wind speed, turbulence intensity and the transient operational state of the turbine. Due to a large number of simulations with different input parameters, it can be assumed that for a random operational state and the associated environmental conditions (measured wind parameters, wave parameters, and SCADA data) there is a simulation that gives with a certain accuracy the corresponding fatigue damage.

The lookup table can be used when there are measurements, for example, the ten-minute interval for the average wind speed, the significant wave height and the parameters of the SCADA data. The database of all the simulations can now be consulted to determine the fatigue damage for that ten-minute interval with the given conditions. The total fatigue damage can be estimated by using the same approach for all the intervals. The number of ten-minute intervals can be reduced by using bins with small bin size for the measured parameters, for example, the wind speed.

The lookup table has also limitations, for example, if an event happens which is not taken into account in the simulations that are stored in the database. An example is when a collision with the support structure happens. The lookup table cannot be used to determine the damage of such events. When the lookup table is not sufficient enough, renewed fatigue calculation can be done by rerun the numerical wind turbine models to simulate loads and structural response, taking into account the monitored parameters as described in this section. It is recommended to use a specific turbine model according to DNVGL-ST-0262 standard.

Reassessment method 2.2

A probabilistic assessment is another method to reassess an existing OWT support structure. The uncertainties and the variability in load strength and material resistance are no longer covered by partial safety factors. A general framework for the probabilistic assessment of a structure is described by the structural reliability analysis method (SRA). This method consists of a few steps to determine the reliability of a structure. Figure 4-8 describes these steps and each step will be explained in more detail for an OWT. The first five steps are relevant for the reassessment which are described below.

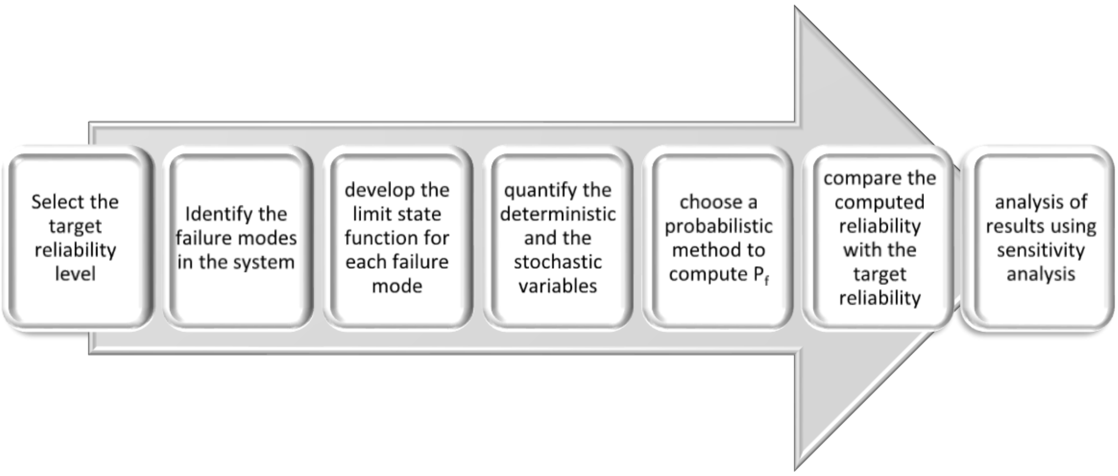


Figure 4-8: SRA flowchart

- **Selection of the target reliability level**

Reliability is defined according to the International Standard Organization (ISO) 2394 as:

> *‘’the ability of a structure or structural element to fulfill the specified requirements, including the working life, for which it has been designed.’’*

A target of design reliability level is the reliability level (safety critical failure per year) the structure is designed to stay at, or below, for the duration of its design life. This can be expressed as the inverse probability of failure: the lower the probability of failure the more reliable a structure. A target reliability level is determined through consideration of various factors including the consequences of failure (impact to people, the environment, economic loss, and social impacts) balanced against the cost of measures that would reduce the risks of failure. Various industries have worked to establish target reliability levels (where needed) that are acceptable to the next stakeholders: regulatory, financial, owner, and the public. These are often communicated in the form of standards and/or regulatory requirements, and are often communicated implicitly in design, siting or operating requirements.

The ISO 2394 provides some general guidance on target reliability levels for a vast range of engineering structures. More specifically to wind turbines, the DNV-OS-J101 standard presents an interpretation of the IEC 614000-1 edition 2 standard in terms of target reliability:

> *''a wind turbine is designed to a reliability level expressed as a nominal annual probability of fatigue failure of one in ten thousand (10^{-4})''*

i.e. the design lifetime is the year at which the expected annual probability of failure reaches 10^{-4} if the site conditions perfectly reflect the IEC design inflow conditions. This will be the target reliability assumed for the rest of the report: an annual Pf of 10^{-4} (which is equal to 1 failure in 10.000). It is noted that the literature often uses two different terms when referring to the target probability of failure, either the annual or the cumulative probability of failure. Both terms are related to each other as expressed in Equation (4).

- **Identification of the failure modes in the system**

To identify failure modes of a structure, fault trees are used nowadays in all engineering fields to show the different ways that can lead to failure of a structure. The idea of a fault tree is to learn how the system works, as described by Dan Golding:

> *''To design systems that work correctly, we need to understand how they can go wrong''*
>
> Dan Golding, NASA, 2000

Figure 4-9 shows the fault tree of an offshore wind turbine with possible events that can occur. Each event in the fault tree can lead to a failure of the system, illustrated by the ''OR'' sign. The events that are illustrated here are the limit states for offshore wind support structures, described by DNV GL. As mentioned earlier, the FLS is the limit state that is considered in this report.

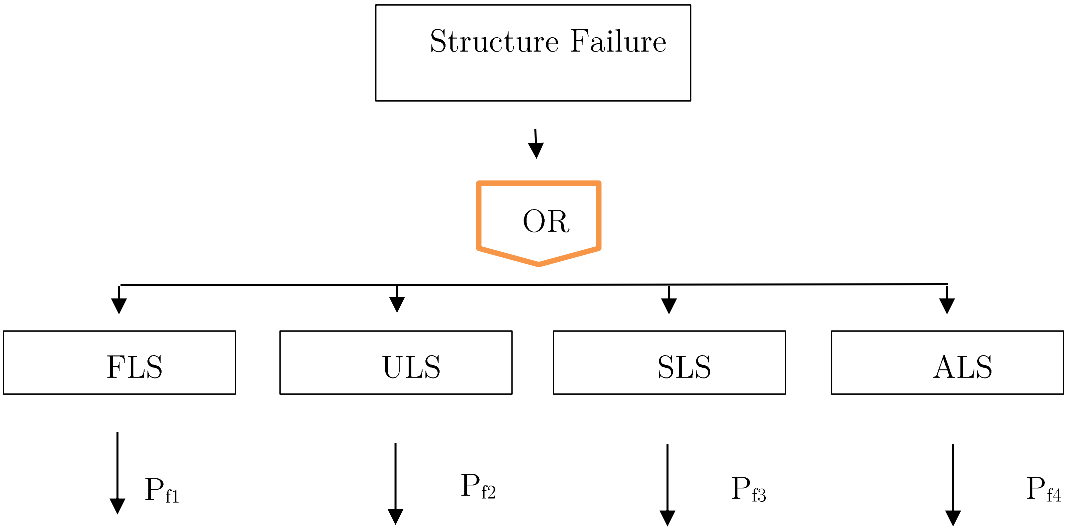


Figure 4-9: Fault tree

- Develop the limit state function for the FLS of an OWT support structure

The limit state function starts with the general expression, Equation (5):

$$G = R - S \tag{5}$$

where,

G = fatigue limit state function
R = resistance term → Miner's rule
S = load side → fatigue damage

Applying the above conditions gives Equation (6):

$$G = X_{miner's} - D \tag{6}$$

- **Quantifying the deterministic and the stochastic variables**

For probabilistic analysis, the most difficult part is to determine the uncertainties. There are various uncertainties during the whole probabilistic analysis. The uncertainties can be determined as follows (if the relevant experience is not enough):

- Using the suggested values given in the standards/rules
- Discussing with more experienced people and try to get suggestions from experts in the relevant field.
- Performing sensitivity studies by assuming different values (usually mean value could be taken from the deterministic analysis, and standard deviation or coefficient of variation (c.o.v.) could be assumed with different values, to check the influence.)

- **The probabilistic methods to compute P_f**

After defining the limit state function and their variables, the reliability of the OWT support structure can be analyzed. There are different levels of reliability methods to calculate the probability of failure for the defined limit state function. The different reliability methods are explained in Appendix E. For each reliability level, different reliability methods can be chosen. The most common reliability methods are:

- Simulation techniques where a lot of samples are used to estimate the probability of failure, like Monte Carlo.
- First Order Reliability Method (FORM) is another method for estimating the probability of failure by linearizing the limit state function in the design points.
- The Second Order Reliability Method (SORM) is another method where a quadratic approximation is used to the limit state function.

RUL prediction method 2.3

The RUL can be predicted by extending method 2.1 until the fit-for-purpose criteria described in section 4.1 is reached. With reassessment, the yearly damage can be calculated. To determine the RUL, the yearly probability of failure can be counted further until the limit is reached.

RUL prediction method 2.4

The SRA can be used again to determine the RUL by taking the last two steps also into account. The uncertainty distribution can now be taken into account in the calculations. The OWT can keep operating until the reliability target has reached.

An overview of the level 2 methodology is presented in Figure 4-10.

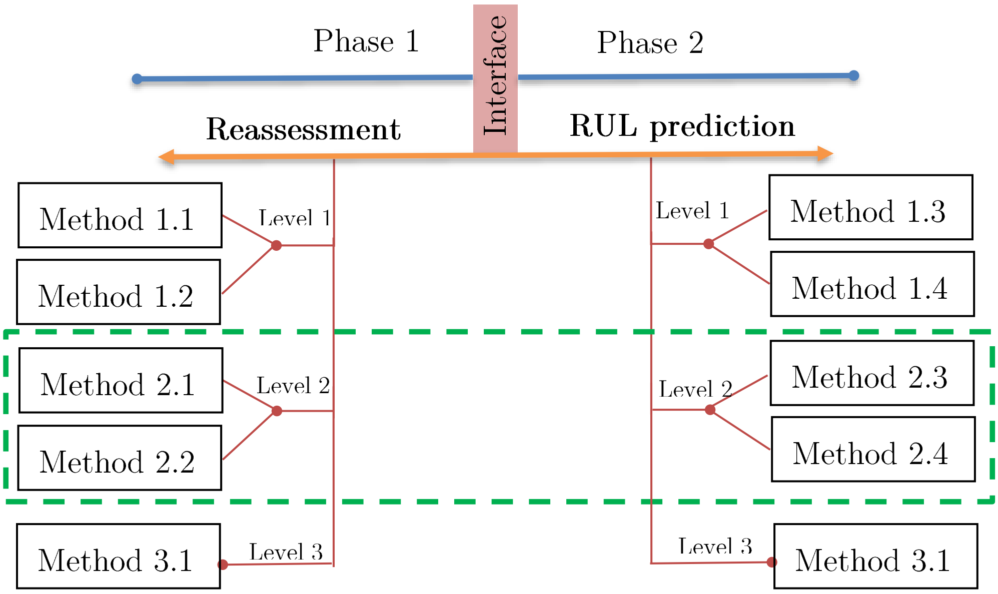


Figure 4-10: Level 2 methodology

4.5 Methodology for level 3 data

The methods for level 1 and level 2 are based on a combination of updated data, inspection data, and renewed fatigue damage simulations to reassess the OWT support structure and to predict the RUL. If the loads that the OWTs are exposed to are known, a better approximation can be made of the actual state of the OWT. The level 3 data pack is the most comprehensive level of available data that consists of stress/strain and acceleration measurements besides level 1 and level 2 data, as presented in Figure 4-11.

Monitoring the stress/strain at critical locations have a great added value to determine the fatigue damage with more certainty, to validate the numerical simulations, and to assess the remaining lifetime of the structure. One limitation is that the amount of existing OWTs with structural measurements are around 15% [38] which makes this approach limited to access. Another limitation is that the available data might not be at a critical point for the fatigue lifetime prediction. Furthermore, at some critical points, direct measurements of the strains are not feasible [42].

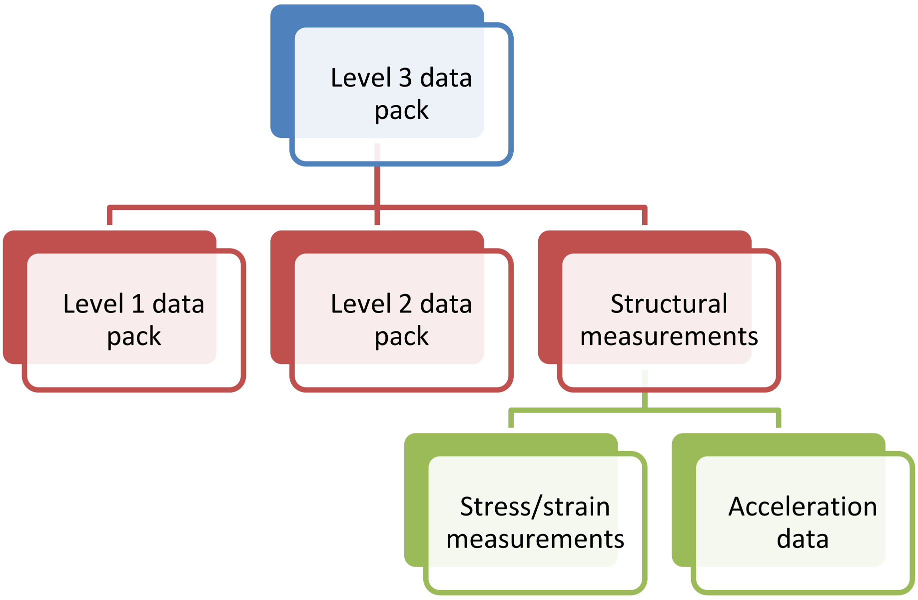


Figure 4-11: Level 3 data pack

Reassessment and RUL prediction method 3.1

A method for the reassessment of the OWT support structure is combining the measured stress/strain response with the SCADA data and if possible with the met-mast/wave buoy data. The measured data can be used to make a comparison between the simulation response and the measured stress/strain response. This comparison is specific for the conditions which will be analyzed and allows to estimate the uncertainty of the design in terms of a simulation error for a specific environmental and operational state. This error can only be assessed in a static way by calculating at least 10 simulated damages of 10 minutes intervals with the 10 minutes measured and post-processed strain measurements. The post-processed strain measurements can now be used for direct load validation and for combining the measured strain/stress data with the SCADA data, met-mast and wave buoy data to determine their fatigue damage and to predict the RUL more accurate.

The post-processing is explained by Clemens Hübler et al. [43] and consists of a few steps as presented in Figure 4-12. First, the strain measurements are stored in time series for every ten minutes. The strain is then converted into stresses by using Hook's law and are orientated in the same direction in case of multiple sensors. Then the rainflow counting algorithm is used to obtain the number of cycles for each stress level at the location of the sensor. Determining the stresses for another location is possible according to Wilberts by extrapolation of the location. For the extrapolation, the geometry and the material parameters like the diameter, wall thickness, and height levels are essential which can be extracted from the as-design or the as-built documents. The translation of the measured stress at one location to another location can be done by following the next steps:

a) Translate the measured stress to the bending moment for the measured location.
b) Calculating the thrust force at the hub height with the bending moment of step a.
c) Calculating the bending moment of the desired location my multiplying the thrust force at hub height by the distance to the desired location.
d) Now the bending moment of the desired position can be translated back to the stress at that location.
e) The final step is to apply correction factors to calculate the real stresses at the hotspots.

For the equations of the above steps, the study of F. Wilberts can be consulted [44]

The Miner's rule can now be applied to determine the short-term ten-minute damages. The short-term damages can be linked to the ten-minute SCADA data and the met-mast/wave buoy data which can be divided into bins. The total damage can now be calculated for the whole operational period. The RUL can be predicted by taking the probability of the expected wind/wave data multiplied by the calculated damage of the measured stresses. The minimum measurement length that is required to get reliable lifetime estimations according Hübler is 9-10 months to achieve unbiased mean values.

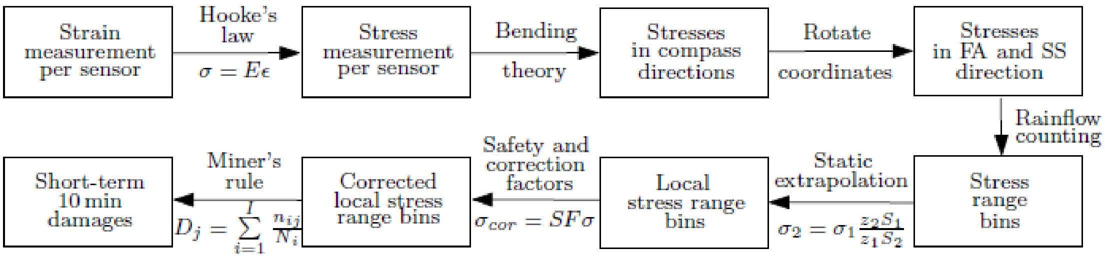


Figure 4-12: Flowchart from strain measurement to RUL [43]

The above static extrapolation method for determining the stresses at other locations is a simplified approach which is only valid for monopiles without the occurring of high wave loads. That is because of the assumption that there is a linear relationship of heights and stresses since it is assumed that all the significant loads are due to the thrust load [43]. It is a good direct measurement approach for the location of the strain sensors. This point might not be critical for the fatigue lifetime prediction, therefore a measured response needs to be extrapolated to a critical point or all points on the structure as mentioned earlier. This additional problem is referred to as state estimation [45] [46]. The benefit of extrapolating the response to any point on the structure is that all locations, also the critical locations, can now be assessed. However, the process of extrapolation adds complexity and potential uncertainty (of to the process) since the extrapolation is mostly based on physical models from the design process. In order to perform full-field response extrapolation, a more elaborate set of measurement data is needed which has to satisfy a number of conditions [47]: in each direction (fore-aft and side-side), at least as many accelerations as dynamically relevant modes (2-3) and at least as many strain measurements as acceleration measurements are required. Also, for each sensor, the location must be chosen in a way that the sensor is not located at a node of a relevant node. There are many algorithms to extrapolate the acceleration and strain measurements to a full field response. The multi-band modal expansion algorithm is one of the methods and is presented by Iliopoulos et al. [48]. The approach consists of two main steps. The first step is a modal decomposition using strain and acceleration measurements and the second step is the integration of the estimated modal acceleration to modal velocity and modal displacements through the frequency domain.

An overview of the level 3 methodology is presented in Figure 4-13 .

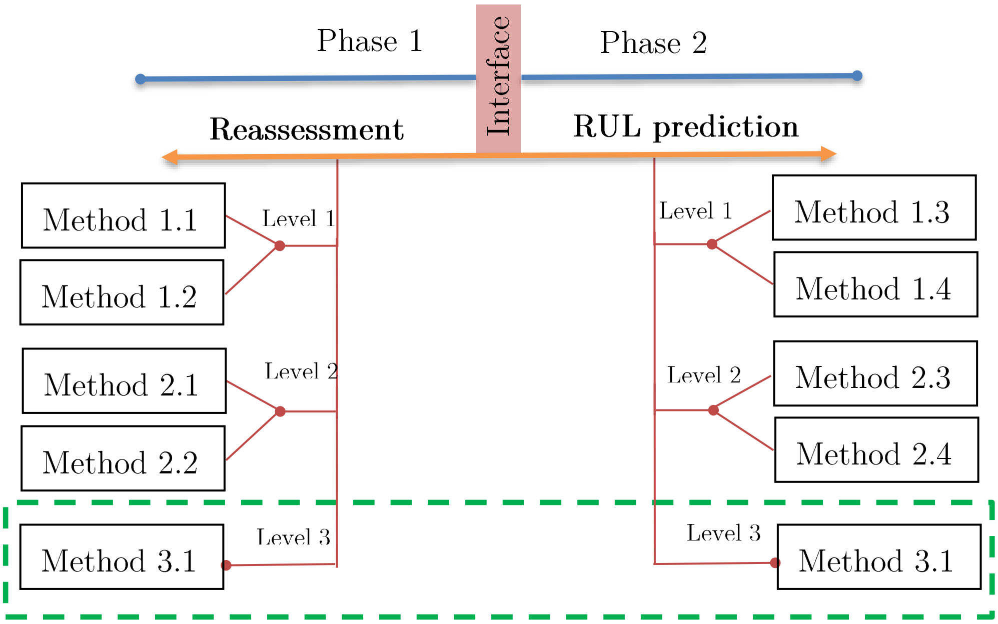


Figure 4-13: Level 3 methodology

4.6 Procedure for combining the different methods

This section studies the interface between the reassessment methods and the RUL prediction methods. When the reassessment method and the RUL prediction method are not from the same type (deterministic - deterministic or probabilistic - probabilistic), a translation is needed from the deterministic fatigue damage after reassessment to a cumulative probability of failure. When this relation is known, the probability of failure can be further counted from the point the reassessment stops until the predicted moment in the future when the target reliability of 10^{-4} is reached. The relation between the deterministic fatigue damage and the cumulative probability of failure is given in Figure 4-15 which is taken from the DNVGL standard. Before Figure 4-15 can be used, a few steps should be made which will be explained below.

1. We need to know what the DFF or the material safety factor is, that is used for the deterministic calculations. The calculated deterministic fatigue damage should be divided again by the DFF or the material safety factor to get the real damage.
2. For the next step, Figure 4-14 can be used. The Figure shows the relation between DFF and the failure probability. For annual target reliability of 10^{-4}, as described by the design standards, the DFF is approximately 9.
3. The calculated deterministic fatigue damage should now be divided by a factor 9. The maximum allowable fatigue damage for the deterministic approach and the probabilistic approach is the same.
4. Figure 4-15 can now be used to combine the deterministic and the probabilistic fatigue approach. This will be illustrated in the case study.

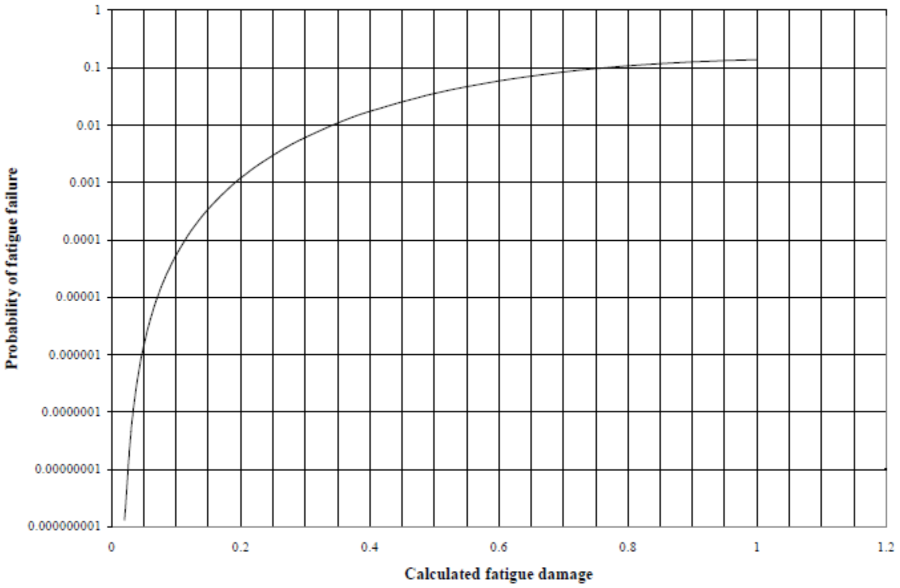


Figure 4-15: Failure probability as function of calculated fatigue damage [31]

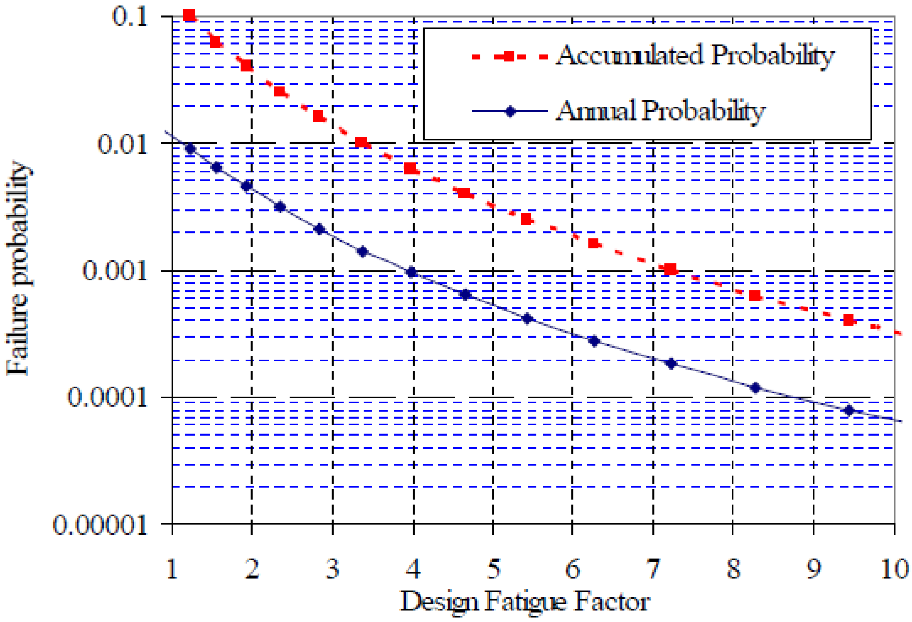


Figure 4-14: Fatigue failure probability as function of design fatigue factor [31]

Chapter 5

5 Case Study

In this chapter, a case study will be studied to illustrate the proposed framework and methodology as described in chapter 4. A simplified structure will be used for the fatigue calculations instead of a detailed offshore wind turbine model, with the aim of making the required steps of the framework understandable.

In the first section, the fundamental idea of the case is described together with an overview for the FLS calculations, since a deterministic and a probabilistic fatigue calculation process will be worked out. The structural model and the load model are defined in section 5.2. Section 5.3 presents the wave loading acting on the structure. Further, the dynamic behavior of the simplified structure will be taken into account by the dynamic amplification factor (DAF), as described in section 5.4. The initial fatigue life is calculated in section 5.5. The reassessment and the remaining useful life prediction, which are the main parts of the framework, will be demonstrated in section 5.5. An overview of this chapter is presented in Figure 5-1.

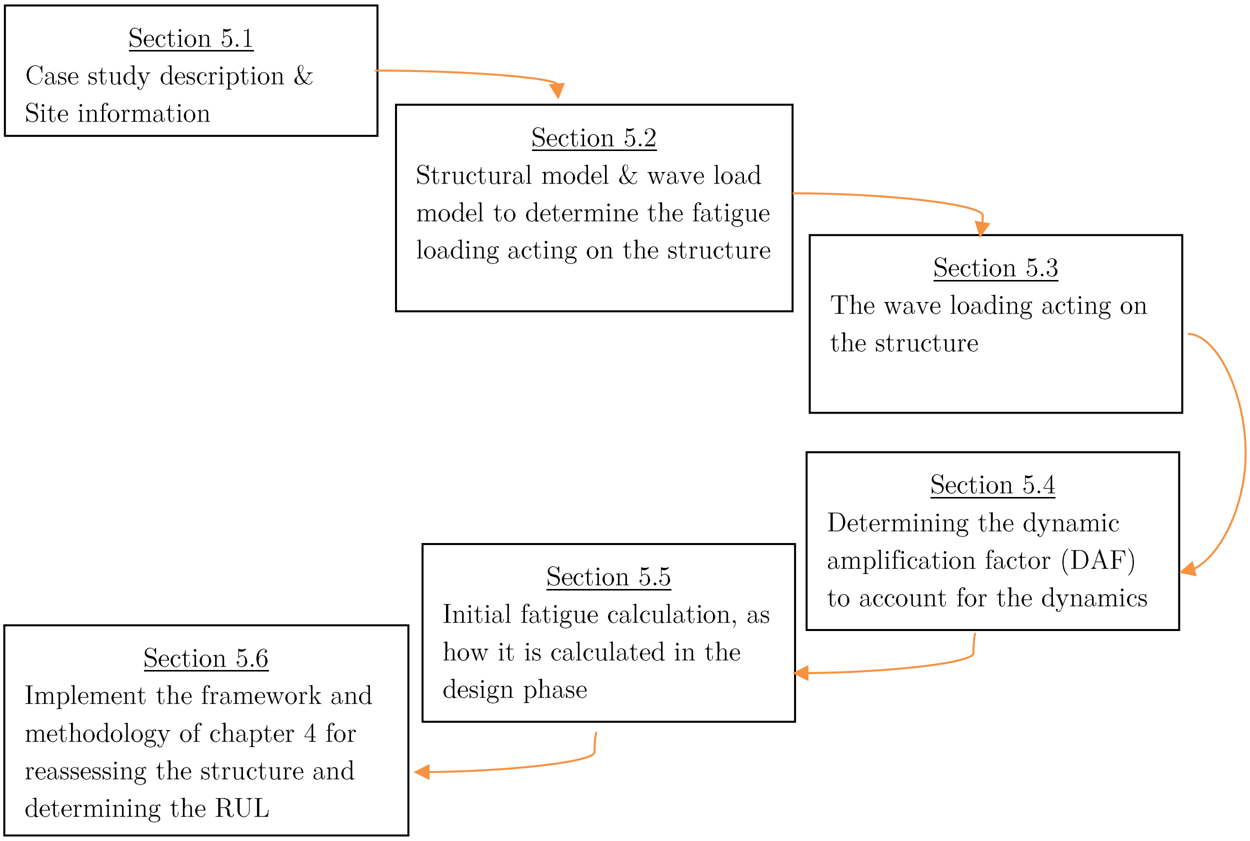


Figure 5-1: Overview Case Study

5.1 Case study description

To have a thorough look into the framework and methodology of chapter 4, a numerical case study of a simplified offshore wind turbine (OWT) is engaged. An existing offshore wind farm in the North Sea is selected as an input for the calculations and will be discussed later on in this section. Since the focus of the current report is on lifetime extension (LTE) of OWT support structures, only the support structure will be analyzed by this case study. As mentioned in the introduction of this chapter, the proposed methodology of chapter 4 will be worked out through this case study.

Figure 5-2 gives an overview of the steps that are taken for this case study. First, the fatigue lifetime is calculated for the simplified structure using a deterministic approach according to the design standards. This is indicated in Figure 5-2 as the initial fatigue damage. Then the structure is reassessed after 15 years of operating. Usually, the OWT is monitored in the operational phase and the new information and data gained during the operational phase will be used again for the reassessment, as described in chapter 4. Since the access for the monitored data during the operational phase is limited in this case, the initial site data will be used for the reassessment by changing the wave height by ±5% and the new fatigue damage will be calculated for the 15 years of operation. Subsequently, the remaining useful life is predicted by using probabilistic methods. The methodology used for this case study is the same methodology as described in chapter 4 for the level 2 available data.

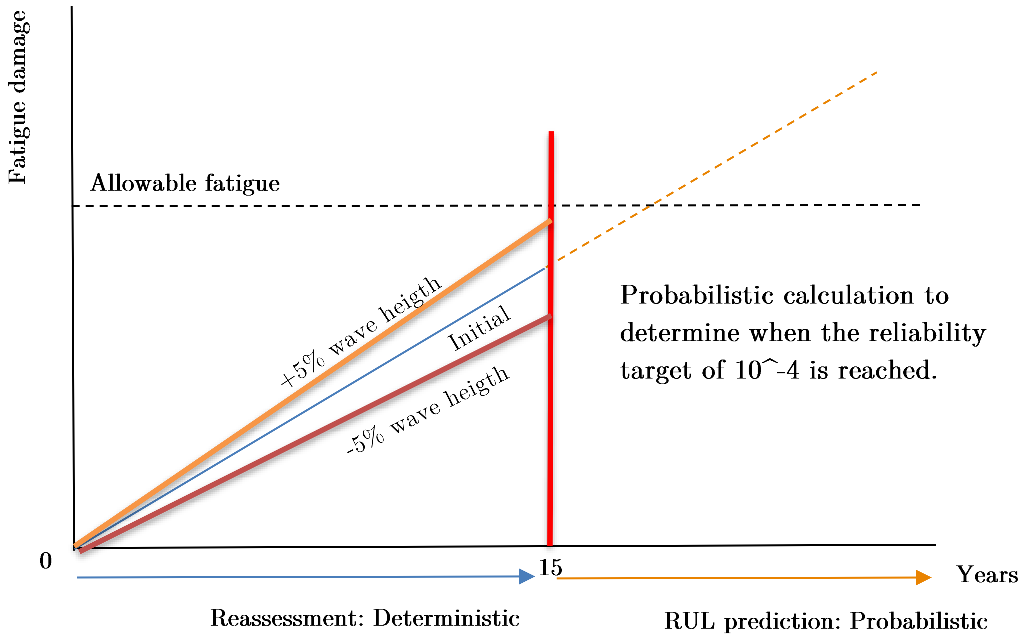


Figure 5-2: Proposed framework for reassessing and RUL prediction

Location and site information

The general information used for this case is taken from several sources: information about the location is from RVO [49], Figure 5-3; the metocean and the bathymetry information are from NOAA [39], listed in Table 5 and Table 6; the turbine specifications are from the Gemini webpage [50], listed in table 7

Figure 5-3: Location of the Gemini wind farm [49]

Table 5: Metocean characteristics

Parameter	Units	Return period [years]			
		Month	5	25	1000
Maximum wave height	[m]	5.5	8.3	9.3	11.5
Wave period	[s]	3.5	4.5	7.5	13.5
Wind speed	[m/s]	18.6	23.2	24.7	27.6
Current speed	[m/s]	0.2	0.3	0.6	1.1

Table 6: Site conditions

Parameter	Units	Value
Water depth (LAT)	[m]	28-36
Tidal range	[m]	2.7
Storm surge	[m]	0.9

Table 7: Specifications of the Gemini wind turbines

Siemens SWT4.0-130	Units	Value
Power	[MW]	4
Mass top	[tons]	200
Hub height	[m]	125
Rotor diameter	[m]	130

5.2 The simplified static offshore wind turbine model

The fatigue calculations in this case are made with a simplified static model. A benefit of a simplified model is that the computational time is much quicker than any aeroelastic software. Although a simplified static model is less accurate than a dynamic model with use of aero elastic software, a simplified static model is an accepted approach for this case since different methods are compared, whereby all of them have the same uncertainties due to the simplifications.

5.2.1. Structural modeling

The offshore wind turbine is simplified to a cantilever beam with a mass on top and clamped at the bottom of the seabed. The beam is discretized into intervals of 1[m] to calculate the wave loading on each section of the beam using a static model. The wave loading is acting in the region of the maximum water depth D_{max}. The diameter is taken as constant and it changes only in the region where marine growth is applicable. An illustration of the simplified model is shown in Figure 5.4 and the specifications for the simplified model are summarized in Table 8.

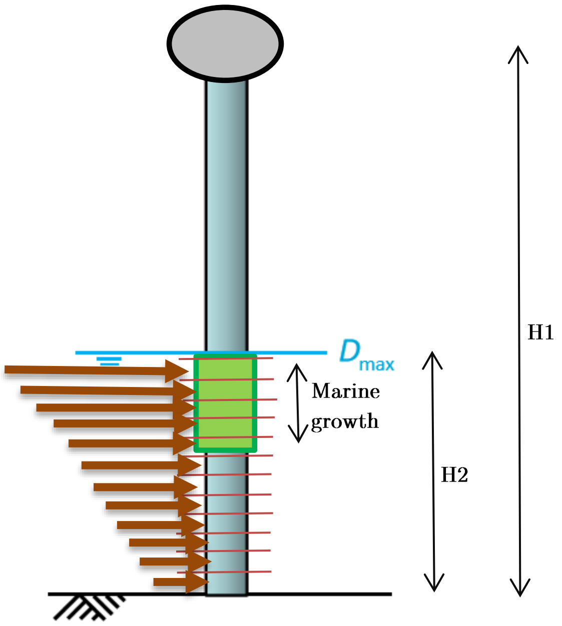


Figure 5-4: Simplified static offshore wind turbine model

Table 8: Specifications of the simplified model

Simplified model	Units	Value
Mass top	$[tons]$	200
Hub height (H1)	$[m]$	125
Max. Waterdepth (H2)	$[m]$	36
Marine growth	$[m]$	15 below D_{max}
Diameter	$[m]$	5
Wall thichness	$[m]$	0.08
Water density	kg/m^3	1025
Steel density	kg/m^3	7850
Yield stress	MPa	355
Wave load	Will be explained in detail	

5.2.2. Wave load modeling

The wave force that is acting on the simplified offshore wind turbine is calculated according to the Morison's Equation (7), that consists of the inertia and the drag force. The water particle motion can be described by Airy linear waves if the deep water theory is valid [51]. This assumption is only valid if the water depth is relatively bigger than the wave length. The wave height above the mean sea level (MSL) is calculated with the Wheeler Stretching theory [50]. The Cd and Cm should be determined specifically for each wind farm location, but in this case the typical values of Cm = 2.0 and Cd = 0.7 are used for the fatigue calculations according to ISO 19901-1.

$$F(z) = ¼ \,.\, \rho_w \,.\, C_m \,.\, D^2 \,.\, a \;+\; ½ \,.\, \rho_w \,.\, C_D \,.\, D \,.\, v_{tot}|v_{tot}| \qquad (7)$$

where

$F(z)$	hydrodynamic load	$[N/m]$
$f_{inertia}$	inertia force	$[N/m]$
f_{drag}	drag force	$[N/m]$
ρ_w	water density	$[kg/m^3]$
C_m	inertia coefficient	$[-]$
C_D	drag coefficient	$[-]$
v_{tot}	total water particle velocity	$[m/s]$
a	water particle acceleration	$[m/s^2]$
D	diameter of the monopile	$[m]$

The total horizontal fluid particle velocity v_{tot} a combination of the horizontal wave speed and the current speed, see Equation (8). In this case, the current speed is not considered since it is constant and not important for fatigue calculations.

$$v_{tot}\,(x,z,t) = v_c(z) * \Upsilon_{block} + v_{wave}(x,z,t) * \Upsilon_{spread} \tag{8}$$

The spreading factor Υ_{spread} and blockage factor Υ_{block} are both set to zero since only one monopile with undisturbed wave inflow conditions is considered. The only term left from Equation 3 is the wave speed v_{wave} that is calculated with Equation (9).

$$v_{wave}(x,z,t) = \zeta\,.\,\omega\,.\,e^{-kz}.\cos(kx - \omega t) \tag{9}$$

The acceleration of the water particles includes only the wave motion since the current velocity is constant and therefore its derivative will be zero. The acceleration of the wave motions can be calculated with Equation (10).

$$\frac{\partial v_{wave}\,(x,z,t)}{dt} = a\,(x,z,t) \quad = -\Upsilon_{spread}\,.\,\zeta\,.\,\omega^2\,.\,e^{-kz}\,.\sin(ks - \omega t) \tag{10}$$

The maximum hydrodynamic force is when the inertia and drag force are combined and when they are 90° out of phase and is calculated with Equation (11).

$$F(z) = \sqrt{f_{inertia}^2 + f_{drag}^2} \tag{11}$$

5.3 Wave loading acting on the structure

The only load that is considered in this case is the wave loading. The permanent loads, the wind speed and the current loads do not influence fatigue since a static model is used for the fatigue calculations. A summary of the approach used in this case study is presented in Figure 5-5. The wave scatter diagram is made from the metocean data and is lumped to reduce the calculation time. The shear force, the bending moment, the maximum stress and the stress ranges are calculated for the simplified structure. In this section, each step of Figure 5-5 will be explained in detailed.

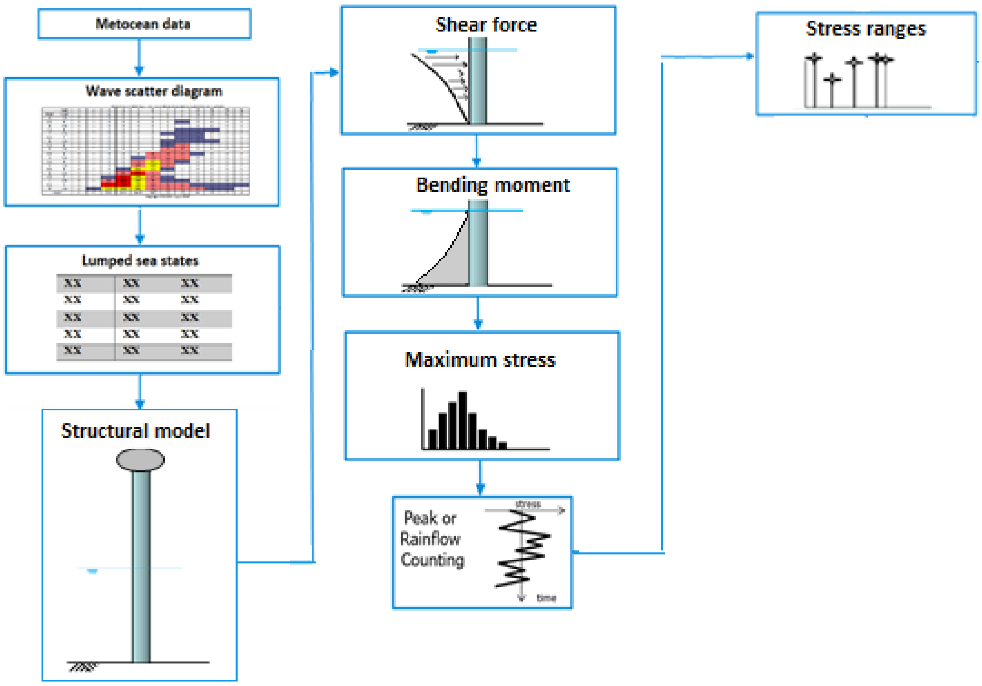


Figure 5-5: Case study overview

Next to the parameters listed in Table 8, the wave data is an important input for the Morison Equation (7) to calculate the wave loading. The wave conditions for the selected site are described by a two-dimensional scatter diagram with classes for the wave height H and the period T. The scatter diagram used for this case study is presented in Appendix G and is made by following the next steps:

- Establish (H_s,T_p) or (H_s,T_z) scatter diagrams from long-term hindcast data series.
- Simulate a high number of 3-hourly time series for every cell in the long-term scatter diagram, assuming an appropriate spectral form (such as JONSWAP).
- Determine H, T scatter from the time series for every cell in long-term scatter diagram (short-term statistics).
- Combine long term and short-term data (i.e. weighting simulated short term H, T) by the probability of occurrence of the H_s, T_z sea states)
- Generate long term (H, T) scatter diagram

The scatter diagram is subsequently lumped to reduce a full-sea-state to 30 load cases by weighting environmental parameters. Here, the wave period is lumped over wave height where certain bin sizes (0.5 [m]) are considered for the wave height. The wave periods that lies in this range are weight averaged, based on their probability of occurrence. This method of lumping is suggested by Martin Kühn [52]. The lumped sea states are listed in Table 9.

Table 9: Lumped sea states

Wave type [-]	Wave height [m]	Wave period [s]	Number of waves [-]
1	0,25	2,7	4967262
2	0,75	4,2	2388241
3	1,25	5,0	910572
4	1,75	5,5	383743
5	2,25	5,9	161292
6	2,75	6,1	67534
7	3,25	6,4	27712
8	3,75	6,6	11383
9	4,25	6,8	4537
10	4,75	6,9	1849
11	5,25	7,1	762
12	5,75	7,2	316
13	6,25	7,3	132
14	6,75	7,5	57
15	7,25	7,5	25
16	7,75	7,6	11
17	8,25	7,6	5
18	8,75	7,7	2
19	9,25	7,7	1
20	9,75	7,8	0
21	10,25	7,9	0
22	10,75	8,1	0
23	11,25	8,3	0
24	11,75	8,5	0
25	12,25	8,7	0
26	12,75	8,9	0
27	13,25	9,1	0
28	13,75	9,3	0
29	14,25	9,4	0
30	14,75	9,5	0
Total	-	-	8.925.438 waves per year

Shear force and bending moment

The output of the Morison equation is the wave force for every 30 wave types of Table 9, for each section[1] of the beam. In order to calculate the stresses, the shear forces and the bending moments are determined for each wave type. This is done by using the Equations (12) and Equation (13).

$$F_{shear}(z) = \int_{z}^{Top} F_{x,environmental}(z)\, dz \tag{12}$$

$$M_{bending}(z) = \int_{z}^{Top} z\, F_{x,environmental}(z)\, dz \tag{13}$$

The shear forces and bending moments are plotted for the different wave types in Figure 5-6 and Figure 5-7, respectively. The shear force and the bending moment are maximum for the beam at the point where it is clamped at the bottom. This point is the most critical point and the fatigue should be calculated for this location.

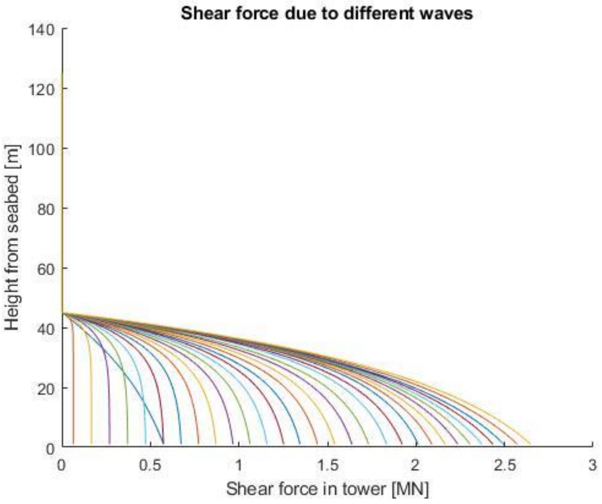


Figure 5-6: Shear force for the different wave types

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[1] The beam is discretized into intervals of 1[m]

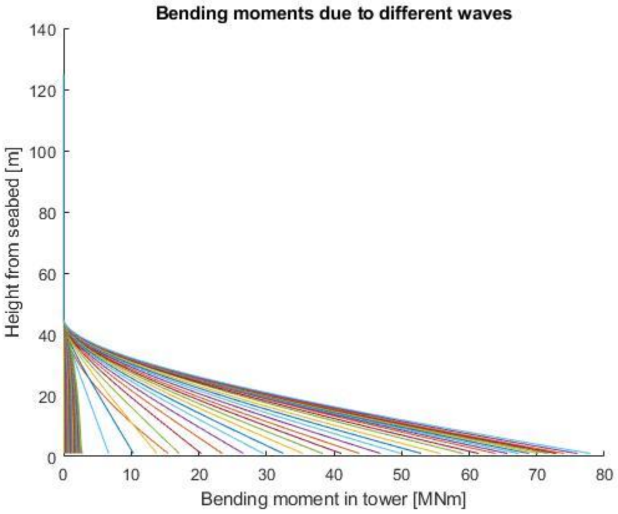

Figure 5-7: Bending moment for the different wave types

Stresses due to wave loading

After the shear force and bending moment calculations of each wave type, the maximum stress should be determined. The maximum stress of each wave type is shown in Figure 5-8 and is calculated with Equation (14).

$$\sigma_{max} = \frac{M_{max} \cdot D_{tower}}{I_{tower}} \tag{14}$$

where

σ_{max}	The maximum stress	MPa
M_{max}	The maximum bending moment	MNm
D_{tower}	Diameter of the tower	m
I_{tower}	Moment of inertia	$kg.m^2$

The moment of inertia is calculated with Equation (15).

$$I_{tower}= \frac{\pi \cdot (D_{out}^4 - D_{in}^4)}{64} \tag{15}$$

where

D_{out}	Outer diameter of the tower	m
D_{in}	Inner diameter of the tower	m

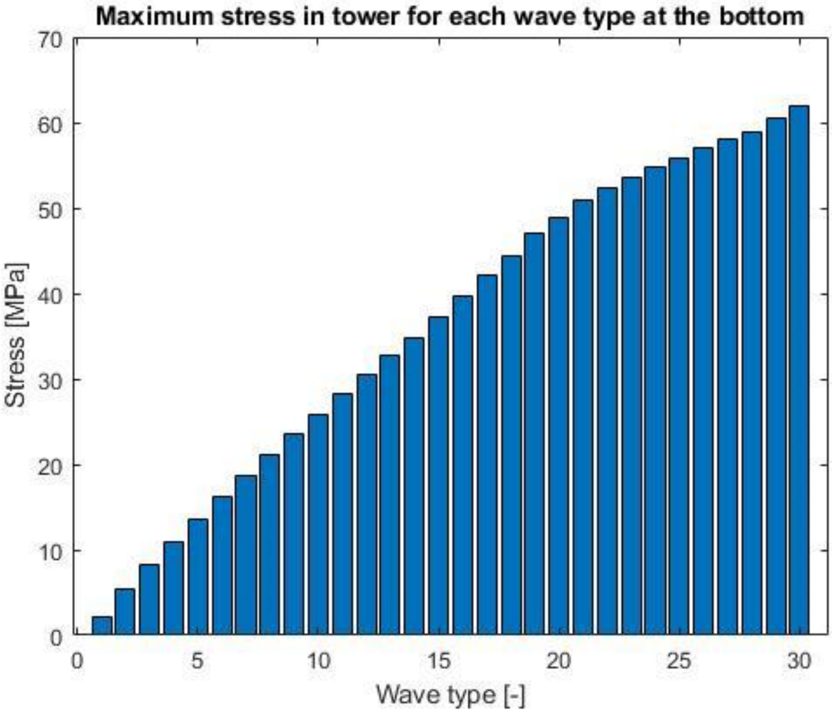

Figure 5-8: Maximum stress for different wave types

To calculate the fatigue damage, the stress cycle should be determined for each wave type from the maximum stresses. The peak counting method is used since there is only maximum stress data available. According to the peak counting method, the stress range is two times the maximum stress [53]. The stress range for each wave type is presented in Figure 5-9. The methods for determining the stress ranges are described in more detail in Appendix D.

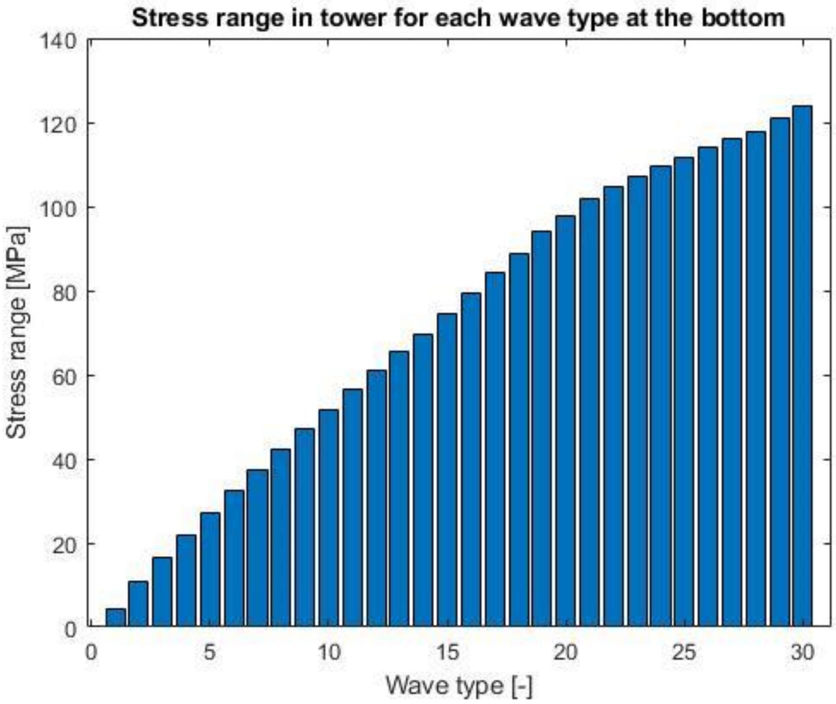

Figure 5-9: Stress range for each wave type

5.4 The dynamic behavior of the structure

A lot of fatigue in offshore monopiles is caused by resonance, especially when the turbine is not operating, and the aerodynamic damping is almost zero. Since a static approach is used for this case study, a part of the resonance can be taken into account by the dynamic amplification factor DAF. The assumptions made for the DAF calculation is that the system is considered as a single degree of freedom (SDOF) system with the first mode shape equal to the static deflection. For the DAF calculation, only the first mode is considered. In this case study, the dynamic behavior of the simplified structure is taken into account by multiplying the stress ranges by the dynamic amplification factor (DAF), as illustrated in Figure 5-10. The steps for calculating the DAF are explained in this section.

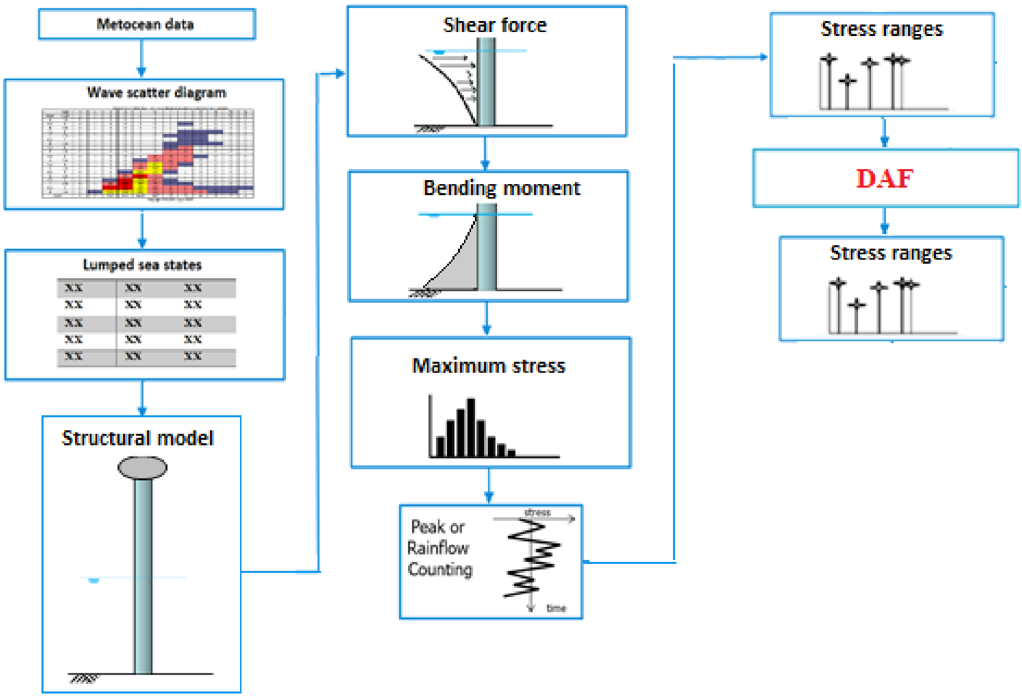

Figure 5-10: Flowchart for implementing DAF

Natural frequency

First, the natural frequency of the simplified structure needs to be determined. The only mass taken into account is the turbine mass, which is at top of the structure. Further, only the wave forces are taken into account for the calculations. Due to the assumption of having only a mass on top of the structure, the natural frequency ω_n can be calculated using Equation (16):

$$\omega_n = \sqrt{\frac{K}{M}} \tag{16}$$

where,

K = spring coefficient [N/m]
M = Mass at top of the structure [kg]

Although the mass is a given parameter since the turbine type is known, the spring coefficient K has to be calculated and this can be done according to Equation (17):

$$k = \frac{q_0}{d} \tag{17}$$

where,
q_0 = maximum value of the distributed Morison force [N]
d = deflection at the top of the structure [m]

The deflection can be calculated using the Euler beam equations. The equations are taken from Figure 5-11 to find the deflection and the inclination angle, which are combined because of the decreasing hydrodynamic loads towards the seabed. The maximum deflection is defined as v_{max} and is given in Equation (18):

$$d = \frac{11\,.\,q_0\,.\,L_b^4}{120\,.\,E\,.\,I} + L_a\,.\tan\left(\frac{q_0\,.\,L_b^3}{8\,.\,E\,.\,I}\right) \tag{18}$$

where,
L_b = length of the structure below the sea surface [m]
L_a = length of the structure above the sea surface [m]

The first term of Equation (18) is the deflection due to Morison forces up to the sea surface. This term is found by combining the first and fourth equation shown in Figure 5-11. By using the deflection formula for a distributed load over the whole beam (equation at the top of Figure 5-11) and subtracting the formula for a linearly decreasing distributed load (equation at the bottom of Figure 5-11) the formula for a linearly increasing distributed load is obtained. The deflection below the sea surface is given in Equation (19).

$$d_b = \frac{-q_0 \, . \, L_b^4}{8 \, . \, E. \, I} - \frac{-q_0 \, . \, L_b^4}{30 \, . \, E. \, I} = \frac{11 \, . \, q_0 \, . \, L_b^4}{120 \, . \, E. \, I} \qquad (19)$$

d_b = deflection below sea surface [m]

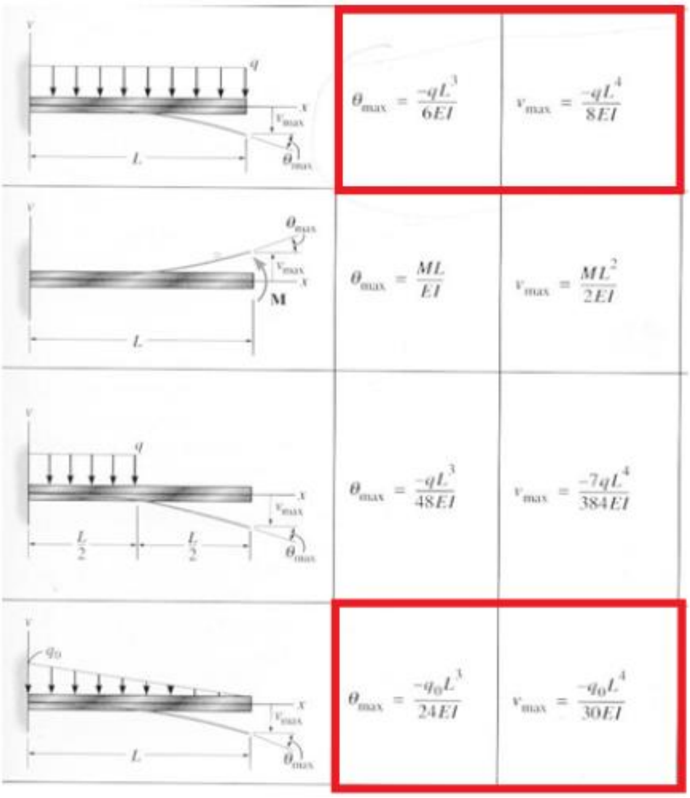

Figure 5-11: Euler-Bernoulli beam

Although this formula gives the deflection at the point of the beam where the forces stop acting, this is not the point where the beam itself ends. This issue is taken care of by the second term in Equation (18). The inclination angle is calculated in the same way as the deflection except for the fact that the formulas for the inclination angle are used, Equation (20).

$$\theta_{sea,surface} = \frac{-q_0 \,.\, L_b^3}{6 \,.\, E.\, I} - \frac{-q_0 \,.\, L_b^3}{24 \,.\, E.\, I} = \frac{q_0 \,.\, L_b^3}{8 \,.\, E.\, I} \tag{20}$$

By multiplying the tangent of this inclination angle by the length of the beam above the sea surface the extra deflection due to the length of the beam above the sea surface is calculated as follows,

$$d_a = L_a \,.\tan(\frac{q_0 \,.\, L_b^3}{8 \,.\, E \,.\, I}) \tag{21}$$

d_a = deflection above sea surface [m]

Adding both the deflection above and below water together with the formula for the total deflection at the top of the structure is obtained, Equation (22)

$$d = d_b + d_a = \frac{11 \,.\, q_0 \,.\, L_b^4}{120 \,.\, E \,.\, I} + L_a \,.\tan(\frac{q_0 \,.\, L_b^3}{8 \,.\, E \,.\, I}) \tag{22}$$

Now that the deflection is known, the spring coefficient k can be calculated. For small deflections, which is a valid assumption for this offshore structure, amplification can be made regarding the tangent term:

$\tan(x) = x \quad for\, x \ll 1$

This gives Equation (23)

$$d = \frac{11 \,.\, q_0 \,.\, L_b^4}{120 \,.\, E \,.\, I} + L_a \,.\frac{q_0 \,.\, L_b^3}{8 \,.\, E \,.\, I} \tag{23}$$

The spring stiffness is given in Equation (24)

$$k = \frac{q_0}{d} = \left(\frac{11 \,.\, q_0 \,.\, L_b^4}{120 \,.\, E \,.\, I} + L_a \,.\frac{q_0 \,.\, L_b^3}{8 \,.\, E \,.\, I}\right)^{-1} \tag{24}$$

Now that both k and M are known, Equation (16) can be used for calculating the natural frequency ω_n:

$$\omega_n = \sqrt{\frac{K}{M}} = 1.73\, rad/s$$

This natural frequency corresponds to a natural period of 3.65 seconds, which is by no means outside of the wave spectrum. In reality, however, the natural frequency would be lowered significantly by the weight of the chord and the added mass and damping of the water.

Dynamic amplification factor

Now the natural frequency is known, the DAF can be calculated with Equation (25) [54]

$$DAF = \sqrt{\left\{\left[1 - \left(\frac{\omega}{\omega_n}\right)^2\right]^2 + \left(\frac{2 \,.\zeta_t \,.\, \omega}{\omega_n}\right)^2\right\}^{-1}} \tag{25}$$

The only unknown for calculating the DAF is the total damping coefficient ζ_t of the structure. The total damping coefficient is a combination of four independent damping coefficients as given is Equation 26. The damping is higher while the turbine is operating than when the turbine is parked. The reason is that the aerodynamic damping increases while operating. A value of 4% is taken for this case study for the calculation of the DAF which is an average for a parked and operating offshore wind turbine [55].

$$\zeta_t = \zeta_s + \zeta_a + \zeta_{soil} + \zeta_h \tag{26}$$

where,

$\zeta_s = structural\ damping$
$\zeta_a = aerodynamic\ damping$
$\zeta_{soil} = soil\ damping$
$\zeta_h = hydrodynamic\ damping$

The DAF is presented in Figure 5-12, the maximum stress for the different wave types in Figure 5-13 and the stress ranges for the different wave types in Figure 5-14.

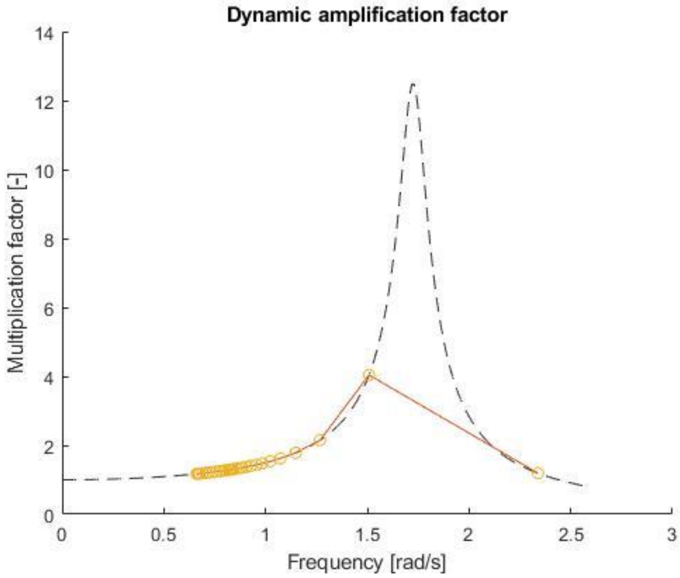

Figure 5-12: DAF with 4% damping

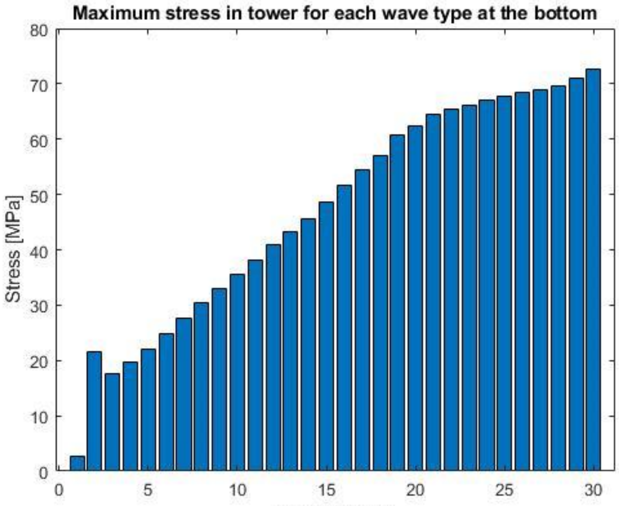

Figure 5-13: Maximum stress with DAF

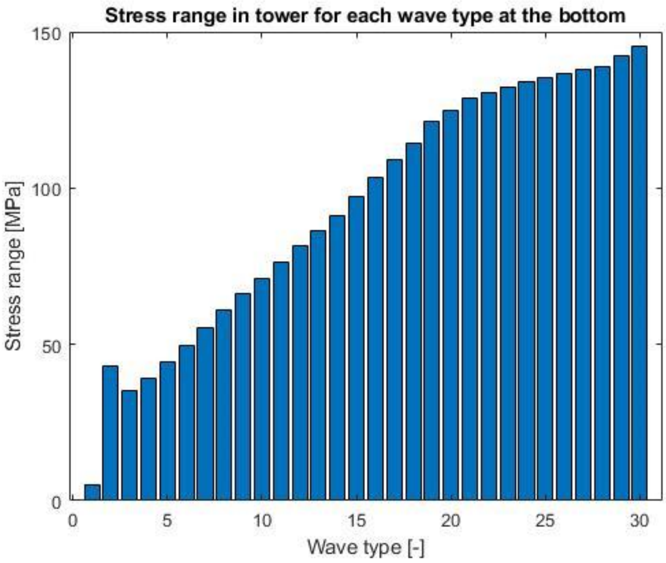

Figure 5-14: Stress ranges with DAF

5.5 Deterministic fatigue assessment

In this section, the deterministic fatigue lifetime is calculated with the initial wave scatter diagram. The steps to calculate the deterministic fatigue lifetime are presented in Figure 5-15. In order to calculate the fatigue lifetime, the stress range $\Delta\sigma$, the number of cycles n_i and the S-N curve are the necessary input. The structural model presented in section 5.2 is used to derive the maximum stress and the stress ranges at the bottom of the monopile due to the static wave loads as presented in section 5.3. The stress ranges are then multiplied by the DAF and the material factor[2] and used in conjunction with the lumped sea states of Table 9 to determine the number of cycles n_i and to compute the fatigue damage for each wave type based on the S-N curve and the Miner's sum, expressed in Equation (27).

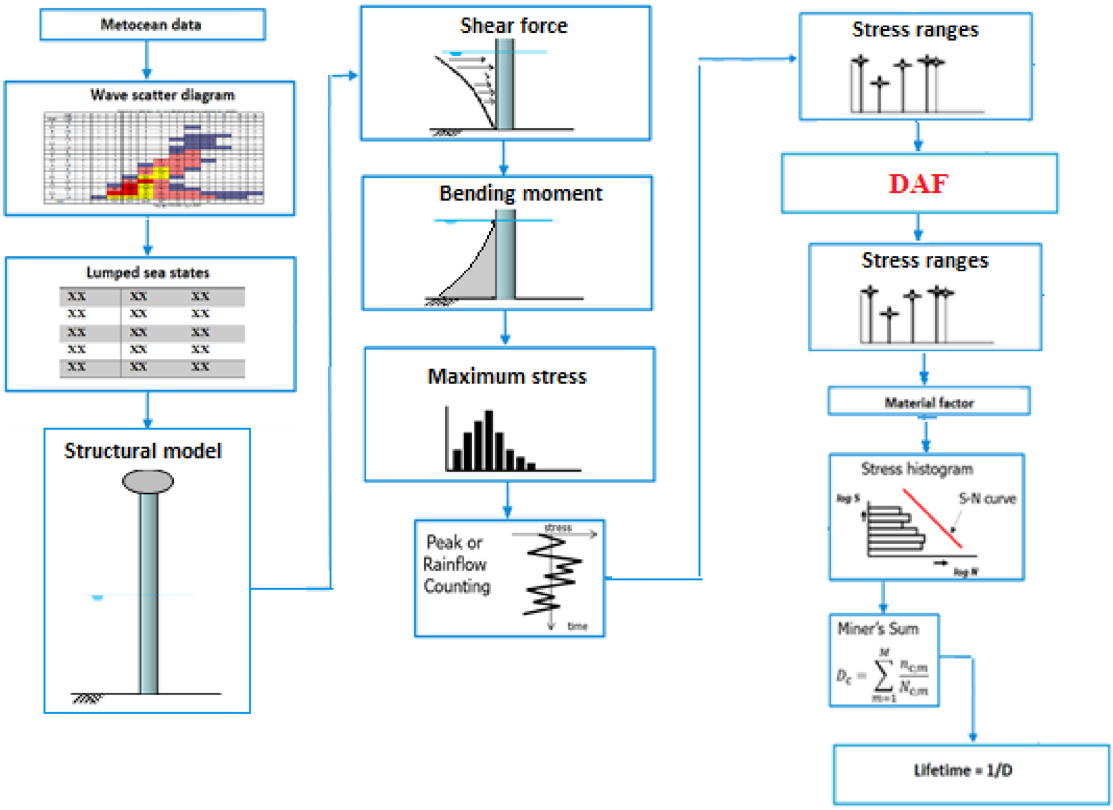

Figure 5-15: Deterministic fatigue approach

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[2] The material factor for the scour zone is used and explained in chapter 3.

$$D = \sum_{i=1}^{30} \frac{n_i}{N_{Allowable,i}} \tag{27}$$

with

D	The total damage	$[-]$
n_i	Cycles for each stress range	$[-]$
$N_{Allowable,i}$	Allowable cycles for each stress range	$[-]$

The calculated stress ranges are used to compute the number of allowable cycles using Equation (28)[3]. Subsequently, based on the lumped scatter diagram presented in Table 9, the fatigue damage for each wave type is presented in Figure 5-16.

$$N_{Allowable,i} = \frac{\bar{a}}{\Delta\sigma,_i^m}, \begin{cases} \log \bar{a} = 12.049\,; m = 3.0 & for\ N \leq 10^6 \\ \log \bar{a} = 16.081\,; m = 5.0 & for\ N > 10^6 \end{cases} \tag{28}$$

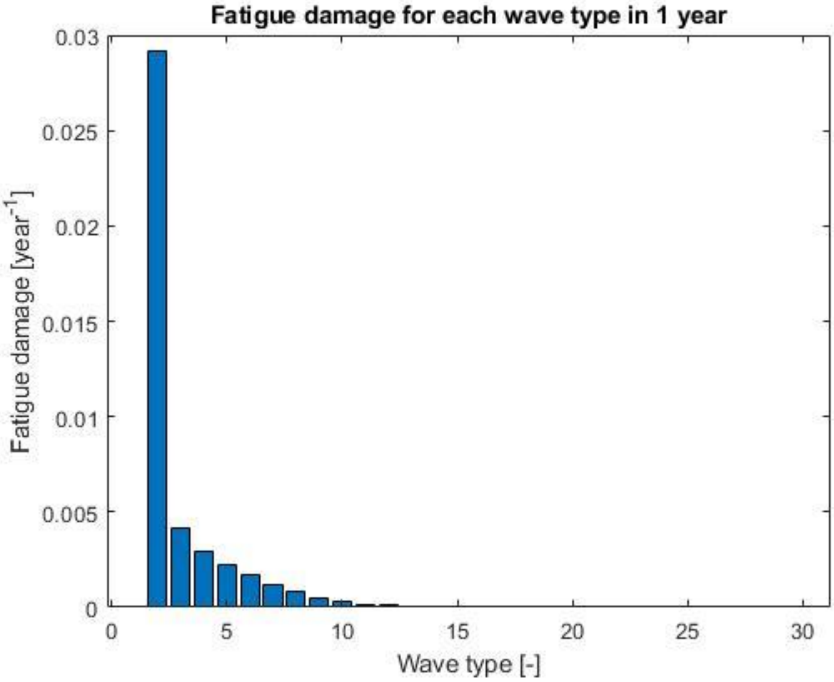

Figure 5-16: Yearly fatigue damage

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[3] The B2 S-N curve is used for this case study

The total yearly damage for the initial lumped sea states of Table 9 is equal to:

$$D_{year} = \sum_{n=1}^{30} \frac{n_i}{N_i} = 0.0434\ [-]$$

The lifetime can now be easily calculated with Equation (29) and is equal to:

$$Lifetime = \frac{1}{D_{year}} = 23\ years \tag{29}$$

5.6 Deterministic fatigue reassessment

In order to see how changes in sea state can affect changes in fatigue damage, the simplified structure will be reassessed after 15 years of operation. The reassessment is the first phase of the proposed framework of chapter 4 to determine the actual fatigue damage of the simplified structure. Since there is no updated data is available for the Gemini Wind Farm to compare it with the as-designed data for public users, the wave height of Table 9 will be changed separately by ±5% to demonstrate the effect on the fatigue damage. This can be seen as level 2 of available data, as described in chapter 4. New fatigue simulations will be run again with the new data for 5% higher wave heights and -5% lower wave heights comparing with the initial case of section 5.3. The orange dashed box in Figure 5-17 gives the region that has been changed for the new fatigue calculations. The same procedure as section 5 will be followed for both cases. It is assumed that the wave height and the wave period are not correlated which is not the right assumption in reality.

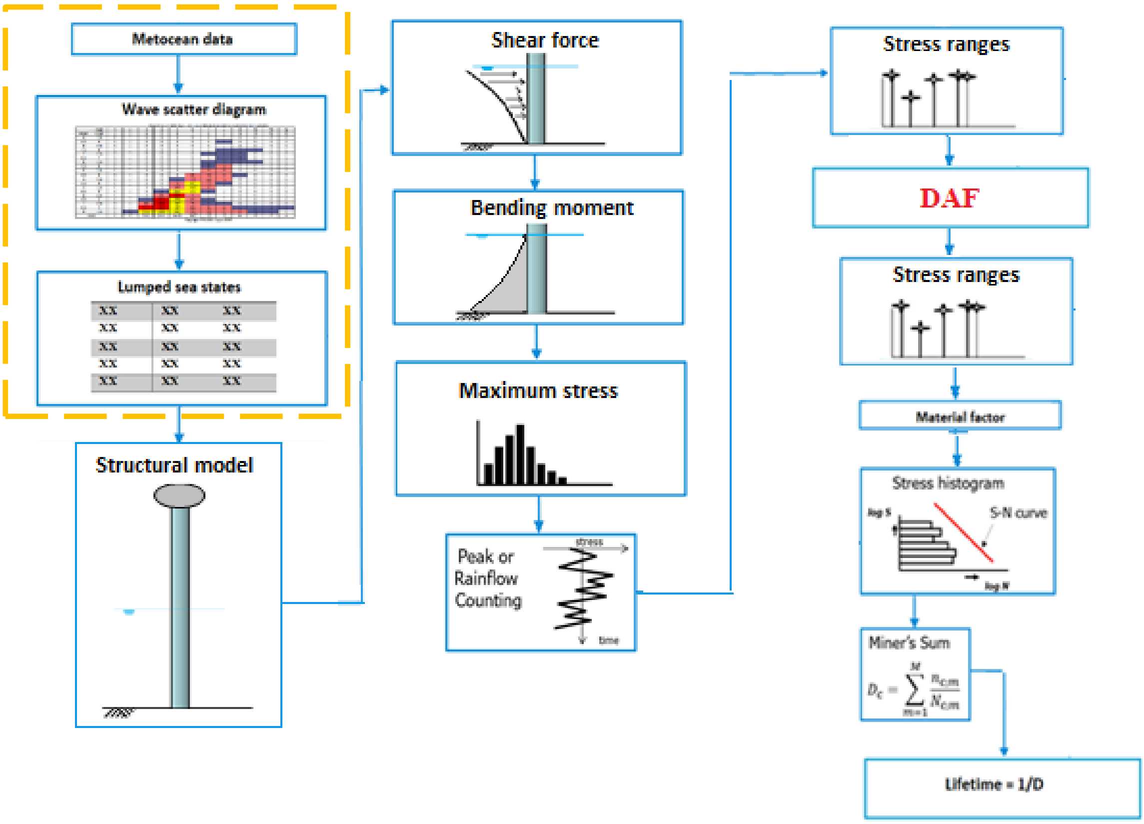


Figure 5-17: Deterministic fatigue reassessment with new data

Reassessment 1: +5% wave height

In this case, the initial wave height of table 9 increased with 5% and is used as monitored data for the previous 15 years. With this new data, new simulations are run to calculate the new fatigue damage. The maximum stress for the different wave types is presented in Figure 5-18 and the stress ranges for the different wave types in Figure 5-20. The total yearly damage for the new lumped sea states is given in Figure 5-19, and is equal to:

$$D_{year} = \sum_{n=1}^{30} \frac{n_i}{N_i} = 0.0554\ [-]$$

The new lifetime can be calculated with Equation (29) and is equal to:

$$Lifetime = \frac{1}{D_{year}} = 18\ years$$

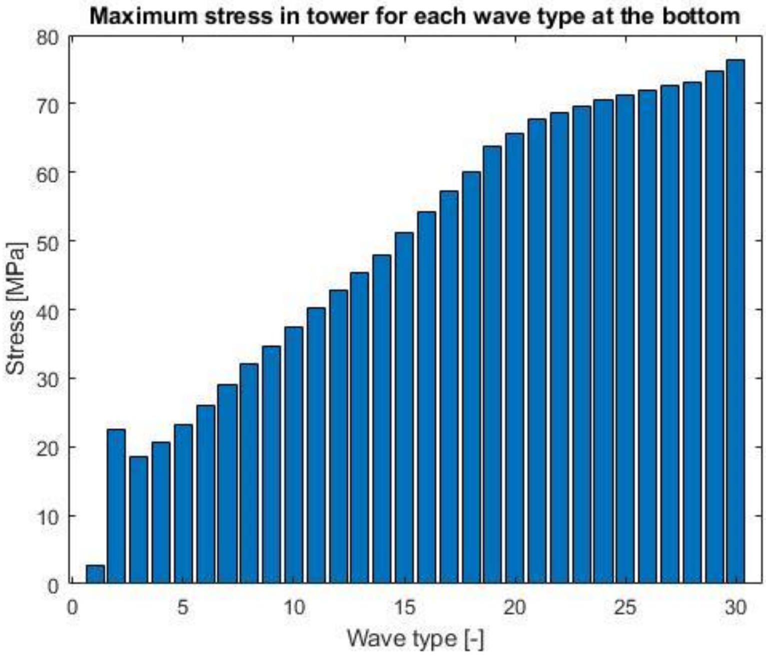


Figure 5-18: Maximum stress reassessment 1

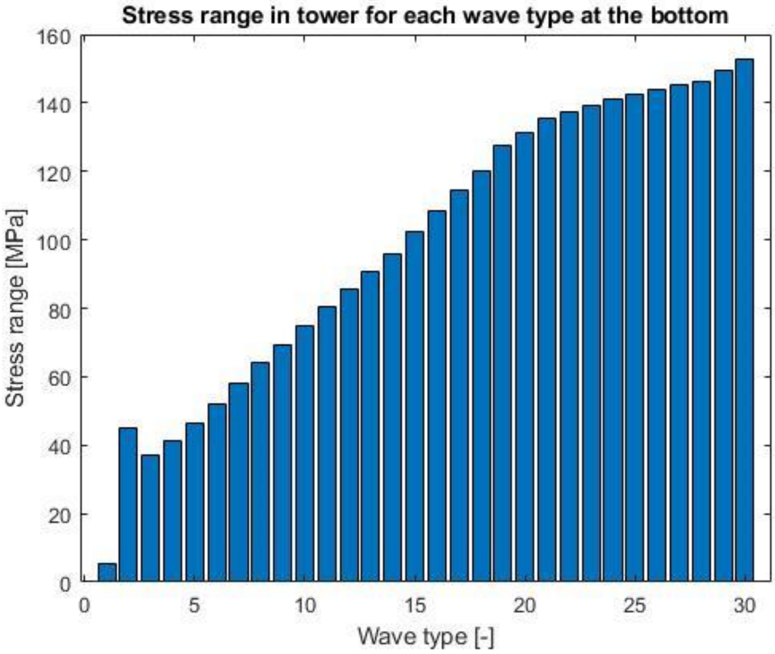


Figure 5-20: Stress range reassessment 1

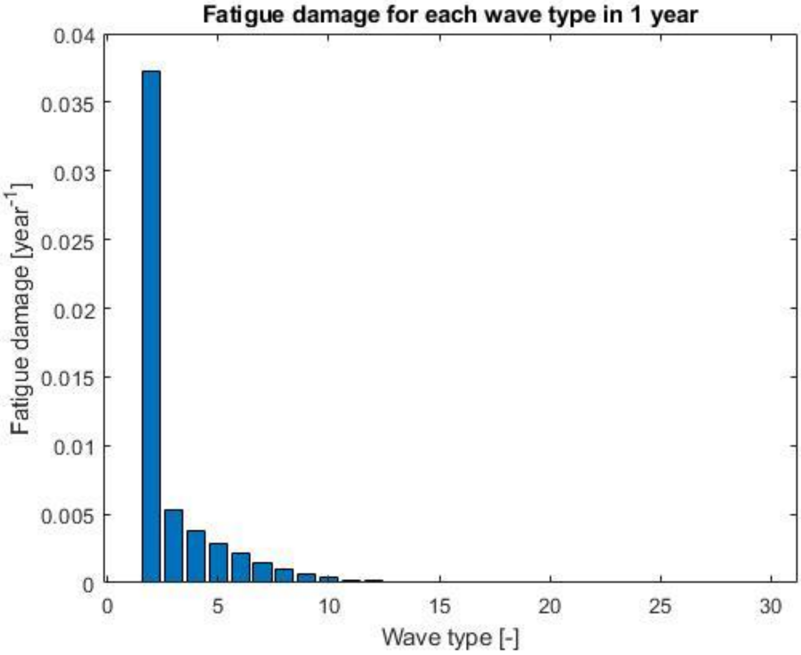


Figure 5-19: Yearly fatigue damage after reassessment 1

Reassessment 2: -5% wave height

In this case, the initial wave height of table 9 decreased with 5% and fatigue damage is calculated again. The maximum stress for the different wave types is presented in Figure 5-21 and the stress ranges for the different wave types in Figure 5-22. The total yearly damage for the new lumped sea states is given in Figure 5-23 and is equal to:

$$D_{year} = \sum_{n=1}^{30} \frac{n_i}{N_i} = 0.0336\ [-]$$

The new lifetime can be calculated with Equation (29) and is equal to:

$$Lifetime = \frac{1}{D_{year}} = 30\ years$$

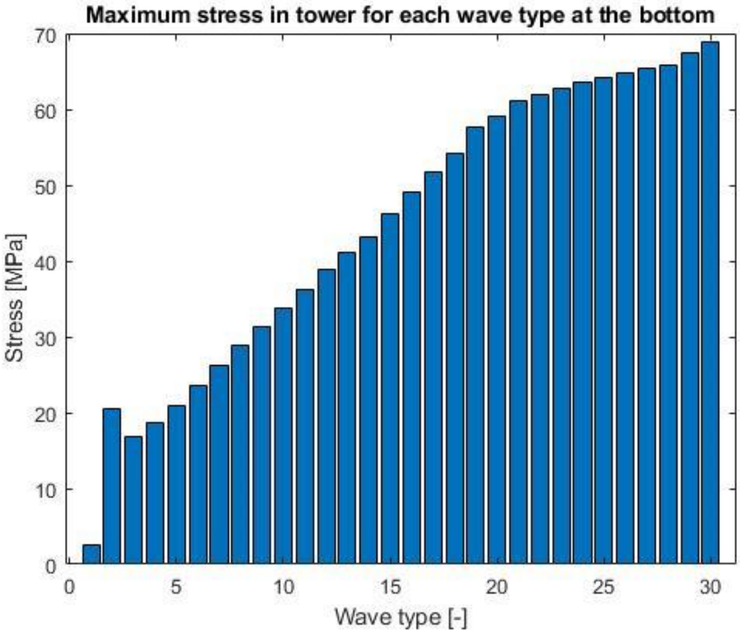


Figure 5-21: Maximum stress reassessment 2

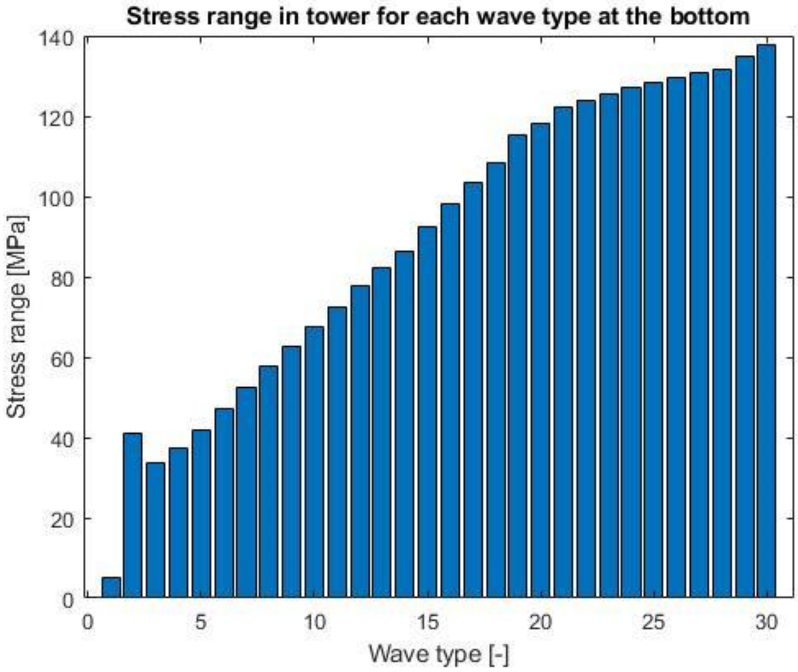


Figure 5-22: Stress range reassessment 2

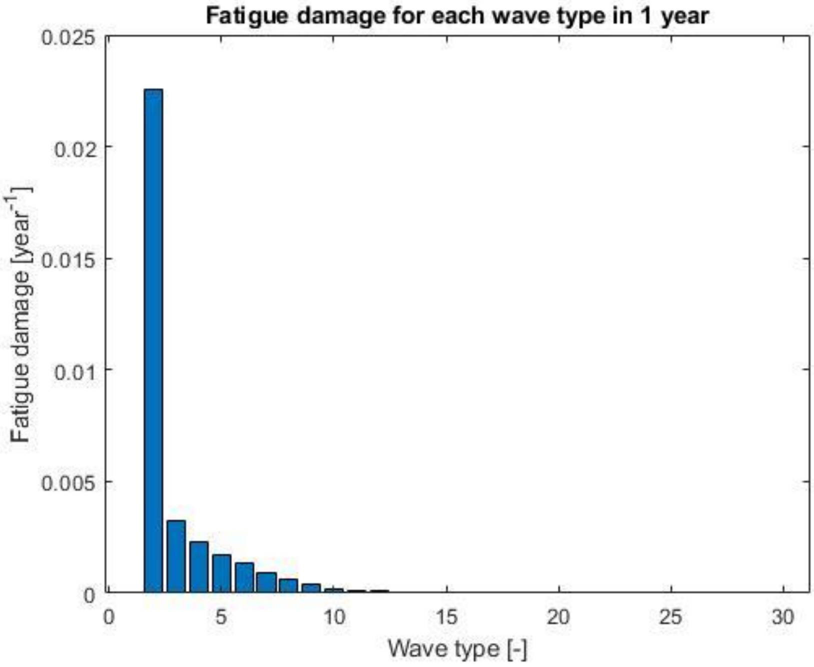


Figure 5-23: Yearly fatigue damage after reassessment 2

Figure 5-24 gives a summary of the fatigue reassessment calculations. The calculated fatigue damage for the initial scatter diagram after 15 is equal to 0.65 while the fatigue damage becomes 0.83 for +5% wave height and 0.51 for -5% wave height. From these results can be concluded that a small change in the environmental conditions has a significant influence on the fatigue lifetime. So, reassessment is essential when new data is available during the operational phase. The only task left is to determine the remaining useful life. The RUL will be studied in the next section, using a probabilistic method.

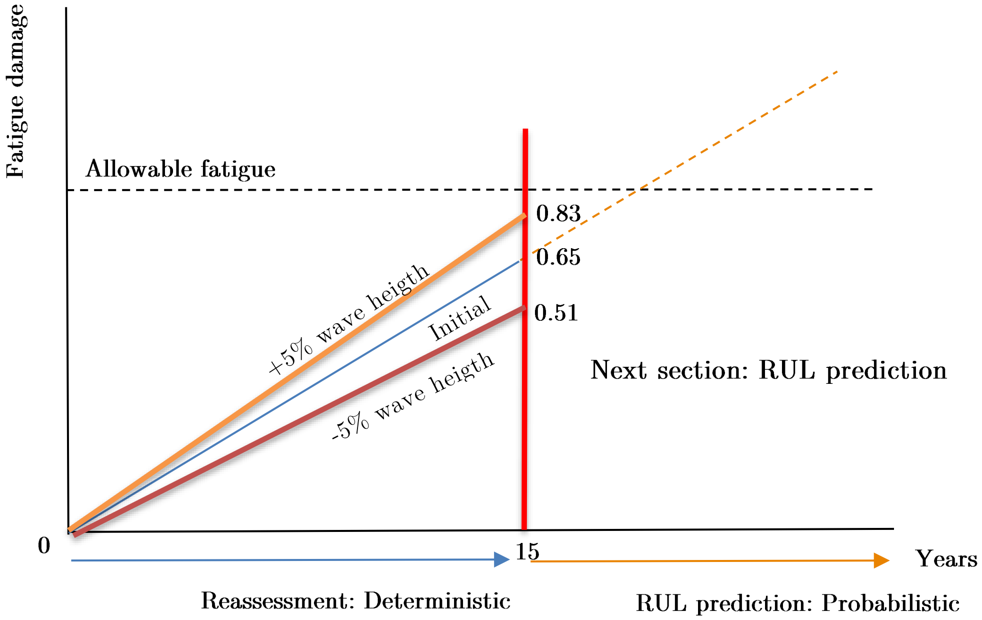


Figure 5-24: Deterministic fatigue damage after reassessment

5.7 The remaining useful life prediction

In this section, the remaining lifetime of the simplified structure will be studied using a probabilistic method. The method used in this case is the SRA as described in chapter 4. The probabilistic approach used in this case is a simplified approach. From the physics point of view, the waves are stochastic processes, and wave heights and wave periods should be stochastic parameters, which are not deterministic. Then the stresses are also stochastic parameters. However, in our numerical simulation environments, relevant simplifications have to be made to perform the analysis. For probabilistic fatigue analysis, the common way is to multiply the calculated stresses with the relevant uncertainty distributions, which is easy and time saving to operate. An overview of this method is given in Figure 5-25 and worked out in this section.

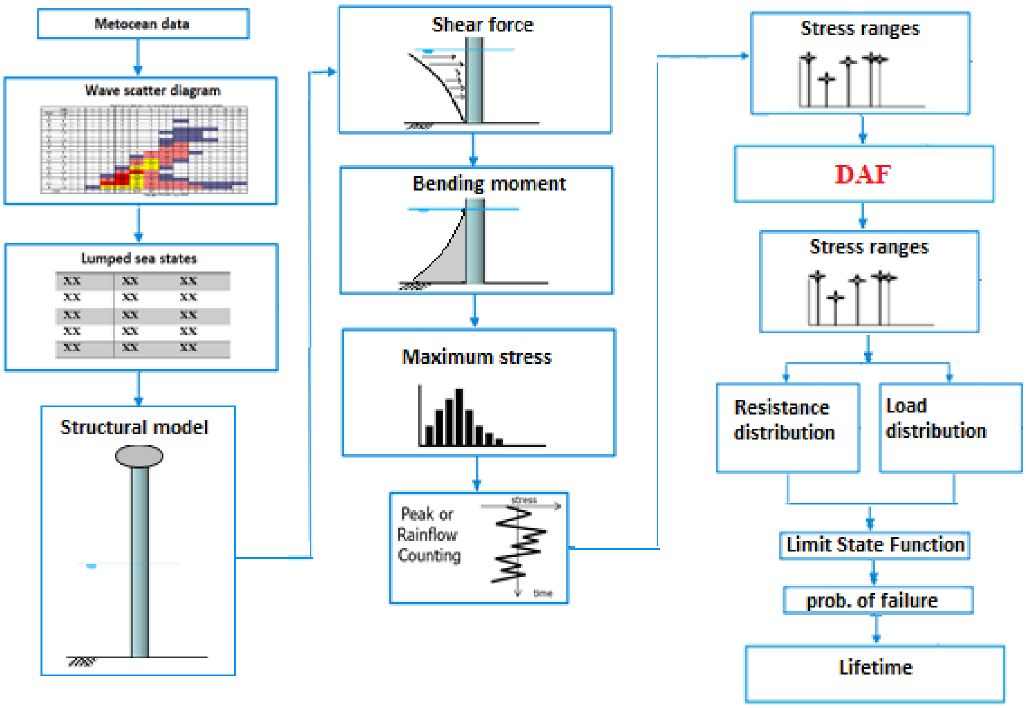


Figure 5-25: Probabilistic fatigue approach

The limit state function for this case study is defined following a few steps as demonstrated in equation 31-37.

$$G = X_{Miner's} - D \tag{30}$$

where

G	Fatigue limit state function
$X_{Miner's}$	Resistance → Miner's rule
D	Load → fatigue damage

$$D = \sum_{i=1}^{30} \frac{n_i}{N_{Allowable,i}} \tag{31}$$

$$\log N_{Allowable,i} = \log \bar{a} - m \log \Delta\sigma \tag{32}$$

$$N_{Allowable,i} = \frac{\bar{a}}{{\Delta\sigma_i}^m} \tag{33}$$

$$D = \frac{1}{\bar{a}} \sum n_{i\,x}\ \Delta\sigma_i^m \tag{34}$$

The stress range $\Delta\sigma_i$ is now multiplied by the uncertainty distributions, instead of the material factor as in the deterministic case. This is shown in Equation (35).

$$\Delta\sigma_i = \chi_1 \chi_2 \ldots \chi_n \cdot \Delta\sigma_{0,i} \tag{35}$$

Finally, the limit state function used for this case study is given in Equation (36).

$$G = X_{miner's} - \frac{{\chi_1}^m {\chi_2}^m \ldots\ldots {\chi_n}^m}{\bar{a}} \sum n_i . \Delta\sigma_{0,i}^m \tag{36}$$

The parameters used for the limit state function are listed in Table 10. The green color indicates the uncertainties at the resistance side; orange indicates the uncertainties at the load side and grey are the deterministic parameters. The plots of the distributions are presented in Appendix F.

Table 10: Probabilistic calculation parameters

Variable	Distribution	Mean	Standard deviation	Comments
$X_{Miner's}$	Log normal	1	0.3	DNV GL RP C210
χ_{load}	Log normal	1	0.1	To account for the model uncertainties related to the assessment of aerodynamic loads and hydrodynamic loads. DNVGL inter
$\chi_{peak\ counting}$	Normal	1	0.1	Uncertainty of peak counting method
$\chi_{lowcycle}$	Normal	1	**0.03**	Uncertainty of low cycle fatigue. DNVGL intern
$\bar{a}$	Deterministic	-	-	Same as deterministic approach
n_i	Deterministic	-	-	Same as deterministic approach
$\Delta\sigma_{R0,i}$	Deterministic	-	-	Same as deterministic approach

The limit state function is solved with the first order reliability method (FORM), with the software program Prob2b. First, the cumulative probability of failure $P_{f,cummulative\_year\_n}$ is calculated for the simplified structure and from the cumulative probability of failure the annual probability of failure is calculated with Equation (37). The end of the lifetime is reached when the annual probability of failure is equal to the target reliability of 10^{-4}. A summary of the results for the 3 cases are presented in Figure 5-26, Figure 5-27, Figure 5-28, Table 11, Table 12, and Table 13. The probabilistic calculations are from the start of the operating phase and the end of life is when the annual probability of failure reaches the target reliability. In this case, the starting point should be shifted since the reassessment after 15 years is done by a deterministic method. Combining the deterministic fatigue damage and the probabilistic fatigue damage will be explained in more detail.

$$P_{f,annual\_n} = P_{f,cummulative\_year\_n} - P_{f,cummulative\_year\_n\,-1} \qquad (37)$$

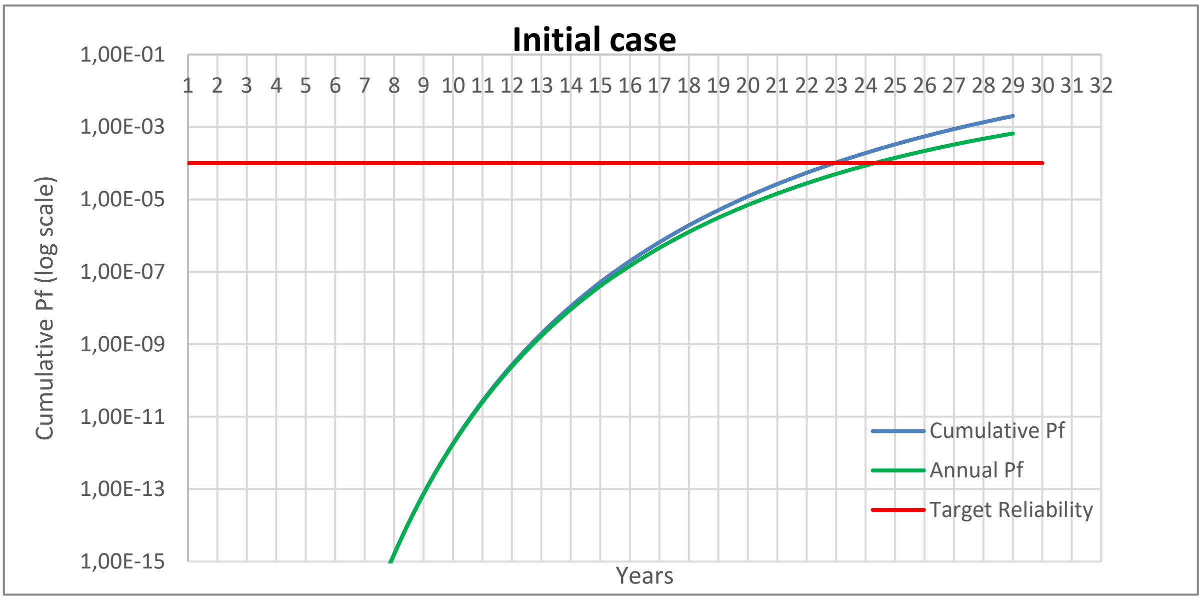


Figure 5-26: Probabilistic calculations results of the initial case

Table 11: Probabilistic calculation results of the initial case

Initial case		
Year	**Cumulative prob. of failure**	**yearly prob. of failure**
1	4,74E-71	4,74E-71
2	2,11E-46	2,11E-46
3	4,96E-35	4,96E-35
4	3,36E-28	3,36E-28
5	1,47E-23	1,47E-23
6	3,58E-20	3,58E-20
7	1,41E-17	1,40E-17
8	1,60E-15	1,59E-15
9	7,53E-14	7,37E-14
10	1,84E-12	1,76E-12
11	2,72E-11	2,53E-11
12	2,71E-10	2,44E-10
13	1,98E-09	1,71E-09
14	1,12E-08	9,19E-09
15	5,18E-08	4,06E-08
16	1,97E-07	1,45E-07
17	6,53E-07	4,56E-07
18	1,91E-06	1,26E-06
19	5,01E-06	3,10E-06
20	1,20E-05	6,96E-06
21	2,64E-05	1,44E-05
22	5,42E-05	2,78E-05
23	1,04E-04	5,02E-05
24	1,90E-04	8,59E-05
25	3,30E-04	1,40E-04
26	5,48E-04	2,18E-04
27	8,73E-04	3,26E-04
28	1,35E-03	4,72E-04
29	2,00E-03	6,59E-04
30	2,90E-03	

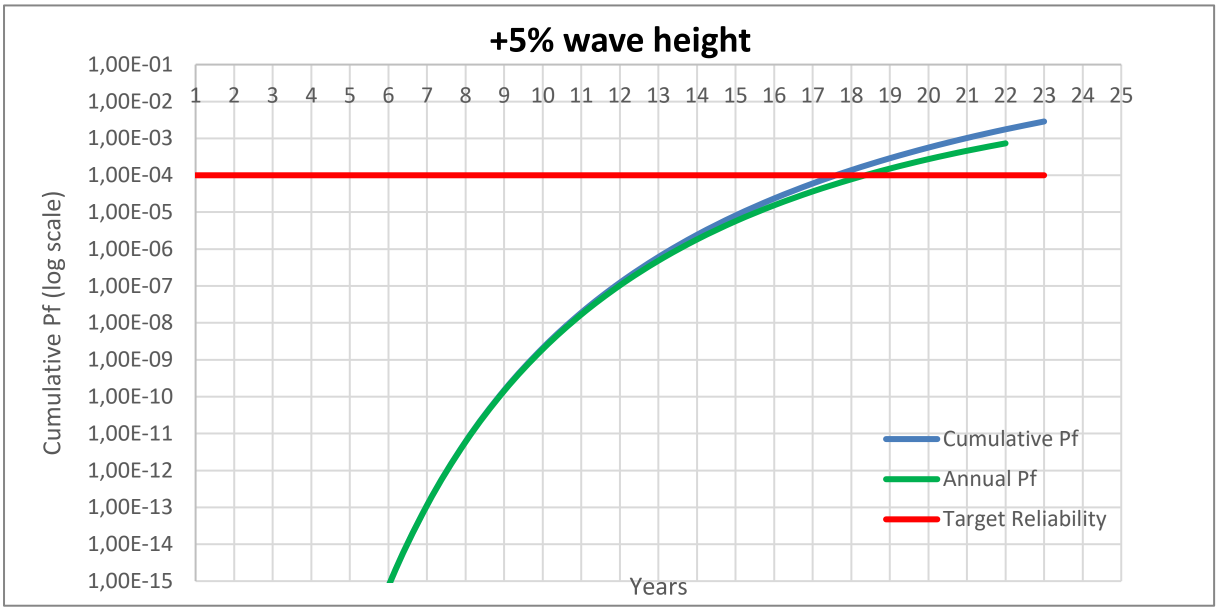


Figure 5-27: Probabilistic calculations results +5% wave height

Table 12: Probabilistic calculation results +5% wave height

+5% wave height		
Year	**Cumulative prob. of failure**	**yearly prob. of failure**
1	7,95E-61	
2	9,58E-39	7,95E-61
3	1,03E-28	9,58E-39
4	9,25E-23	1,03E-28
5	9,27E-19	9,25E-23
6	7,25E-16	9,27E-19
7	1,14E-13	7,24E-16
8	6,05E-12	1,13E-13
9	1,49E-10	5,94E-12
10	2,08E-09	1,43E-10
11	1,89E-08	1,93E-09
12	1,22E-07	1,68E-08
13	6,07E-07	1,04E-07
14	2,43E-06	4,85E-07
15	8,12E-06	1,82E-06
16	2,35E-05	5,70E-06
17	5,99E-05	1,53E-05
18	1,38E-04	3,64E-05
19	2,89E-04	7,77E-05
20	5,63E-04	1,52E-04
21	1,02E-03	2,74E-04
22	1,76E-03	4,61E-04
23	2,87E-03	7,33E-04

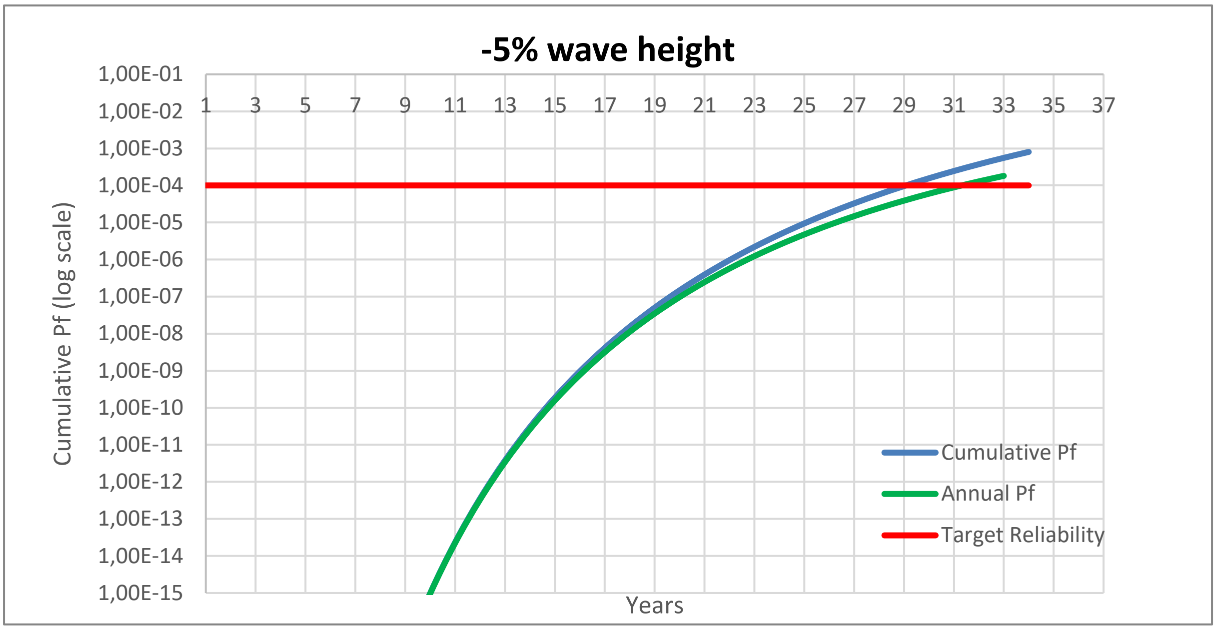


Figure 5-28: Probabilistic calculations results -5% wave height

Table 13: Probabilistic calculation results -5% wave height

Wave height -5%		
Year	**Cumulative prob. of failure**	**yearly prob. of failure**
1	3,90E-81	3,90E-81
2	4,53E-54	4,53E-54
3	1,98E-41	1,98E-41
4	9,18E-34	9,18E-34
5	1,63E-28	1,63E-28
6	1,19E-24	1,19E-24
7	1,13E-21	1,13E-21
8	2,68E-19	2,67E-19
9	2,35E-17	2,33E-17
10	9,87E-16	9,63E-16
11	2,35E-14	2,25E-14
12	3,57E-13	3,33E-13
13	3,79E-12	3,44E-12
14	3,01E-11	2,63E-11
15	1,88E-10	1,58E-10
16	9,59E-10	7,71E-10
17	4,12E-09	3,16E-09
18	1,53E-08	1,12E-08
19	5,02E-08	3,48E-08
20	1,47E-07	9,72E-08
21	3,93E-07	2,46E-07
22	9,65E-07	5,72E-07
23	2,20E-06	1,23E-06
24	4,70E-06	2,50E-06
25	9,45E-06	4,76E-06
26	1,81E-05	8,60E-06
27	3,29E-05	1,48E-05
28	5,74E-05	2,45E-05
29	9,63E-05	3,90E-05
30	1,56E-04	5,99E-05
31	2,45E-04	8,90E-05
32	3,74E-04	1,29E-04
33	5,55E-04	1,81E-04
34	8,04E-04	

The remaining useful life for the case of +5% higher wave height and -5% lower wave height can now be determined following the approach of section 4.6.

Remaining useful life prediction 1

In this part, the RUL will be predicted for the first reassessment case, namely the +5% wave height. The deterministic fatigue damage is equal to:

$$D_{year} = \sum_{n=1}^{30} \frac{n_i}{N_i} = 0.0554\ [-]$$

For this calculation, a material safety factor of 1.25 is used, which is equal to a DFF of 3 according to table 2 of chapter 3. The fatigue damage without any safety factors would be:

$$D_{year\_no\_safety} = \frac{D_{year}}{DFF} = 0.0185$$

The deterministic fatigue damage after 15 years without safety factors would be:

$$D_{15\,years} = D_{year\_no\_safety} * 15 = 0.277$$

The get the same target reliability level for the deterministic and the probabilistic method, the deterministic fatigue damage should be divided by DFF factor according to Figure 4-14, which is 9 if the annual target reliability is 10^{-4}.

$$D_{15\,years\_} = \frac{D_{15\,years}}{9} = 0.031$$

Damage of 0.031 gives a cumulative probability of failure of $6.0*10^{-7}$ as shown in Figure 5-29 with an orange line. Now Table 12 can be used, which is made from the probabilistic calculations. A cumulative probability of failure of $6.0*10^{-7}$ is approximately equal to 13 years according to table 12. Using the same table 12 and counting further from a cumulative probability of failure of $6.0*10^{-7}$ to annual target reliability of failure of 10^{-4} gives a remaining lifetime of approximately 7 years. The total lifetime becomes now 20 years, which is approximately 2 years more than the deterministic lifetime calculation.

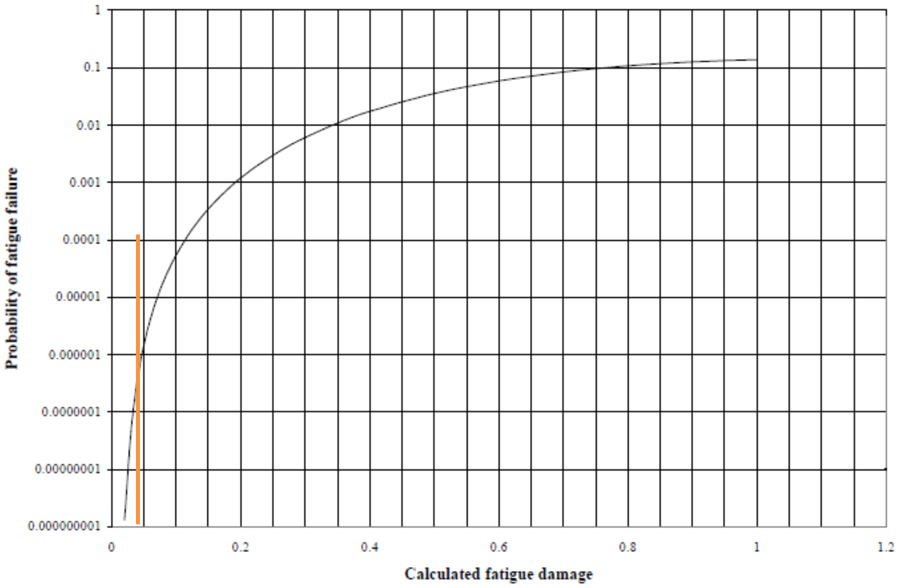


Figure 5-29: Translation +5% case

Remaining useful life prediction 2

The same approach is used for the RUL prediction for the second reassessment case, namely the -5% wave height. The deterministic fatigue damage is equal to:

$$D_{year} = \sum_{n=1}^{30} \frac{n_i}{N_i} = 0.0336\ [-]$$

For this calculation, a material safety factor of 1.25 is used, which is equal to a DFF of 3 according to table 2 of chapter 3. The fatigue damage without any safety factors would be:

$$D_{year\_no\_safety} = \frac{D_{year}}{DFF} = 0.0112$$

The deterministic fatigue damage after 15 years without safety factors would be:

$$D_{15\,years} = D_{year\_no\_safety} * 15 = 0.168$$

The get the same target reliability level for the deterministic and the probabilistic method, the deterministic fatigue damage should be divided by DFF factor according to Figure 4-14, which is 9 if the annual target reliability is 10^{-4}.

$$D_{15\,years\_} = \frac{D_{15\,years}}{9} = 0.019$$

Damage of 0.019 gives a cumulative probability of failure below $1.0*10^{-11}$ as shown in Figure 5-30 with an orange line. Using a cumulative probability of failure of $1.0*10^{-11}$ and Table 13, the RUL can be predicted. Counting further from a cumulative probability of failure of $1.0*10^{-11}$ of Table 13 to annual target reliability of failure of 10^{-4} in Table 13 gives a remaining lifetime of approximately 18 years. The total lifetime becomes now around 33 years, which is around 3 years more than the deterministic lifetime calculation.

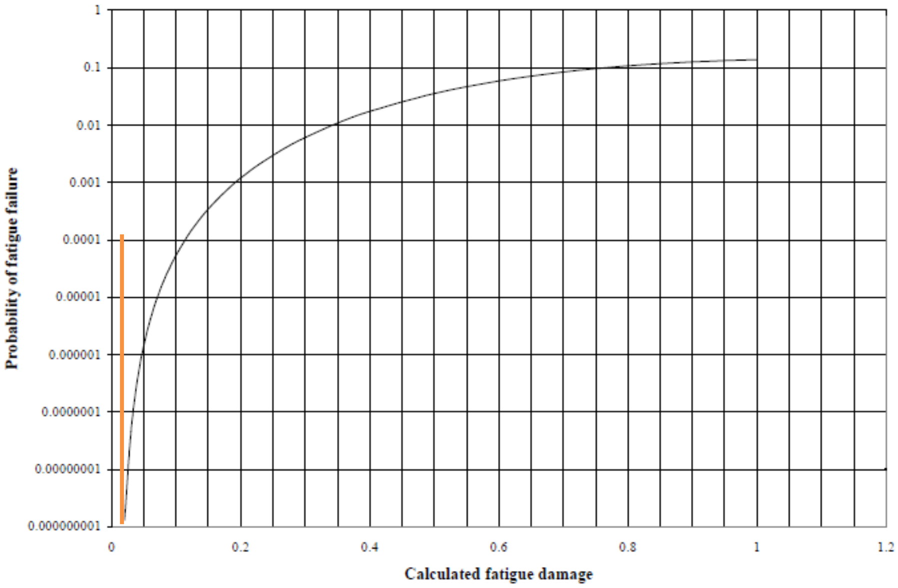


Figure 5-30: Translation -5% case

Chapter 6

6 Conclusions and Recommendations

The main objective of this thesis study is to investigate the operating life of existing offshore wind turbine support structures. For this work, a framework has been presented, consisting of a reassessment phase and a remaining useful life prediction phase. For both phases, different methods are suggested depending on the available data of the offshore wind turbine. The suggested framework is subsequently demonstrated in a simplified case study.

6.1 Conclusions

6.1.1.Literature review

The literature survey has introduced the reader to the existing procedures for reassessing offshore structures in general. Reassessment approaches by the design standards and other researcher are presented and the essential information is used as a starting point for this study.

It is concluded that reassessment is essential if there are changes in environmental conditions, if degradation rate is higher than the as-designed for rate, if there is a change of a component of the structure, and if exceedance of the design life takes place. The available data and information about the environment conditions and the structural conditions during the different phases are essential for a meaningful reassessment and remaining useful life prediction. It can also be concluded that there is not a clear framework and methodology describing step by step the reassessment and the remaining useful life prediction of offshore wind turbine support structures.

6.1.2. Fatigue analysis

In chapter 3, the fatigue calculation process is explained for the offshore wind turbine support structure. The general fatigue calculation approach is presented to the reader including the deterministic approach by applying partial load and resistance factors according to the design standard and the probabilistic approach by replacing the partial load and resistance factors to the uncertainty distributions.

It is concluded that in the real world, the wave and wind are stochastic processes, so the wave and wind parameters should also be stochastic parameters, which are not deterministic. Then the stresses are also stochastic parameters. That makes the probabilistic fatigue approach a complex process and that relevant simplification in the numerical simulations have to be made to perform the analysis. An allowable approach is to calculate the deterministic stress response and multiply it by the uncertainty distributions.

6.1.3. Assessment of existing offshore wind turbines

In chapter 4, a framework is suggested with different methods to reassess existing offshore wind turbine support structures and to predict the remaining useful life. The following conclusions can be drawn:

- A framework consisting of two phases is an useful approach for reassessing offshore wind turbine support structures. The first phase is a reassessment phase to determine the current health of the offshore wind turbine. The reassessment is essential since environmental and operational assumptions are made in the design phase which may not match the reality. The second phase is the remaining useful life prediction phase.
- For each phase of the framework, different methods can be used. The available data and information determine which method can be used. That makes monitoring of the OWT essential for the reassessment to obtain external and operational data.
- The available data and information can be divided into three levels. The methods belonging to level 3 gives the most accurate results, but only a small percentage of the existing wind turbines have a level 3 data pack.

6.1.4. Case study

In chapter 5, a simplified case study is studied to demonstrate the proposed framework. The following conclusions can be drawn:

- The proposed framework is easy to implement and gives useful results.
- Lumping the wave scatter diagram gives high uncertainties in the calculation process.
- From the case study, it can be concluded that the reassessment is essential since a small change in wave height leads to significant deviations on the lifetime.
- The availability of the offshore wind turbine is relevant data that should be taken into account for the reassessment. While the turbine is operating (availability) the aerodynamic damping increases which causes less damage.
- The probabilistic fatigue calculations method for offshore wind turbine support structures is a complex method, especially in defining the fatigue limit state.
- For the probabilistic calculations, the partial load and resistance factors are replaced by uncertainty distributions. That results in different lifetimes for the same structure. The structural reliability analysis (SRA) is the most promising probabilistic approach.
- The deterministic fatigue results and the probabilistic fatigue results can be combined which makes it possible to use a combination of deterministic and probabilistic methods.

6.2 Recommendations

There are some topics related to this thesis that requires further investigation. The recommendations for further work are listed in this section.

- Lumping the scatter diagram gives significant uncertainties. It is recommended to investigate the scatter lumping process to find out which method provides with the least uncertainty.
- Not all the offshore wind turbines in the wind farm are reaching the end of life at the same time. This problem should be studied in more detail so that decommissioning of the whole wind farm takes place at the same time.
- The S-N curves have different regions as described in chapter 3. Stress ranges below the cut-off limit are not considered. The effect of the cut-off limit on the fatigue damage should be studied since stress ranges below the cut-off could be important if stress ranges above the cut-off limit have occurred earlier.

- An additional aspect that should be added and investigated for the probabilistic calculations is to use Bayesian techniques for updating the probability of failure.
- Fracture mechanics is another method to predict the remaining lifetime. This method should be investigated to determine when it can be applied and how it can be combined with the deterministic methods.
- The partial safety factors given in the design standards are obtained by probabilistic analysis. It is recommended to investigate the partial safety factor for a wind farm with a level 1 probabilistic method to make a comparison with the given values in the design standards.
- More research is required on combining the probabilistic and the deterministic methods to validate the described approach in this thesis.

Appendices

Appendix A

A Uncertainties

Uncertainties in the offshore wind turbine design

Sudret [56], underlines the important distinction between a designed system on the one hand, and a real-life system on the other hand:

> *"A designed system is an abstract object whose model and associated input parameters have been selected in such a way that design criteria (related with the purpose of the system) are fulfilled. In contrast, a real system never matches the initial design, at least for the following reasons:*
>
> - *The true dimensions of the real system do not correspond exactly with the design due to imperfections in the manufacturing process;*
> - *The material properties of the real system may differ slightly from the codified properties of the class of material it is supposed to be made of;*
> - *The loading (resp. boundary conditions) of the designed system is idealized so that they roughly represent the complexity of the real system.*

Various types of uncertainty exist, generally classified as either:

- **Aleatoric** uncertainty

  The aleatoric uncertainties are defined as irreducible uncertainties which are due to the inherently random nature of an underlying physical process and cannot be controlled. The environmental conditions are a good example of aleatoric uncertainties.
- **Epistemic** uncertainty

  This type of uncertainties refers to a lack of knowledge on the part of the designer and is therefore in principle reducible with the application of better, or more complete, knowledge of a given system. Uncertainties of this type can be due to limited data, human errors or model uncertainties.
- **Mixed** uncertainty

  There is no clear distinction between aleatoric and epistemic sources of uncertainty for this type of uncertainties. An example of a mixed uncertainty is material strength which will always be somewhat uncertain, but that uncertainty can be reduced by testing and quality control.

The limit states used in a wind turbine design contain both loading- and resistance-related variables. In order to consider what uncertainties may reside within the limit state analysis for a wind turbine design, it is instructive to consider the participation of the uncertainty sources. A simple representation of the uncertainties sources is provided in Table 14 for an offshore wind turbine.

Table 14: Uncertainty sources in an offshore wind turbine

External conditions	Applied loads	Internal loads	Local stresses	Material properties
Aerodynamics - Turbulence spectrum - Mean wind speed & direction - Wind shear - Air density - Extreme gusts Hydrodynamics - Significant wave height - Mean zero-up crossing period - Wave period Current - Current speed Soil parameters - Internal angle of friction - Relative density - Weight of overburden - Cohesion	- Blade loads - Tower and nacelle forces - Time-varing gravity loads on the blades - Centrifugal forces and Coriolis forces - Gyroscopic forces due to yawing - Braking forces - Morison's equation - Soil resistance force	- Structural dynamics - Damping - Actuator models - Electrical sytems - Control sytems - Seed effects - Azimuthal effects - Transient events - Cut-in and cut-out	- Geometry - SCF	- Variable load amplitudes - Miner's rule - S-N curve - Thickness correction - Buckling - Yield - Stiffness - Material and fabrication imperfections

Appendix B

B Methods for continued operation according DNVGL

Table 15: Methods for lifetime extension assessment according to DNVGL [19]

Method	*Scope*	*Main outcome*	
Lifetime extension inspection (LEI)	— visual inspection of all load-transferring and safety-relevant components — review of maintenance reports and inspection reports for specific turbine — consideration of SCADA data — consideration of wind turbine type related field experience — simple tests	evaluation, if turbine is suitable for lifetime extension	
Simplified approach for lifetime extension	Analytical part: — load calculation, may be performed using generic turbine model — calculation of possible extension of lifetime based on environmental conditions as per original design vs. environmental conditions at the site — possibly accompanied by load measurements Practical part: — inspection based on general inspection plan — visual inspection of all load-transferring and safety-relevant components — review of maintenance reports and inspection reports for specific turbine — consideration of SCADA data — consideration of wind turbine type related field experience — performance of tests	— specification of possible lifetime extension — specification of required inspection scope and intervals based on inspection results and results analytical part — specification of restrictions/conditions (.e.g. component exchange, installation of CMS, etc.) if required	Proof of strength and stability
Detailed approach for lifetime extension	Analytical part: — load calculation based on specific turbine model — calculation of possible extension of lifetime based on environmental conditions as per original design vs. site specific environmental conditions and utilization rate of components — reserve calculations on load-transferring components — possibly accompanied by load measurements — possibly optimization of control system — consideration of turbine type related field experience — development of turbine-specific inspection plan Practical part: — inspection as per turbine-specific inspection plan that has been developed in the analytical part — visual inspection of all load-transferring and safety-relevant components — review of maintenance reports and inspection reports for specific turbine — consideration of SCADA data — consideration of wind turbine type related field experience — performance of tests	— specification of possible lifetime extension — specification of required inspection scope and intervals based on inspection results and results analytical part — specification of restrictions/conditions (.e.g. component exchange, installation of CMS, etc.) if required	
Probabilistic approach for lifetime extension	Analytical part: — structural reliability analysis (stochastic approach) — calculations based on generic or specific turbine model — selection of reliability levels — identification of failure modes — possibly accompanied by load measurements — possibly optimization of control system — consideration of turbine type related field experience — development of turbine or site-specific inspection plan Practical part: — inspection as per turbine-specific inspection plan that has been developed in the analytical part — visual inspection of all load-transferring and safety-relevant components — review of maintenance reports and inspection reports for specific turbine — consideration of SCADA data — consideration of wind turbine type related field experience — performance of tests	— specification of possible lifetime extension respectively reliability level — specification of required inspection scope and intervals based on inspection results and results analytical part — specification of restrictions/conditions (.e.g. component exchange, installation of CMS, etc.) if required	Proof of strength and stability

Appendix C

C Loads and load cases

In this part, the loads and the design principles of an OWT will be explained, starting with the loads that are acting on the OWT, the combined load cases, the design limit states and finally the uncertainties in the design. Since fatigue is an accumulation of damage as a function of time, it becomes an essential limit state that should be checked when looking to lifetime extension of OWTs [19].

Loads acting on offshore wind turbines

Wind turbines in general and offshore wind turbines in particular are structures that experience many loads. The various load sources, acting on the OWTs have a big influence on the design and the operating lifetime. Therefore, the expected loads and load combinations should be chosen carefully for a successful design to withstand all the possible loads and load combinations during the operating life. The loads that are working on wind turbines can be divided into three categories, as presented in Figure C-0-1 [8] [34] [57].

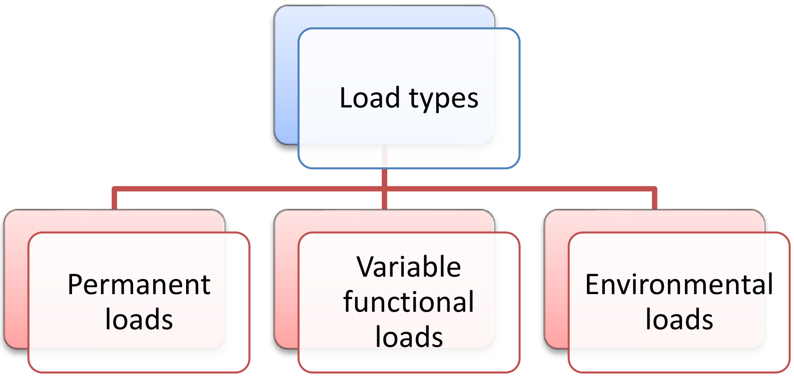


Figure C-0-1: Load types acting on OWTs

Permanent loads

The permanent loads also called death loads or static loads, are loads that generally remain constant during the lifetime of the wind turbine and do not vary in magnitude, the position of direction during the lifetime. The permanent loads may include the gravity loads and the hydrostatic loads. These loads can be calculated with high accuracy, by assessing the weight of the materials and their volumes. Although the permanent loads can be calculated very accurately, engineers are conservative with their calculations and allow a margin of error, to exceed the expectations in practice. Figure C-0-2 gives an overview of the permanent loads.

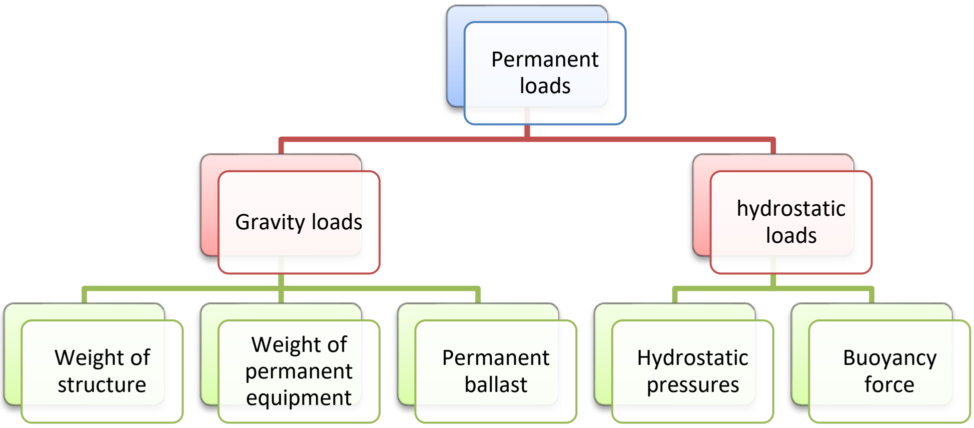


Figure C-0-2: Permanent loads acting on OWTs

Variable functional loads

Variable functional loads are the opposite of the permanent loads and may change in magnitude, position, and direction during the lifetime of the structure. The variable function loads of a wind turbine can be divided into actuation loads, which is due to the operation and the control of the turbine, loads from operation and ship impacts. An overview of the loads can be found in Figure C-0-3.

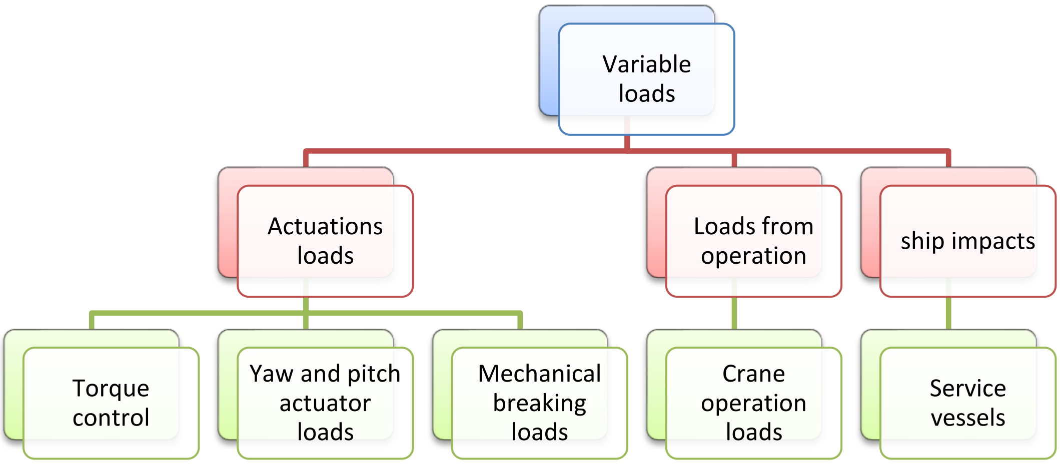


Figure C-0-3: Variable functional loads of OWTs

Environmental loads

Another important type of loading, especially for the offshore wind turbines, is environmental loads, which are caused by the environmental phenomena. The environmental loads vary during the lifetime, depending on the site conditions and the concept of the support structure. The environmental loads are determined as quantiles, with a certain probability of exceedance. The loads are random, and the uncertainty depends on the measuring time of the environmental loads, for which examples are given in Figure C-0-4

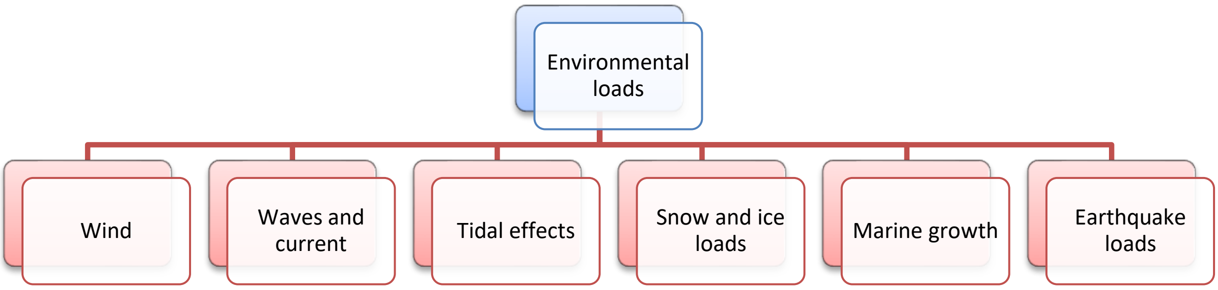


Figure C-0-4: Environmental loads of OWTs

The limit states

Offshore wind turbine support structures experience different loads and load combinations during their lifetime. To ensure that the structure is able to satisfy the design requirements, several limit states are defined in the design standards. The Limit state design (LSD) is a structural design method, which gives the limit where the structure's condition no longer fulfills its design criteria. In the LSD method, all the possible actions are considered, to ensure that the structure remains fit during the lifetime.

- **Fatigue limit state (FLS)**
  The FLS analysis are performed to verify the resistance to the cumulative damage, due to repeated loading during the lifetime of the structure. In this thesis, the FLS will be further examined, since fatigue is the important factor for lifetime extension of the OWT.
- **Ultimate limit state (ULS)**
  The ULS analysis is performed to verify the maximum load-carrying resistance, during the lifetime of the structure. The combinations of maximum wind and waves during 1-year, 5-year, and 50-year are used for the calculations.
- **Serviceability limit state (SLS)**
  The SLS analysis is performed to verify that the structure is useable during operation. The SLS check is to prove that under un-factored design loads (characteristic loads), the structural behavior does not exceed the SLS criteria value.
- **Accidental limit state (ALS)**
  The ALS analysis is performed to verify that the structure will not fail after an operational failure or an accidental event.

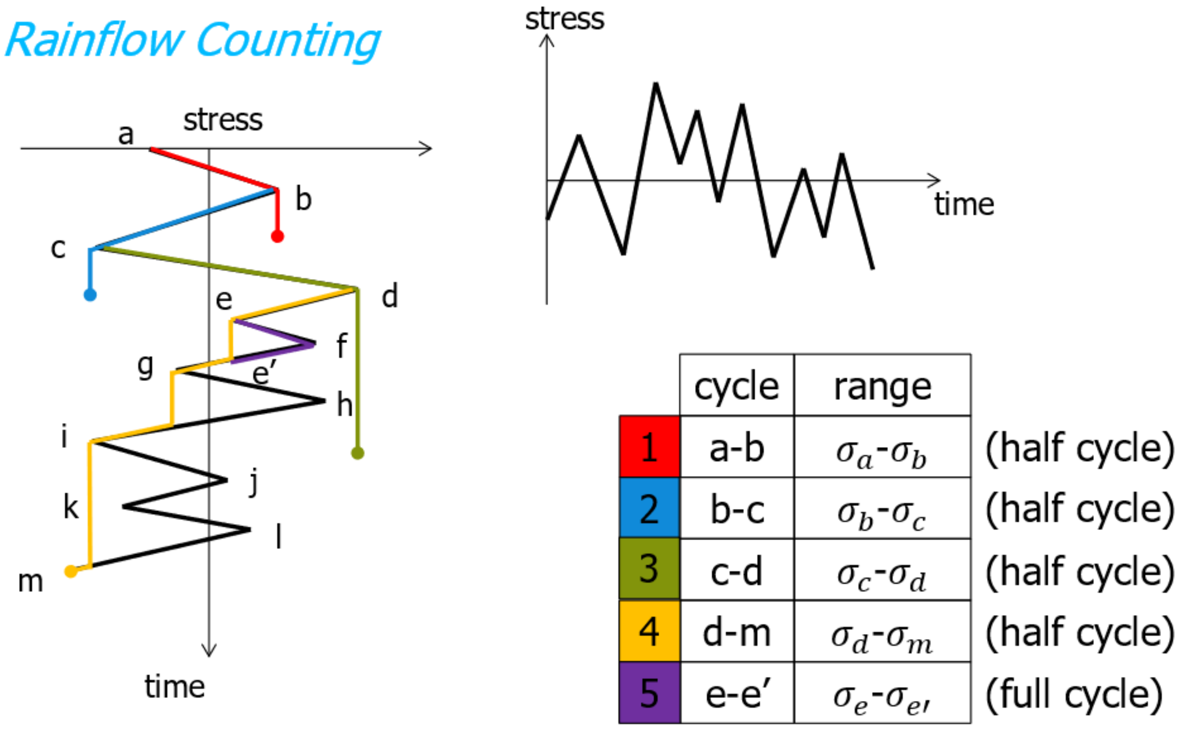
The design limit states are calculated according to IEC 61400-3 [58] by taking the load contributing events that occur during the OWT lifetime. In the preliminary design, only the basic load combinations are sufficient for obtaining the preliminary dimensions. In the detailed design phase, the basic load combinations are combined with the eight design situations that are described in the IEC 61400-3. These are the minimum load cases that should be considered in the OWT design. For each load case, multiple simulations of ten minutes need to be set up. The design load cases are briefly listed below.

- DLC 1.1-6 | Power production
- DLC 2.1-4 | Power production plus occurrence of a fault
- DLC 3.1-3 | Start-up
- DLC 4.1-2 | Normal/extreme shut down
- DLC 5.1 | Emergency shut down
- DLC 6.1-4 | Parked (standstill or idling)
- DLC 7.1-2 | Parked and fault conditions
- DLC 8.1-3 | Transport, assembly, maintenance and repair.

Appendix D

D Peak or rainflow counting

A structure or a member of a structure can resist a limited number of cycles of a certain stress range. It is important to know how many stress cycles occur. To count the stress cycles, the rainflow counting method of the peak counting method is used. An illustration of both methods are given in Figure D-0-5 and Figure D-0-6



	cycle	range	
1	a-b	σ_a-σ_b	(half cycle)
2	b-c	σ_b-σ_c	(half cycle)
3	c-d	σ_c-σ_d	(half cycle)
4	d-m	σ_d-σ_m	(half cycle)
5	e-e′	σ_e-$\sigma_{e'}$	(full cycle)

Figure D-0-5: Rainflow counting method [9]

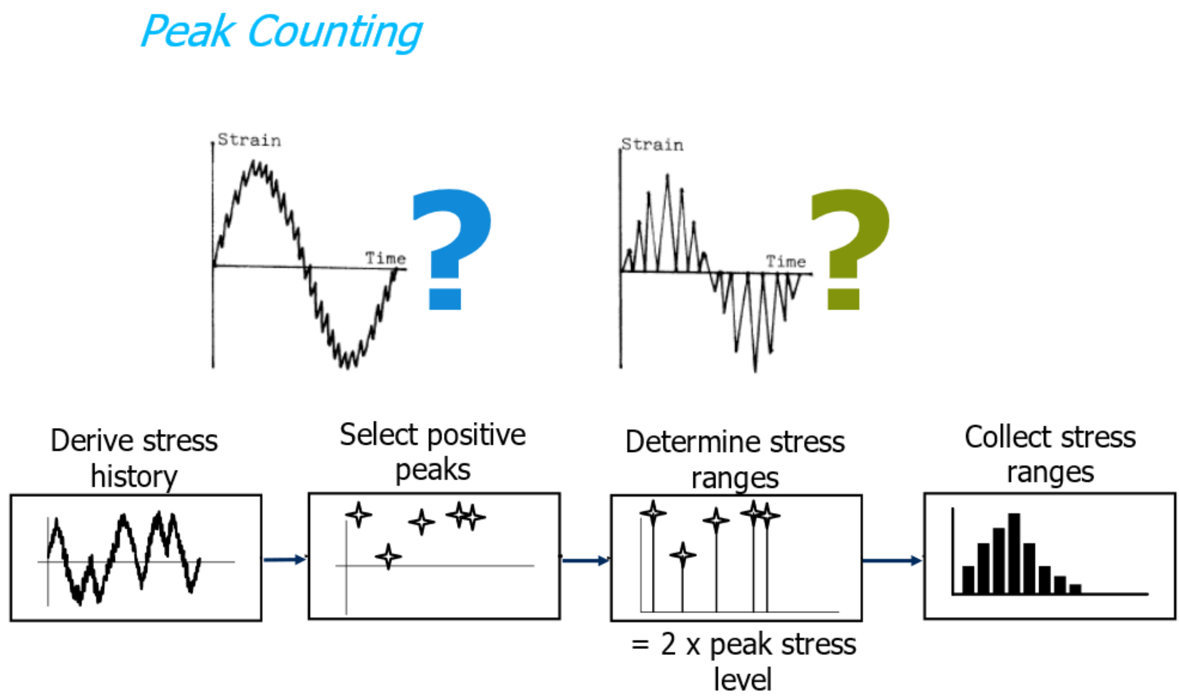


Figure D-0-6: Peak counting method [9]

Appendix E

E Probabibilistic background

A probabilistic background information is essential to understand the probabilistic calculation procedure. The different uncertainty types as described in Appendix A, gives a level of risk in most of the daily activities, especially in the design and management of engineering systems. Therefore, risk and safety are a key concept in the engineering world and should be considered explicitly.

Risk definition [59]
The term risk is used in different ways, with different definitions depending on the field. The most common definition, which is used in the probabilistic design is as follows:

Risk is the probability of an undesired event multiplied by the consequences

$$\sum_{si=1}^{i} E\ d\ = pi\ x\ di$$

The unit of the risk also expressed as the expected value E(d) is a function of the probability p_i and the consequences d_i. The probability of an event is generally expressed per unit time and the consequences are multi-dimensional with different types of consequences. The main part of the probabilistic design is to analyze and manage the risks. The objectives of a risk analysis are to identify the risks in the system, to know how the system can fail, their acceptability's and the weakest points in the system.

Structural reliability [60]
Structural reliability P_r is defined according to International Standards Organization (ISO) [61], *General Principles on Reliability for Structures* as the "ability of a structure or structural element to fulfill the specified requirements, including the working life, for which it has been designed." A **target or design reliability level** is the reliability level that the structure is designed to be at, or above, for the duration of its design life. This can be expressed as the inverse probability of failure P_f: the lower the probability of failure, the more reliable the structure, and the higher the reliability level.

$$Pf + Pr = 1$$

Reliability Levels [62]

The reliability of a structure is based on the applied safety margin between the resistance of the structure and the loads acting on the structure. The international joint committee for structure safety (JCSS) have determined various methods to describe and determine the safety level of structures. This section will give an overview of the four reliability levels and a summary is presented in Table 16.

- Level III method | Fully probabilistic
- Level II method | Probabilistic with approximations
- Level I method | semi-probabilistic (with safety factors)
- Level 0 method | deterministic (no reliability analysis)

Level III method

The level III method is the highest level in the probabilistic design that is fully probabilistic. The probability of failure is P_f is calculated exactly without any assumptions and simplifications. Different tools are used for the calculations, such as analytical formulations, numerical integration and varies types of Monte Carlo simulations. The performance of a structure can be described by a limit state function (LSF). The limit state function describes when a failure will occur and when no failure will occur. Let's take for example the performance of a system/structure, which is described by one limit state function Z, as:

$$Z = R - S$$

Where

Z = LSF
R = The resistance term
S = The action term

The LSF of the structure is now defined as a function of two stochastic variables. The structure will fail if the action term is larger than the resistance term, in other words, the structure will fail if $Z < 0$ and the structure is safe if $Z > 0$. The probability of failure is defined as follows:

$$Pf = P(R - S < 0)$$

The joint distribution of R and S and the failure plane of the limit state are illustrated in Figure E-0-7. The volume for which failure takes place can be calculated analytically for simple cases. When the limit state becomes more difficult, Monte Carlo simulation can be used to calculate the probability of failure.

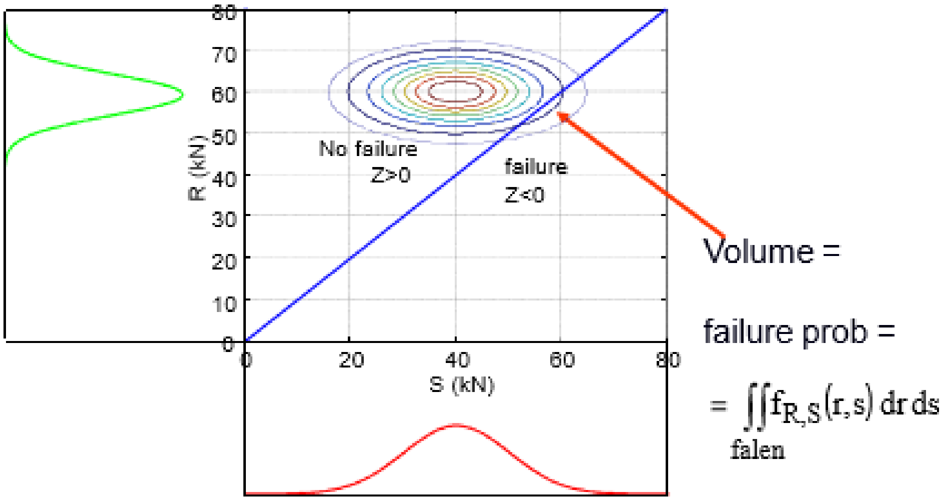


Figure E-0-7: Joint Distribution of R and S

Level II method

The reliability calculations in the level II are simplified in such a way that the limit state function is linearized and that all the distribution functions are approximated by normal or lognormal distributions. The linearization and the normalization take place in the design point. The design point is the point on the failure boundary with the highest probability density. If the parameters are normally distributed and the relation is linear, then the mean and the standard deviation of Z are defined as:

$$\mu_Z = \mu_R - \mu_S$$

$$\sigma_Z = \sqrt{\sigma_R^2 + \sigma_S^2}$$

Where,

µ = Mean value
σ = Standard deviations

If R and S are normal distributed, then Z is also normal distributed. In the case that R and S are not normal distributed, the distribution of Z is unknown. In order to calculate the probability of failure, the limit state is linearized in such a way that the error is very small. This point on the failure boundary is called the design point. The design point is defined as:

$$X_i = \mu_i - \alpha_i \beta_i \sigma_i$$

Where,
α = Sensitivity factor
β = Reliability index.

The sensitivity factor and the reliability index can be calculated using different probabilistic tools, such as FORM, SORM, etc.

Level I method

This probabilistic analysis method makes use of a partial safety factor both on the load side and on the resistance side. Using of partial safety factors is also the way that in many building codes the safety is dealt with. The safety factors in the design standards are calibrated in such a way that the structure has the required safety and in a way for which the following must hold, as illustrated in Figure 3.13:

$$\Upsilon_m = \frac{\mu_S - \alpha_S \beta \sigma_S}{R_{kar}}$$

$$\gamma_S = \frac{\mu_S + \alpha_S \beta \sigma_S}{S_{kar}}$$

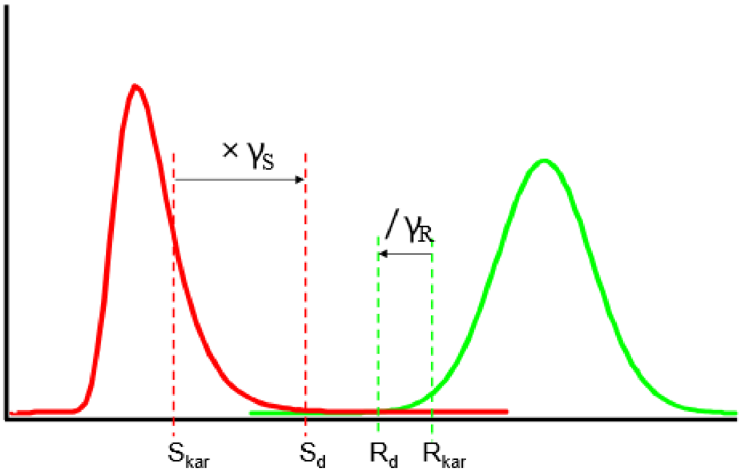
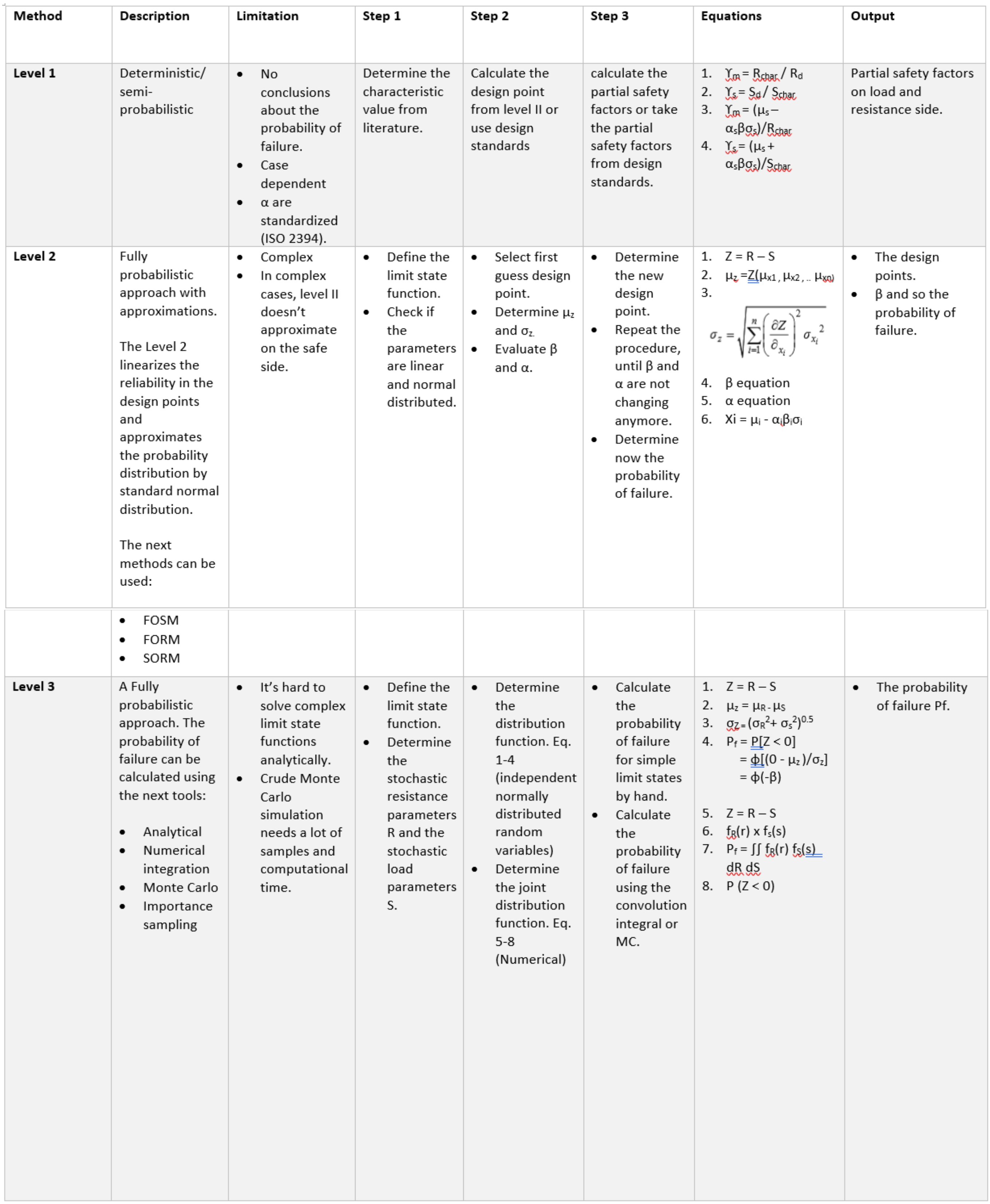


Figure E-0-8: Design values and partial factors

Table 16: Overview of the probabilistic levels



Method	Description	Limitation	Step 1	Step 2	Step 3	Equations	Output
Level 1	Deterministic/ semi-probabilistic	• No conclusions about the probability of failure. • Case dependent • α are standardized (ISO 2394).	Determine the characteristic value from literature.	Calculate the design point from level II or use design standards	calculate the partial safety factors or take the partial safety factors from design standards.	1. $\Upsilon_m = R_{char} / R_d$ 2. $\Upsilon_s = S_d / S_{char}$ 3. $\Upsilon_m = (\mu_s - \alpha_s \beta \sigma_s)/R_{char}$ 4. $\Upsilon_s = (\mu_s + \alpha_s \beta \sigma_s)/S_{char}$	Partial safety factors on load and resistance side.
Level 2	Fully probabilistic approach with approximations. The Level 2 linearizes the reliability in the design points and approximates the probability distribution by standard normal distribution. The next methods can be used:	• Complex • In complex cases, level II doesn't approximate on the safe side.	• Define the limit state function. • Check if the parameters are linear and normal distributed.	• Select first guess design point. • Determine μ_z and σ_z. • Evaluate β and α.	• Determine the new design point. • Repeat the procedure, until β and α are not changing anymore. • Determine now the probability of failure.	1. $Z = R - S$ 2. $\mu_z = Z(\mu_{x1}, \mu_{x2}, .. \mu_{xn})$ 3. $\sigma_z = \sqrt{\sum_{i=1}^{n} \left(\frac{\partial Z}{\partial_{x_i}} \right)^2 {\sigma_{x_i}}^2}$ 4. β equation 5. α equation 6. $Xi = \mu_i - \alpha_i \beta_i \sigma_i$	• The design points. • β and so the probability of failure.
	• FOSM • FORM • SORM						
Level 3	A Fully probabilistic approach. The probability of failure can be calculated using the next tools: • Analytical • Numerical integration • Monte Carlo • Importance sampling	• It's hard to solve complex limit state functions analytically. • Crude Monte Carlo simulation needs a lot of samples and computational time.	• Define the limit state function. • Determine the stochastic resistance parameters R and the stochastic load parameters S.	• Determine the distribution function. Eq. 1-4 (independent normally distributed random variables) • Determine the joint distribution function. Eq. 5-8 (Numerical)	• Calculate the probability of failure for simple limit states by hand. • Calculate the probability of failure using the convolution integral or MC.	1. $Z = R - S$ 2. $\mu_z = \mu_R - \mu_S$ 3. $\sigma_Z = (\sigma_R^2 + \sigma_s^2)^{0.5}$ 4. $P_f = P[Z < 0]$ $= \phi[(0 - \mu_z)/\sigma_z]$ $= \phi(-\beta)$ 5. $Z = R - S$ 6. $f_R(r) \times f_s(s)$ 7. $P_f = \iint f_R(r) f_S(s) dR dS$ 8. $P (Z < 0)$	• The probability of failure Pf.

The relation between the reliability index and the probability of failure is given in Table 17 and Table 18.

Table 17: Relationship between β and Pf [62]

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1.3	2.3	3.1	3.7	4.2	4.7	5.2

Table 18: Reliability indices (and the probability of failure in parentheses) for various consequences of failure and structure types [51]

Structure type	Target reliability (probability of failure)		
	Low consequence of failure	Moderate consequence of failure	High consequence of failure
Redundant structure	3.1 (10^{-3})	3.7 (10^{-4})	4.2 (10^{-5})
Non-redundant structure but significant warning before failure	3.7 (10^{-4})	4.2 (10^{-5})	4.7 (10^{-6})
Non-redundant structure and no warning before failure	4.2 (10^{-5})	4.7 (10^{-6})	5.2 (10^{-7})

Appendix F

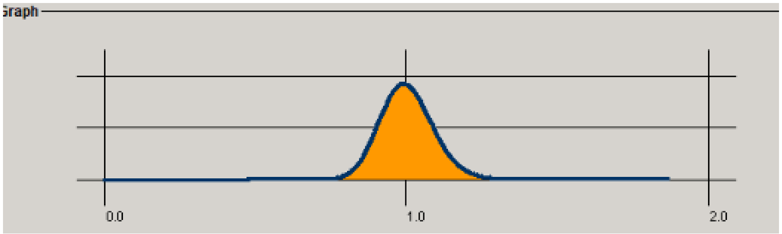
F Case study: uncertainty distributions

$X_{Miner's}$

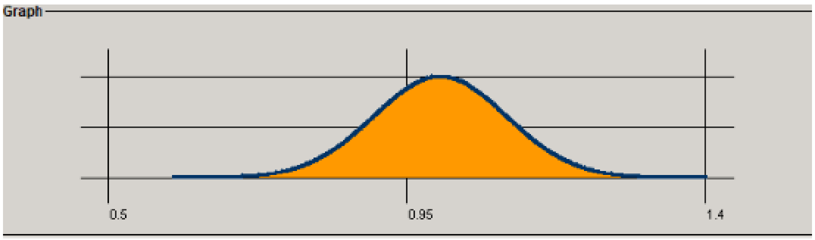
This distribution is lognormal so first convert the mean and standard deviation to lambda λ and zeta ζ:

$$\lambda = \ln(\mu)$$
$$\zeta = \sqrt{\ln((1+\delta^2))}$$

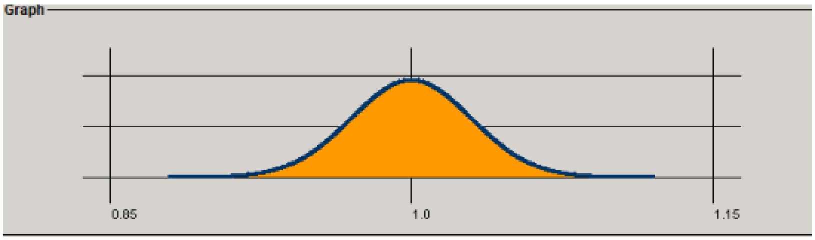
with
$$\delta = \frac{\sigma}{\mu}$$



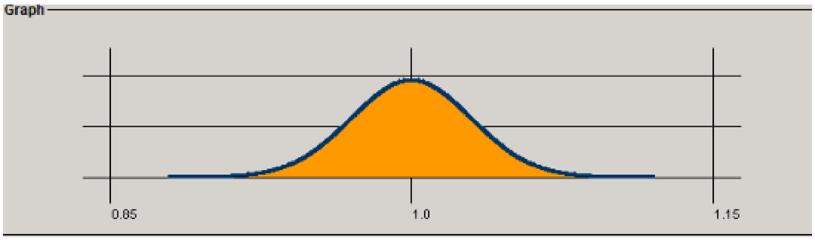
X_{load}:



$\chi_{peak\ counting}$:



$\chi_{lowcycle}$:



Appendix G

G Wave scatter diagram

	T (s)	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	
H (m)		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	
0,0	0,5	340898	1456555	1547001	831076	425466	206694	92565	38454	15279	6538	3204	1664	882	464	250	133	68	36	18	9	4	2	1	0	0	0	0				4967262
0,5	1,0	1	48143	439909	704975	600908	355870	165157	55316	13836	3179	699	176	52	16	4	1	0			0											2388241
1,0	1,5		611	33508	180188	286116	233857	118733	42096	11905	2810	591	128	22	5	1	0	0	0													910572
1,5	2,0		2	1821	35265	109299	122884	72145	29724	9241	2501	662	155	35	7	1	0	0	0													383743
2,0	2,5			93	5234	35357	55115	39441	17563	5937	1860	529	129	28	5	1	0	0	0													161292
2,5	3,0			2	685	10222	22850	19630	9129	3456	1142	325	75	15	2	0	0	0														67534
3,0	3,5			0	76	2709	8601	9018	4587	1879	621	173	40	7	1	0	0	0														27712
3,5	4,0				13	674	3273	3822	2214	964	319	83	18	3	0	0	0															11383
4,0	4,5				1	146	1114	1556	1052	470	150	40	7	1	0	0																4537
4,5	5,0				0	32	371	634	496	221	75	18	3	0	0																	1849
5,0	5,5					5	122	261	222	108	36	7	1	0	0	0																762
5,5	6,0					1	42	102	99	52	17	3	0	0																		316
6,0	6,5					0	11	41	47	24	7	1	0	0																		132
6,5	7,0					0	2	17	22	12	3	0	0	0																		57
7,0	7,5					0	1	7	10	6	1	0	0																			25
7,5	8,0					0	0	3	5	3	1	0																				11
8,0	8,5						0	1	2	1	0	0																				5
8,5	9,0						0	0	1	1	0	0	0																			2
9,0	9,5						0	0	0	0	0		0																			1
9,5	10,0							0	0	0	0																					0
10,0	10,5							0	0	0	0																					0
10,5	11,0							0	0	0																						0
11,0	11,5							0	0	0																						0
11,5	12,0								0	0																						0
12,0	12,5							0	0	0																						0
12,5	13,0								0																							0
13,0	13,5																															
13,5	14,0								0																							0
14,0	14,5																															
14,5	15,0																															
		340900	1505311	2022335	1757512	1470936	1010807	523132	201040	63395	19259	6335	2398	1046	501	257	135	68	36	18	9	4	2	1	0	0	0	0				8925438

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