Investigation of lumping methods for correlated wind-wave data and Frequency-Domain approach

Abhishek
Reduction Of Fatigue Computational Time For Offshore Wind Turbine Jacket Foundations

Investigation of lumping methods for correlated wind-wave data and Frequency-Domain approach

by

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An electronic version of this thesis is available at http://repository.tudelft.nl/.
Recognizing climate change as a result of more than 150 years of industrial activity, and due to the emission reduction protocols set internationally [29], wind energy has become the mainstream source of energy for many developed nations. Due to the prospect of large scale energy generation, wind energy has moved offshore. Offshore wind farms are relatively more complex to design and costlier to install. Since, substructure design of an offshore wind turbine (OWT) plays an important role and is one of the key drivers for capital cost, optimization of support structure design becomes imperative.

Driven by the universal goal of cost saving in offshore wind industry, the main objective is set to – “reducing computational time required for a typical fatigue design cycle of OWT support structure”. From the perspective of a foundation designer, following practical challenges are identified and investigated:

- Lumping methodology for correlated wind and wave scatter diagrams
- Reduction in number of fatigue design load cases in time domain simulations
- An alternative fatigue design method in frequency domain

This thesis focuses on the fatigue design of jacket type support structure. First, a conceptual 3-legged jacket with a light weight pyramid shaped transition piece is modeled in finite element software. Subsequently, an overview of loads, dynamic properties and fatigue analysis of support structure is presented.

In OWT support structure design, wind and wave loads are equally significant. For fatigue design, the combinations of loads which are likely to occur simultaneously during the design life of the structure are estimated. The correlation established between wind and wave is generally presented in the form of scatter diagrams. To reduce the computational time, common practice is to condense wind and wave scatter diagram and assess fatigue damage for the lumped states. Thus, in this thesis, an investigation is completed on commonly used lumping methodologies and comparison is established on the accuracy of fatigue results. In this study, existing lumping methods are validated for a jacket support structure and it is proven that dynamic characteristic of the structure plays an important role in condensing scatter diagrams.

Traditionally, the complete fatigue load case design of OWT support structure is based on time domain simulations, which is very time consuming. Hence, an important query raised by the Company (KCI) was, whether it is possible to reduce the number of design load cases while retaining a high level of precision in fatigue damage results. A novel method is presented in this thesis, which attempts to reduce the number of design load cases. For jacket support structure, linear-quasi-static assumptions are made to estimate approximate fatigue damages due to wind load alone. Using the estimated damage, critical wind seeds are identified and are sorted for reduction. Results from this case study show that with the reduction of load cases, the accuracy in fatigue damage is also compromised. This experimental method provides good insight into the load case reduction capabilities and is recommended for preliminary design phase.

Since, time domain fatigue design requires very large time records to accurately describe the random loading processes, it proves prohibitive for many finite element analyses. Thus, an alternate fatigue design approach in frequency domain is introduced in this thesis. Contrary to popular use of power spectral density functions, a different method is presented which successfully preserves the phase information of load time series. Frequency domain method offers many advantages like providing clear information regarding the environmental conditions and the dynamic properties of the structure. First, essentials of frequency domain method for a single degree of freedom system is described. Subsequently, this approach is
exemplified on OWT support structure with interface wind loads. Based on the assessment of response from time domain and frequency domain analysis, it is proved that frequency domain method is an effective tool which can lead to vast savings in computational times while still producing accurate fatigue results.

To summarize, this report gives an overview of the work done to reduce the computational time and effort required for a typical fatigue design cycle of an OWT support structure. Three measures were presented which helps with the reduction of computational time and the accuracy of end results were checked with full time domain simulations. As a result, one can perform more informed design optimization leading to reduction in costs for the support structure.
Acknowledgments

I would like to thank few too many people for their assistance, guidance and expert opinion throughout this difficult project.

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Delft, August 2018
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<th>Description</th>
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<tr>
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<td>Acceleration in $x$-direction</td>
<td>$[m/s^2]$</td>
</tr>
<tr>
<td>$\dot{x}$</td>
<td>Velocity in $x$-direction</td>
<td>$[m/s]$</td>
</tr>
<tr>
<td>$\hat{F}$</td>
<td>Amplitude of external Load</td>
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<td>$\hat{x}$</td>
<td>Amplitude of displacement</td>
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<td>$\mu$</td>
<td>Mean of stress time series</td>
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<td>$\phi$</td>
<td>Phase angle</td>
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<td>$\sigma$</td>
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<td>Axial stress due to force $F$</td>
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<td>Bending stress about $Y$ axis</td>
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<td>$3^{rd}$ natural frequency corresponding to $2^{nd}$ bending mode</td>
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<td>$f_p$</td>
<td>Peak frequency</td>
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<td>Description</td>
<td>Unit</td>
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<tr>
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<td>--------</td>
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<td>Spring stiffness</td>
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<td>Mass</td>
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</tr>
<tr>
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<td>Number of cycles to failure</td>
<td>[-]</td>
</tr>
<tr>
<td>$n_i$</td>
<td>Number of stress cycles in a block</td>
<td>[-]</td>
</tr>
<tr>
<td>$S_{\zeta \zeta}$</td>
<td>JONSWAP Wave spectral value</td>
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</tr>
<tr>
<td>$t$</td>
<td>Time</td>
<td>[s]</td>
</tr>
<tr>
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<td>Period associated with $H_{max}$</td>
<td>[s]</td>
</tr>
<tr>
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<td>Equivalent peak period</td>
<td>[s]</td>
</tr>
<tr>
<td>$T_p$</td>
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<td>[s]</td>
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<td>Mean zero crossing period</td>
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<td>$V_w$</td>
<td>Mean wind speed</td>
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<td>Displacement in $x$-direction</td>
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<td>1P</td>
<td>Rotational frequency of rotor turbine</td>
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<tr>
<td>3D</td>
<td>Three dimensional</td>
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<td>3P</td>
<td>Blade passing frequency of three-bladed turbine</td>
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<td>D</td>
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<td>[-]</td>
</tr>
<tr>
<td>D_MK</td>
<td>Hydrodynamic fatigue damage based on M. Kühn</td>
<td>[-]</td>
</tr>
<tr>
<td>D_MS</td>
<td>Hydrodynamic fatigue damage based on M. Seidel</td>
<td>[-]</td>
</tr>
<tr>
<td>D_rep</td>
<td>Representative damage</td>
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</tr>
<tr>
<td>D_SCD</td>
<td>Reference hydrodynamic fatigue damage</td>
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<tr>
<td>DLCs</td>
<td>Design load cases</td>
<td>[-]</td>
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<tr>
<td>DOF</td>
<td>Degrees of freedom</td>
<td>[-]</td>
</tr>
<tr>
<td>FD</td>
<td>Frequency domain</td>
<td>[-]</td>
</tr>
<tr>
<td>FE</td>
<td>Finite-element</td>
<td>[-]</td>
</tr>
<tr>
<td>FEM</td>
<td>Finite-element model</td>
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<tr>
<td>FFT</td>
<td>Fast Fourier Transform</td>
<td>[-]</td>
</tr>
<tr>
<td>FLS</td>
<td>Fatigue limit state</td>
<td>[-]</td>
</tr>
<tr>
<td>FX</td>
<td>Force due to wind load in $X$ direction</td>
<td>[$N$]</td>
</tr>
<tr>
<td>FY</td>
<td>Force due to wind load in $Y$ direction</td>
<td>[$N$]</td>
</tr>
<tr>
<td>FZ</td>
<td>Force due to wind load in $Z$ direction</td>
<td>[$N$]</td>
</tr>
<tr>
<td>GBS</td>
<td>Gravity-based structures</td>
<td>[-]</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>Description</td>
<td>Unit</td>
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<td>Hs_KCI</td>
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<td>Lumped significant wave height based on M. Kühn</td>
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<tr>
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<td>Lumped significant wave height based on M. Seidel</td>
<td>[-]</td>
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<tr>
<td>iFFT</td>
<td>Inverse Fast Fourier Transform</td>
<td>[-]</td>
</tr>
<tr>
<td>LAT</td>
<td>Lowest astronomical tide</td>
<td>[m]</td>
</tr>
<tr>
<td>MX</td>
<td>Moment due to wind load in X direction</td>
<td>[Nm]</td>
</tr>
<tr>
<td>MY</td>
<td>Moment due to wind load in Y direction</td>
<td>[Nm]</td>
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<tr>
<td>MZ</td>
<td>Moment due to wind load in Z direction</td>
<td>[Nm]</td>
</tr>
<tr>
<td>n</td>
<td>Number of load cycles</td>
<td>[-]</td>
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<tr>
<td>OWF</td>
<td>Offshore wind farm</td>
<td>[-]</td>
</tr>
<tr>
<td>OWT</td>
<td>Offshore wind turbine</td>
<td>[-]</td>
</tr>
<tr>
<td>p(n)</td>
<td>Probability for sea state n</td>
<td>[-]</td>
</tr>
<tr>
<td>PDF</td>
<td>Probability density function</td>
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<td>PSD</td>
<td>Power spectral density</td>
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<td>RFC</td>
<td>Rainflow counting</td>
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<tr>
<td>T</td>
<td>Duration of measurement</td>
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<td>TD</td>
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<td>Lumped peak period based on KCI method</td>
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<td>Lumped peak period based on M. Kühn</td>
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<tr>
<td>Tp_MS</td>
<td>Lumped peak period based on M. Seidel</td>
<td>[-]</td>
</tr>
<tr>
<td>WTM</td>
<td>Wind turbine manufacturer</td>
<td>[-]</td>
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</table>
1. Introduction

1.1. Offshore wind

The wind has played a long and important role in the history of human civilization. The popularity of using energy in the wind has always fluctuated with the price of fossil fuels. When fuel prices fell after World War II, interest in wind turbines disappeared. With the oil crises in the beginning of the 1970s, interest in wind power generation resumed. Furthermore, it is well known that wind energy is one of the cleanest and most environment friendly energy sources, and unlike fossil fuels, it will never be depleted [8].

Wind energy is mainly classified into two categories, onshore and offshore. Onshore wind energy is a matured industry and has been around for more than thousand years. It is responsible for partly meeting the energy demands in countries around the world. In contrast, offshore wind power generation is fairly new. In recent years, wind energy has begun to move offshore because of the large-scale electricity generation opportunities. Offshore wind power is more complex and costly to install and maintain but also offers several advantages. There is significantly higher energy production due to typically stronger and more stable wind at sea. Wind turbines can be bigger because of the available large continuous areas. It also eliminates the issues of visual impact and noise as it is far away from residential areas. Figure 1.1 shows the evolution of wind turbine to deeper water.

![Figure 1.1: Expected wind turbine evolution from land to deeper water [36]](image-url)
A major difference between onshore and offshore wind farms is the relative complexity and cost, especially of the substructures required for offshore turbines. In offshore wind industry, substructure supply and installation amounts to almost 20% [6] of the capital costs. Different types of substructures have been utilized and proposed till date. Types of bottom founded structures include monopile, tripod and jacket with various types of foundations at the sea bed which comprises of piles, gravity base and suction caissons. For deeper water depths, where fixed foundation becomes uneconomical or unfeasible, floating wind turbines are needed. Floating wind turbine is an undeveloped industry still fighting to be popular among the available commercial foundation choices. Hywind Scotland is the first operating floating wind farm, with a capacity of 30 MW, installed in the North Sea of Scotland [26].

Important considerations when selecting a support structure type include cost, water depth, seabed conditions, turbine characteristics and technical/commercial risk factors. The majority of the wind farms currently in operation in water depths of under 20-25 meters have monopile foundations, as they are relatively simple to produce, cost less and are easier to install. Gravity-based structures (GBS), which are also relatively easy to produce, make up most of the remainder, while only a small number of space-frame structures (e.g. jackets, tripods and tripiles) have been installed so far [6]. Figure 1.2 illustrates the history of offshore wind turbine foundation market.

As the wind energy industry is progressing, wind turbines are evolving to multi-megawatt machines. With the size of the turbines becoming larger and larger and with the saturation of available space in shallow waters, offshore wind turbines are moving into deep waters. For deeper locations, space frame structures are likely to be considered.

The global market for offshore wind energy is rooted in Europe. Europe still dominates the market with 90% of the global installed capacity. Figure 1.3 shows the offshore wind capacity across six European markets in 2016.
1.2. Study Objective

In detailed design of OWT support structures, high number of load cases have to be assessed, which makes the computational effort for standard time domain fatigue load analysis significant. In real systems, coupled wind and wave excitation lead to dynamic interaction between aerodynamic loads from the turbine rotor and hydrodynamic loads on the support structure.

To accurately estimate hydrodynamic fatigue damages, system has to be solved for full scatter diagrams. In order to reduce computation time, general practice is to lump the correlated wind and wave data. This effectively reduces the full met-ocean data to a lumped state. The idea is to accurately estimate hydrodynamic fatigue damage from lumped sea-state, equivalent to the damage accounting from full wave climate. Lumping methodology becomes very important in fatigue analysis. Hence, the first objective of this thesis is to investigate existing lumping methods for correlated wind and wave data. The final goal is to: “Reduce computational effort and time required for a typical design cycle of OWT support structure by either reducing the number of design load cases (DLCs) in time domain or by venturing frequency domain”.

The most common type of support structure used so far is the monopile. With offshore wind moving into deep waters, expectation is that space frame structures will be more common in the future. Generally speaking, space frame structures can be sub-divided into two categories: multi-pods (including tri-pods and tri-piles) and jackets. Jackets differ from tri-pods and tri-piles as they consist of a larger plan area, positioning the steel further from the center of the axis, which results in significant material savings. Hence, for this reason, a concept design of 3-legged jacket type support structure has been chosen for this study.

1.3. Literature review

1.3.1. Lumping of scatter diagrams

The above mentioned objective of condensing wind and wave scatter diagrams has been addressed by Martin Kühn (2001) in “Dynamics and design optimization of offshore wind energy conversion systems”[24]. M. Kühn suggests an iterative procedure for lumping of load cases by superposition of short-term fatigue loads. The starting point for the iterative wave lumping approach is formed by an initial guess on the significant wave height and zero-up crossing wave period. In analogy to damage equivalent loads, the significant wave height requires the selection of a representative S-N curve slope m. The initial wave period is obtained from

![Figure 1.3: Offshore wind capacity across European market in 2016][9]
1.4. Reference Wind park Site

To increase the realism of this academic thesis, Moray Firth offshore wind farm [30] is selected to serve as the reference. KCI was involved as the supporting design consultant for Van Oord in conceptual design of jacket foundation.

The Moray Firth Offshore Wind Farm is located on the Smith Bank in the outer Moray Firth in Scotland region. It lies 12 nautical miles (approximately 22km) from the Caithness coast and covers an area of 281 square nautical miles or 520 square km. The water depth
for the OWF is in the range of 39.82m to 47.75m. At the time, the wind farm was proposed to comprise of 63 Vestas turbines of 8 MW. Refer figure 1.4 for the location.

Figure 1.4: Moray Firth offshore wind farm location [25]

1.5. Thesis Approach
This thesis investigates various ways to optimize required time for fatigue analysis of OWT support structure. Chapter 2 provides general insight on the design considerations and the support structure chosen for this thesis. Design parameters along with environmental data and load parameters are introduced. Dynamic properties of the support structure are obtained after performing natural frequency analysis. Afterwards, general industry procedure for fatigue limit state analysis is shown.

In chapter 3, widely used existing lumping methodologies for correlated wind and wave data are introduced. Three main approaches are investigated for current support structure design with available environmental data. Then, these lumping methods are compared based on the resulting hydrodynamic fatigue damage at three locations within the jacket support structure.

A new hypothesis is proposed in chapter 4 for the reduction of design load cases (DLCs) based on the available wind time series. It is of company’s key interest to check if a simple methodology can be proposed to select few design load cases in order to reduce the computation time required for fatigue analysis in time domain. Chapter 5 gives a general overview of frequency domain approach. Furthermore, method to perform fatigue analysis in frequency domain for wind loads are explored. Chapter 6 provides a summary of conclusions and gives a viewpoint on potential developments for fatigue design practice of OWT support structures.

1.6. Software
The following computer programs are used in this thesis:

- SACS, structural global analysis, Bentley
- ANSYS, general purpose finite element program, Ansys Inc.
- MATLAB, mathematical modelling program, MathWorks Inc.
- EXCEL, spreadsheet program, Microsoft Inc.
2.1. Introduction
The design cycle of an offshore wind turbine (OWT) support structure is presented in Section 5.1, Figure 5.1. In order to design a support structure, various load inputs are required. An OWT support structure is mainly exposed to three types of loading – wave loads acting on the submerged part, wind load acting over the entire exposed area and the operational loads due to the wind turbine rotor. Modeling of these loads are presented in this chapter, which will be used to perform case studies in the succeeding chapters.

This chapter also describes the support structure design, in which a 3-legged jacket structure is found to be the most suitable for the given local conditions. Subsequently, natural frequency analysis of jacket structure is performed, followed by fatigue limit state (FLS) design criteria for the design of the support structure.

2.2. Wave and current loads
Sea-surface elevation in the presence of wind-generated waves, at any location and any moment in time is random. To catch this random process, some models have been developed over the years. Measurement of surface elevation in time for a random sea can be represented both as a time varying signal or as an energy density spectrum. Some characteristic parameters can be defined. The significant wave height of the spectrum, $H_s$, is defined as the mean of the 1/3 highest waves in the time series. The mean zero crossing period $T_z$ is defined by dividing the measurement time by the number of zero up-crossings. A visually more characteristic parameter is the frequency at which the peak occurs, $f_p$, as is its inverse $T_p$, which is referred to as peak frequency and peak period.

A frequently used spectral shape is the Pierson-Moskowitz wave spectrum [33]. This spectrum represents sea states that are fully developed. Further measurements of wave spectra were done in the Join North Sea Wave Project from which the JONSWAP spectrum instigated [17]. This spectrum represents sea states that are not fully developed under a certain wind condition. Although these two are the most common spectra, other descriptions also exist. JONSWAP spectrum for a significant wave height of 0.25m and peak period of 4.0s is shown in figure 2.1, which is derived in accordance with ISO 19901-1 [2].
To calculate wave forces on the structure, wave kinematics are used which can be determined using a linear or non-linear wave theory [2]. Hydrodynamic loading is then calculated by using the wave particle kinematics with the help of Morison Equation [13]. The extreme wave heights together with the associated wave period are listed in Table 2.1.

<table>
<thead>
<tr>
<th>Return period</th>
<th>Significant wave height</th>
<th>Peak period</th>
<th>Maximum Wave height</th>
<th>Period associated with Hmax</th>
</tr>
</thead>
<tbody>
<tr>
<td>RP [years]</td>
<td>$H_s$ [m]</td>
<td>$T_p$ [s]</td>
<td>$H_{max}$ [m]</td>
<td>$T_{ass}$ [s]</td>
</tr>
<tr>
<td>1</td>
<td>7.6</td>
<td>14.7</td>
<td>12</td>
<td>14.2</td>
</tr>
<tr>
<td>10</td>
<td>8.5</td>
<td>15.5</td>
<td>12.6</td>
<td>15.8</td>
</tr>
<tr>
<td>50</td>
<td>8.9</td>
<td>15.9</td>
<td>14.5</td>
<td>16.7</td>
</tr>
<tr>
<td>100</td>
<td>9.1</td>
<td>16.1</td>
<td>14.6</td>
<td>17</td>
</tr>
</tbody>
</table>

In addition, effect of marine growth is also considered as per DNV guidelines [12]. The support structure is also exposed to currents which are caused due to tidal variations, storm surge and atmospheric pressure variations. The currents are usually largest where large tidal variation occurs or where the local bathymetry has drastic influence. In this thesis, fatigue analysis is of prime concern. And since, current is assumed to induce a static load, it is not considered in fatigue load cases. Thus, detailed information about current is not provided furthermore.

For more in-depth knowledge on waves and current loading, reader should refer to [20].

### 2.3. Wind and Operational loads

#### 2.3.1. Wind load

Dependence of wind loads acting on OWT can be mainly characterized into the following aspects:

- Wind velocity
- Wind shear
- Turbulence
Wind velocity measured in the field varies with space, time and direction. Wind speed is height dependent, where mean wind speed increases with height. This effect is known as wind shear. The change in wind speed with varying height depends on the surface roughness of the area. Since, the surface roughness at sea is very low, the wind shear is smaller. The shear effect on the mean wind speed at a certain elevation is mainly described by two models: the logarithmic profile and the power law profile [12].

The mean wind speed is superimposed by temporal and spatial fluctuations of the wind speed at a specific location. These fluctuations are defined as turbulence and the amount of turbulence is specified by turbulence intensity. It is based on roughness and altitude parameters and is defined as the standard deviation of the time varying wind speed divided by the mean wind speed. The largest wind loads are attracted by the blades of the turbine, consisting of lift and drag forces. As these load components are structure and location dependent, the wind loads acting on the OWT vary significantly for different structures and locations.

For detail information on wind load, readers can refer to [35].

For this project, wind loads are provided by the turbine manufacturer (Vestas), in the form of time series of dynamic response corresponding to 6 degrees of freedom (DOF) at the interface location. Time series is generated for a random seed corresponding to a specific design load case (DLC). An example of wind load time series corresponding to random seed is graphically represented in figure 2.2.

![Wind Time series](image)

(a) FX, FY and FZ

(b) MX, MY and MZ

Figure 2.2: Wind load time series in the form of forces and moments

### 2.3.2. Operation load

For OWT system, an important source of excitation is the rotor. The rotational speed of the rotor is the first excitation frequency, referred to as 1P. The structure can experience an excitation at this frequency due to mass imbalance loads. The second excitation frequency is the rotor blade passing frequency, i.e. for a turbine with three bladed rotor, it is 3P. There is a temporary change in the loads acting on the blade as it passes the tower, which excites the structure.

The support structure can get excited because of harmonic rotor loads at multiples of rotor speed, i.e. 6P, 9P and so on. Since, the wind turbines operate at variable speed, it introduces an excitation frequency band at 1P and 3P frequencies (refer figure 2.3). For this case study, the rotor speed ranges from 6.5 to 14 RPM, with a nominal speed of 10.5 RPM. Thus, to avoid resonance, the allowable frequency range specified by the manufacturer is 0.192 to 0.261 Hz [1].
2.4. Design considerations

2.4.1. Design Basis
A site in 49.93m water depth (w.r.t. Lowest astronomical tide) located on the Smith bank in the outer Moray Firth, Scotland region is selected. The approximate location is indicated in Figure 1.4, Section 1.4. Metocean data was provided by ABP Marine Environmental Research Ltd [25]. The data includes wind, wave, current and water level measurements. Key environmental data is presented in Section 2.2 and 2.3.

The turbine used in this research is V164-8.0 MW turbine [1]. The nacelle weighs 289.1 tons, while the rotor assembly weighs 185.4 tons. The rotor diameter is 164m. The key parameters are listed in table 2.2.

Table 2.2: Important elevations

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hub elevation</td>
<td>115.00m</td>
</tr>
<tr>
<td>Top of tower</td>
<td>112.00m</td>
</tr>
<tr>
<td>Tower height</td>
<td>83.60m</td>
</tr>
<tr>
<td>Distance of Tip from Hub</td>
<td>82.00m</td>
</tr>
</tbody>
</table>

2.4.2. Base model
A 3D stiffness idealization of 3-legged jacket support structure model is built using “Beam” elements in SACS. Henceforth, this 3D idealization will be referred to as the “Base model”. This is built in accordance with the following principles:

- All the sections are modeled explicitly at their actual dimensions and axial offsets are applied.
- J-tubes are modeled as structural elements to attract wave load.
- Turbine sections and tower are modeled as per the provided information from turbine manufacturer and mass properties of the rotor and nacelle are simulated at corresponding locations.
- Double boat landing including ladder is incorporated in the model to accurately estimate the wave loads.
- Effective length factors are included for the member code checks.
- Jacket legs and piles are considered flooded and braces are considered buoyant.

\(^1\)Elevations are with respect to lowest astronomical tide (LAT)
2.4. Design considerations

- The transition piece is modeled using plate elements.
- The corrosion allowance is considered by reducing the wall thicknesses.
- Design pile stick ups with pre-piling strategy is used, with a pile penetration depth of 38m.

The configuration of base model built in SACS is shown in figure 2.4.

![Figure 2.4: Geometric configuration of Base model](image)

The pile foundation is simulated using pile-soil interaction module of SACS. The soil stiffness properties (shear, lateral and bearing) are used as per the Geo-technical report [10]. Key parameters for jacket design are shown in table 2.3.

Table 2.3: Key elevations and dimensions for jacket design

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interface elevation</td>
<td>28.40m$^1$</td>
</tr>
<tr>
<td>Height of transition piece</td>
<td>5.25m</td>
</tr>
<tr>
<td>Bottom of Deck level</td>
<td>16.70m$^1$</td>
</tr>
<tr>
<td>Air gap</td>
<td>1.78m</td>
</tr>
<tr>
<td>Splash zone upper limit</td>
<td>8.52m$^1$</td>
</tr>
<tr>
<td>Splash zone lower limit</td>
<td>-1.69m$^1$</td>
</tr>
<tr>
<td>Top width of jacket</td>
<td>11.77m</td>
</tr>
<tr>
<td>Base width of jacket</td>
<td>27.5m</td>
</tr>
<tr>
<td>Foundation pile diameter</td>
<td>2.4m</td>
</tr>
</tbody>
</table>

$^1$Elevations are with respect to lowest astronomical tide (LAT)
2.4. Design considerations

Detail structural drawings for the chosen concept is presented in Appendix A.

2.4.3. Natural frequency analysis

For natural frequency analysis, target natural frequency is determined based on the turbine rotational speed as shown in figure 2.3. The structure natural frequency should not coincide with either 1P or 3P excitation frequency band. This leaves us with three possible frequency intervals. Natural frequency less than 1P (soft-soft structure), natural frequency between 1P and 3P (soft-stiff structure) and natural frequency greater than 3P (stiff-stiff structure). Different intervals are indicated in figure 2.5.

![Figure 2.5: Allowable frequency range](image)

When the structure is designed with first eigenfrequency in the soft-soft region, there is a greater chance of resonance due to wave excitation. And when the structure is designed with first eigenfrequency in the stiff-stiff region, the structure becomes too expensive because of unnecessary material. Hence, the most common option is the soft-stiff region. Accordingly, the structure is designed with first eigenfrequency of 0.239 Hz \( (f_1) \) falling under soft-stiff region. Table 2.4 summarizes the natural frequency values for base model. The first few mode shapes are illustrated in top view in figure 2.6. Mode shapes in front and side view are referred to Appendix B.

Table 2.4: Natural frequency for Base model

<table>
<thead>
<tr>
<th>Tower mode</th>
<th>Natural frequency [Hz]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st Tower Side-to-Side</td>
<td>0.239</td>
</tr>
<tr>
<td>1st Tower Fore-Aft</td>
<td>0.243</td>
</tr>
<tr>
<td>2nd Tower Side-to-Side</td>
<td>0.776</td>
</tr>
<tr>
<td>2nd Tower Fore-Aft</td>
<td>0.782</td>
</tr>
<tr>
<td>3rd Tower Side-to-Side</td>
<td>1.369</td>
</tr>
<tr>
<td>3rd Tower Fore-Aft</td>
<td>1.555</td>
</tr>
<tr>
<td>1st Tower Torsion</td>
<td>1.575</td>
</tr>
</tbody>
</table>
2.4. Design considerations

Figure 2.6: Mode shapes illustrated in top view

(a) 1st tower side to side: \( f_1 = 0.239 \text{Hz} \)
(b) 1st tower fore-aft: \( f_2 = 0.243 \text{Hz} \)
(c) 2nd tower side to side: \( f_3 = 0.776 \text{Hz} \)
(d) 2nd tower fore-aft: \( f_4 = 0.782 \text{Hz} \)
(e) 3rd tower side to side: \( f_5 = 1.369 \text{Hz} \)
(g) 3rd tower fore-aft: \( f_6 = 1.555 \text{Hz} \)
(f) 1st tower torsion: \( f_7 = 1.575 \text{Hz} \)

Figure 2.6: Mode shapes illustrated in top view
2.4. Design considerations

2.4.4. Fatigue limit state analysis

A comprehensive design of OWT support structure involves checks for various limit state design, such as, ultimate limit state, serviceability limit state, accidental limit state, fatigue limit state and so on. Usually, the design of OWT foundations is governed by the fatigue limit state. Since, this thesis is mainly focused on fatigue analysis, only fatigue limit state is introduced in this section.

By definition, fatigue is the process of gradual damage that occurs when a structure is subjected to continuously changing stresses or alternating loads. Due to these stress changes, the structure starts to deteriorate, developing cracks leading to failure. OWT structures are by default exposed to cyclic loading from wind as well as waves. Hence, fatigue analysis is an important design criterion when it comes to OWT structures.

Most common practice to calculate fatigue damage or fatigue life in time domain is based on S-N fatigue approach under the assumption of linear cumulative damage (Palmgren-Miner rule). This is an empirical design method where S-N curve is created for a typical structural connection based on fatigue tests. In these curves the stress amplitude is plotted versus the logarithm number of cycles which provides information about the maximum number of cycles of a certain stress, up to crack initiation. To use S-N fatigue approach, first the number of alternating stresses and corresponding cycles has to be extracted from the stress time series. Though there are a number of cycle counting algorithms, Rainflow counting method is the most popular. For more information regarding the counting methods, readers can refer to [35].

Next, taking the associated maximum allowable number of cycles for each stress range from the S-N curve, Palmgren-miner rule is applied. Miner rule is given by equation 2.1 which states that the cumulative fatigue damage $D_{fat}$ is equal to the sum of number of stress cycles in a block ($n_i$) over number of cycles to failure ($N_i$) for all stress range class.

$$ D_{fat} = \sum \frac{n_i}{N_i} $$

Miner rule states that if the value of $D_{fat}$ is less than 1, then failure will not transpire.

2.4.5. Finite-Element model

To carry out time domain fatigue analysis in this thesis, two FE software are used - namely, SACS and ANSYS Mechanical APDL. SACS offshore suite is capable of generating dead, buoyancy, wave and current loads on offshore structures and can analyze and perform member code checks. Additionally, wind time series can be included to account for loads generated from wind. Thus, for industrial practice, SACS program is very user-friendly and can directly generate fatigue damage values in members for combined wind and wave load effect. This makes the post processing faster and it does not need large storage capacity. But, the software in itself is like a black box, where it was found very difficult to extract intermediate steps in the design process. For example, during the fatigue analysis in time domain, it was not possible to extract even the resulting stress response time series. This proves to be a big limitation for research purposes.

On the other hand, ANSYS APDL offers an extensive outlook and transparency while designing a structure. The drawback with ANSYS APDL is that, it is less user-friendly and does not have automatic post processing capabilities like SACS. Another downside with computation in ANSYS is that it creates very large database files and requires more computational time to post-process. Thus, the computer system requirements are rather high in this case.

In this thesis, analysis performed in Chapter 3 and 4 are based on SACS FEM and the computations executed in Chapter 5 are using ANSYS FEM. Switch from SACS to ANSYS is made in Chapter 5 because it is essential to derive intermediate results, which was not possible with SACS program. Figure 2.7 shows a 3D representation of FEM in SACS and ANSYS.
Figure 2.7: 3D FEM idealization in SACS and ANSYS
Lumping of correlated wind-wave scatter diagrams

3.1. Introduction

Among the many challenges and uncertainties faced during a design cycle of offshore wind turbine structures, a crucial problem that stands out is the establishment of wind-wave correlation based on met-ocean data. In order to decrease the required number of calculations, the correlated wind-wave data, generally in the form of scatter diagrams (wind speed vs. wave height & wave height vs. wave period) are often condensed or lumped. Establishment of such a "lumping" is very important for an accurate estimate of design loads. Thus, it is crucial for the estimation of fatigue damage that the structure incurs. During the thesis, it was observed that a strong basis for calculation of lumped sea-states was missing. Hence, further investigation is done into lumping methodologies to improve the accuracy of fatigue damage results.

This chapter presents a brief introduction and a worked out example of the existing lumping methods for correlated wind-wave data. Mainly the following three approaches is discussed:

- Martin Kühn’s simplified damage equivalent approach [24]
- Marc Seidel’s spectral energy approach [34]
- KCI method similar to probability based averaging [30]

To calculate the lumped sea-state parameters using different methods, two scatter diagrams which are generated based on the hind-cast data are used as shown in figure 3.1 and 3.2 [25].

![Figure 3.1: Wave height, wind unidirectional scatter diagram with number of occurrences][1]

---

[1]: #Figure 3.1: Wave height, wind unidirectional scatter diagram with number of occurrences [25]
3.2. Martin Kühn’s simplified damage equivalent approach

3.2.1. Introduction

According to Martin Kühn [24] by assuming a linear response, the stress ranges are considered proportional to the significant wave height as well as the standard deviation of the wind speed and thus mean wind speed, presuming constant turbulence intensity. The number of stress cycles is of lower influence on the damage than the stress range. Therefore, the number of stress cycles varies with the inverse zero-crossing period which holds, strictly only for a quasi-static response. The damage itself is proportional to the $m$th power of the stress ranges and the total number of cycles, $n_{total}$.

Simplified damage equivalent wave heights

This approach aims at the preservation of the wave period distribution while establishing an associated distribution of wave heights for all wave period classes. The estimate of lumped wave height parameters is given by equation 3.1.

$$H_{x,m,j} = \left( \frac{\sum_i N_{i,j} p_{i,j} H_{x}^{m,i}}{p_{j}} \right)^{1/m}$$

where:

- $i$ = index of number of wave height classes
- $j$ = index of number of wave period classes
- $p_{i,j}$ = probability of $i^{th}$ wave height and $j^{th}$ wave period
- $p_{j}$ = probability of $j^{th}$ wave period class
- $m$ = S-N curve slope
- $N_{i,j}$ = number of stress cycles
- $H_{x}$ = significant wave height
- $T_{x,j}$ = zero crossing wave period

Equation 3.1 is based on the assumption that fatigue loads are linearly proportional to wave heights and consequently that fatigue damages are proportional to the probability of occurrence and the $m$th power of the significant wave height.

Analogous to the above approach, same formula is followed for establishing associated distribution of wave heights for all wind speed classes.

Averaging of wave frequencies

This approach aims at the preservation of the significant wave height distribution while establishing an associated wave period distribution for all wave height classes based on an averaging of the relative number of waves as the product of probability and wave frequency. Lumped wave peak period is estimated by equation 3.2 while preserving the significant wave height distribution.

$$T_{z,n,i} = \left( \frac{\sum_j N_{j,n} p_{j,l}/T_{z,j}}{p_{l}} \right)^{-1}$$

Where, $T_{z} = $ zero crossing wave period.
3.2.2. Application

From the two scatter diagrams generated based on hind-cast data (figure 3.1 and 3.2), probability distribution is calculated for each of those scatter diagrams. Next, Martin Kühn’s approach of simplified damage equivalent wave height and period is applied. For this application, S-N curve with slope $m$ is considered to be 3.

From wave height and wind speed scatter diagram, the correlated wind speed and wave height data are lumped. Similarly, from wave height and peak period scatter diagram, correlated wave height and peak period data are lumped using averaging of wave frequencies as per Martin Kühn. Results from lumping are shown in figure 3.3 and 3.4.

![Figure 3.3: Lumping of correlated wind speed and wave height](image1)

![Figure 3.4: Lumping of correlated wave height and peak period](image2)

3.2.3. Results

Finally, the polynomial equation obtained in figure 3.3 and 3.4 is used to find wave peak period corresponding to the significant wave height obtained with respect to the wind speed class. Results are shown in table 3.1 and figure 3.5.
3.2. Martin Kühn’s simplified damage equivalent approach

It has to be noted that the wave height lumping method will show large difference for variations in S-N curve slope $m$. Hence, the fatigue damage calculated by using this lumping method will largely depend on the slope of S-N curve. Improvements in the accuracy for all damage equivalent lumped waves can be achieved by an individual derivation for each different S-N curve slope $m$ of the relevant details. It is also worth mentioning that all the assumptions made in M. Kühn’s approach holds true only for quasi-static response.
3.3. Marc Seidel’s spectral energy approach

3.3.1. Introduction
Marc Seidel proposes an approach for lumping of scatter diagram in a way that both the quasi-static contribution and the dynamic (resonant) contribution is captured.

Quasi-static lumping: Equivalent significant wave height
Wave loads on substructure members have a quasi-static effect, i.e. they cause internal forces in the members where they apply. From Morison’s equation description of wave loads, it can be established that wave induced forces are proportional to the water particle acceleration for inertia term and the square of water particle velocity for the drag term. Since, water particle acceleration and velocity are linearly dependent on wave amplitude for linear waves, it follows that quasi-static fatigue loads can be assumed to be proportional to $H^\lambda$. Where $\lambda$ is equal to 1 if the wave loading is inertia dominated and 2 if the wave loading is drag dominated.

An equivalent wave height can therefore be computed for each wind speed by using equation 3.3.

$$H_{s,eq} = \left( \frac{\sum_n H_{s,n}^\lambda * p(n)}{\sum_n p(n)} \right)^{\frac{1}{\lambda}}$$

Where, $p(n)$ = Probability for sea state n with $H_{s,n}$ and $V_{w,n}$ in wind speed vs. wave height scatter diagram.

$m$ = negative inverse slope of the S-N curve.

Dynamic (resonant) lumping: Equivalent peak period
Wave loading induces global dynamic excitation all over the structure which is often dominated by the structural response in the first mode. For slender structures, like monopiles, this is often the dominant effect. For stiff structures, like jackets, this effect may be negligible. For the current exercise i.e. for a jacket, dynamic effect is considered for lumping.

M. Seidel has shown in his paper [34] that damage or dynamic fatigue loads are proportional to the square root of spectral wave energy at the first natural frequency. Hence, his approach is based on the idea of weighting on the basis of the spectral value at the natural frequency. Equation 3.4 is used to derive equivalent spectral energy of the wave spectrum at first natural frequency.

$$\sqrt{S_{\zeta\zeta}(\omega_0)_{eq}} = \left( \frac{\sum_n \left[ S_{\zeta\zeta}(H_{s,n}|T_{p,n}|\omega_0) \right]^m * p(n)}{\sum_n p(n)} \right)^{\frac{1}{m}}$$

Where, $S_{\zeta\zeta}(H_{s,n}|T_{p,n}|\omega_0)$ is the spectral value depending on wave height, peak period and first natural frequency which has to be determined for each entry in the wave height vs. peak period scatter matrix.

As the gradient of the wave spectra is high adjacent to the peak period, ideally refinement of peak period by a factor of 10 is recommended by means of a spline interpolation. The said interpolation is performed only for peak period closer to the first natural frequency in the present scenario. An equivalent spectral value is determined for the respective wave height row of the scatter diagram. Based on the equivalent spectral value and the significant wave height, the equivalent peak period can be recalculated. This ensures that the target value for the spectral energy at the natural frequency is achieved.

3.3.2. Application
First, quasi-static lumping is performed to determine equivalent significant wave height with respect to each wind speed class. In order to apply the quasi-static lumping equation, probability of each sea-state in wind speed vs wave height scatter diagram is determined from figure 3.1 and 3.2.

For the jacket structure, based on some preliminary calculations, it was found that drag is the dominant part of the hydrodynamic load. Hence, $\lambda$ value is chosen as 2 in this case.
S-N curve slope m equal to 3 is used for this exercise. Keeping that in mind, lumping of wind speed and wave height is established and the result is shown in figure 3.6.

![Figure 3.6: Lumping of correlated wind speed and wave height](image)

Next, equivalent peak period is calculated based on dynamic (resonant) lumping approach. As mentioned before, M. Seidel recommends refinement of peak period by a factor of 10 by means of a spline interpolation. Spline interpolation is performed near the fundamental period of structure. To determine spectral energy at first natural frequency, JONSWAP spectrum formula is used as per equation B.2 from EN 61400-3 [4].

Based on the dynamic approach, equivalent spectral energy is calculated which is then used to back-calculate equivalent peak period. Figure 3.7 shows the lumped wave height and peak periods.

![Figure 3.7: Lumping of correlated wave height and peak period](image)

### 3.3.3. Results

The polynomial equation of order 6 obtained from previous step is used to find equivalent peak period corresponding to the equivalent significant wave height obtained with respect to each wind speed class. Results for the same are shown in figure 3.8 and table 3.2.
3.3. Marc Seidel’s spectral energy approach

There are few drawbacks while using this method. First is the constraint of using single S-N curve slope $m$. In general bilinear S-N curves are used for welded details with values of $m=3$ and $m=5$. Using a single S-N curve slope as the representative value might underestimate and/or overestimate lumped values. Hence, there will be variations in the lumped wave height and peak period values while using different slope $m$.

Moreover, author fails to recommend which gamma parameter values should be used when back-calculating equivalent peak period from equivalent spectral energy at first natural frequency. For this exercise, an average value of gamma for all sea-states with non-zero probability is used. Again, it is worth mentioning that different choice of gamma will result in slightly different lumped values.

### Table 3.2: Lumping using M. Seidel’s approach

<table>
<thead>
<tr>
<th>Wind speed $V_w$ [m/s]</th>
<th>Equivalent Significant wave height $H_{s_eq}$ [m]</th>
<th>Equivalent Peak Period $T_{p_eq}$ [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1.14</td>
<td>4.38</td>
</tr>
<tr>
<td>4</td>
<td>1.24</td>
<td>4.40</td>
</tr>
<tr>
<td>6</td>
<td>1.28</td>
<td>4.42</td>
</tr>
<tr>
<td>8</td>
<td>1.47</td>
<td>4.55</td>
</tr>
<tr>
<td>10</td>
<td>1.70</td>
<td>4.82</td>
</tr>
<tr>
<td>12</td>
<td>2.02</td>
<td>5.35</td>
</tr>
<tr>
<td>14</td>
<td>2.46</td>
<td>6.26</td>
</tr>
<tr>
<td>16</td>
<td>3.01</td>
<td>7.42</td>
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<tr>
<td>18</td>
<td>3.51</td>
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<td>22</td>
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<td>9.05</td>
</tr>
<tr>
<td>24</td>
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<td>9.06</td>
</tr>
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<td>5.15</td>
<td>9.02</td>
</tr>
<tr>
<td>28</td>
<td>6.19</td>
<td>8.46</td>
</tr>
<tr>
<td>30</td>
<td>6.56</td>
<td>7.88</td>
</tr>
</tbody>
</table>
3.4. KCI lumping method

3.4.1. Introduction

Lumping approach used in this case is fairly simple. To define the associated wave height with respect to a specific wind speed, the wave height with the highest probability of occurrence has been used from the available scatter diagram (refer figure 3.1). And the wave period has been defined based on weighted averaging over probabilities of each wave height bins from wave height vs. peak period scatter diagram (refer figure 3.2).

\[ H_{s,j} = H_{s,i} \text{ corresponding to maximum } p_{i,j} \]  
\[ T_{p,i} = \frac{\sum_j p_{i,j} T_{p,j}}{p_i} \]  

3.4.2. Results

Based on the obtained wave height and wave period using equations 3.5 and 3.6, relationship is established using best fit polynomial trend. Figure 3.9 and table 3.3 illustrates the lumped wind and wave data.

![Lumping of Wind-Wave data: KCI](image)

Figure 3.9: Lumping of correlated wind and wave data using KCI's method

<table>
<thead>
<tr>
<th>Wind speed Vw [m/s]</th>
<th>Wave height Hs [m]</th>
<th>Peak period Tp [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.25</td>
<td>3.99</td>
</tr>
<tr>
<td>4</td>
<td>0.25</td>
<td>3.99</td>
</tr>
<tr>
<td>6</td>
<td>0.75</td>
<td>5.05</td>
</tr>
<tr>
<td>8</td>
<td>0.75</td>
<td>5.05</td>
</tr>
<tr>
<td>10</td>
<td>0.75</td>
<td>5.05</td>
</tr>
<tr>
<td>12</td>
<td>1.25</td>
<td>6.01</td>
</tr>
<tr>
<td>14</td>
<td>1.75</td>
<td>6.73</td>
</tr>
<tr>
<td>16</td>
<td>1.75</td>
<td>6.73</td>
</tr>
<tr>
<td>18</td>
<td>2.25</td>
<td>7.45</td>
</tr>
<tr>
<td>20</td>
<td>3.25</td>
<td>8.56</td>
</tr>
<tr>
<td>22</td>
<td>4.25</td>
<td>9.42</td>
</tr>
<tr>
<td>24</td>
<td>4.25</td>
<td>9.42</td>
</tr>
<tr>
<td>26</td>
<td>4.75</td>
<td>9.89</td>
</tr>
<tr>
<td>28</td>
<td>6.75</td>
<td>10.29</td>
</tr>
<tr>
<td>30</td>
<td>6.75</td>
<td>10.29</td>
</tr>
</tbody>
</table>
3.5. Example method assessment

3.5.1. Introduction
This section introduces an example of the investigated alternative wind-wave lumping methods. The quality of the lumped wave parameters from previously introduced existing methods are assessed on the basis of resulting hydrodynamic fatigue damages over wind speed and are compared with the reference results. Brief explanation of steps involved to obtain reference hydrodynamic fatigue damage is given in the next section.

Emphasis in this section is on the following lumping methods for wind-wave scatter diagrams:

- KCI’s method which is similar to probability based averaging
- Martin Kühn’s damage equivalent approach
- Marc Seidel’s approach

Hydrodynamic fatigue damages over wind speed are calculated for OWT configuration which is based on Base model with jacket substructure at 52.24m water depth and first eigenfrequency of $f_0 = 0.239 \text{Hz}$. Aerodynamic load contribution is neglected for this exercise. The dynamic load calculations are performed with damping of 2%, while aerodynamic damping is ignored. Hydrodynamic fatigue damages are assessed near mudline, interface and an intermediate location as shown in the figure 3.10. Furthermore, the derived lumped sea-state parameters over the whole wind speed range with a non-zero probability of occurrence are shown in the figure 3.10. 1-10 min seed with time step of 1 s is used for the wave time series simulation in time domain.

![Figure 3.10: Damage location on OWT support structure and lumped sea state parameters](image)

3.5.2. Reference hydrodynamic fatigue damage
Computing fatigue analysis on OWT structure for full wave climate conditions will be very time consuming. Since this is not the main objective of this section, a faster but precise way is resolved for comparison purpose only.
First a unit damage scatter matrix, $D^*$ is obtained by calculating hydrodynamic fatigue damages for unit probability, corresponding to all the sea-states with non-zero probability of occurrence in the wave height vs. peak period scatter diagram. This comprises of 113 sea-states in total. Next, actual damage scatter matrix, $D$ is determined by multiplying individual unit damages with their respective probability of occurrences in the wave height vs. peak period scatter diagram. Furthermore, by using probability based averaging, a representative damage, $D_{\text{rep}}$ is determined corresponding to each row which belongs to different significant wave height. Finally, each representative damage is multiplied with the probability of occurrence corresponding to its significant wave height in the wind speed vs. wave height scatter diagram. Cumulative value for each wind speed bin is calculated which represents the equivalent damage, $D_{\text{SCD}}$. Steps acquired for the calculation of reference fatigue damage is also illustrated as a flow chart in figure 3.11.

Again, it has to be underlined that in no way, this reference damage calculation will result in same damage as that from full met-ocean conditions. In this calculation, one representative damage is assumed for each significant wave height. This lumping of damages is done based on the probability distribution over the peak period. Theoretically, this is also an establishment of lumped wind-wave data. Instead of calculating one wave height and peak period for a mean wind speed, an equivalent damage representing a sea-state is calculated for each wind speed. Hence, lumping of sea-states is performed while calculating reference damage which will introduce certain error in the final results. Please note that this is approximate representation of hydrodynamic fatigue damage for full scatter diagram, and is a good start for comparison study.

### 3.5.3. Results and assessment

The individual and cumulative hydrodynamic fatigue damages are calculated at 3 locations within the OWT. Together with the probability distribution, fatigue damages as normalized values are shown in figure 3.12 for entire range of wind speeds. The following denotations are followed in this section:

- $D_{\text{SCD}} =$ Reference hydrodynamic fatigue damages based on section 3.5.2
• D\textsubscript{KCI} = Hydrodynamic fatigue damages from lumped sea-state based on KCI method
• D\textsubscript{MK} = Hydrodynamic fatigue damages from lumped sea-state based on Martin Kühn method
• D\textsubscript{MS} = Hydrodynamic fatigue damages from lumped sea-state based on Marc Seidel method

From the results in figure 3.12, it can be observed that the KCI averaging method and Martin Kühn approach heavily underestimates the hydrodynamic fatigue damages over the entire wind speed range at all the locations. The largest difference in both KCI and MK approach is observed near interface location with an underestimation of total hydrodynamic fatigue damage by 98.9% and 98% respectively.

M. Seidel’s method yields reasonably good results near mudline in comparison to the reference results. It overestimates total hydrodynamic fatigue damage by 5.9%, though individual fatigue damages over the wind speed range is not very accurate. From 0 to 14 m/s wind speed, Seidel’s approach overestimated fatigue damage, while from 16 m/s onwards, fatigue damage is underestimated in comparison to the reference results. This is true for all locations. Seidel’s approach manages to predict good results in terms of total hydrodynamic fatigue damages near mudline and interface location with an error margin of ±6%. Although
at intermediate location, it largely overestimates the total fatigue damage by 29% which could lead to an over-conservative jacket design.

From the results obtained, it is very clear that KCI averaging method and Martin Kühn approach of wind-wave lumping does not incorporate dynamic part of the response in their assumptions. Hence, these two methods are not explored furthermore. To substantiate the results obtained through Marc Seidel approach a separate frequency domain fatigue analysis (not included in this report) is performed on the structure only for hydrodynamic loads. This is done to extract information regarding location specific transfer function which will be used to characterize typical behavior of location specific hydrodynamic response for a given sea-state.

Transfer function relating to stress and wave height over a frequency range is extracted for three locations, i.e. at A004 (near mudline), A010 (intermediate) and A013 (near interface). For better explanation, transfer function is divided into five zones, namely Z1, Z2, Z3, Z4 and Z5 (refer figure 3.13). Based on the hydrodynamic stress response spectrum obtained from the frequency domain analysis (for all sea-states), stress response behavior is recorded for each zone depending on the wave peak period. Findings of such analysis is presented in subsequent sections for all the three locations and the distribution of hydrodynamic fatigue damage is explained simultaneously.

**Location: A004 (near mudline)**

Figure 3.13 shows transfer function of stress per unit wave height for joint A004 (near mudline). The transfer function shows dynamic amplification at first and second bending mode with eigenfrequencies $f_1 = 0.239\,\text{Hz}$ and $f_3 = 0.776\,\text{Hz}$ respectively. It is deduced from the transfer function that in this case, for a stiff structure like jacket, dynamic response due to $2^{nd}$ bending mode will be dominating for short wave periods. Five zones have been defined to approximately predict resonating frequencies which will govern the response of the joint
for different wave excitations. Table 3.4 is a summarized version of stress response behavior of joint near mudline from the study based on the response spectrum of all sea-states.

Table 3.4: Stress response behavior of joint near mudline

<table>
<thead>
<tr>
<th>Zone</th>
<th>Resonating frequencies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z1</td>
<td>$f_3$</td>
</tr>
<tr>
<td>Z2</td>
<td>$f_1$, $f_3$</td>
</tr>
<tr>
<td>Z3</td>
<td>$f_1$, $f_3$ &amp; $f_p$ (dominant)</td>
</tr>
<tr>
<td>Z4</td>
<td>$f_p$ (minor contribution from $f_1$)</td>
</tr>
<tr>
<td>Z5</td>
<td>$f_p$ (Quasi-static response)</td>
</tr>
</tbody>
</table>

where:

- $f_1$ = 1st natural frequency corresponding to 1st bending mode in tower fore-aft direction
- $f_3$ = 3rd natural frequency corresponding to 2nd bending mode in tower fore-aft direction
- $f_p$ = wave peak period

To simplify, if a sea-state with wave peak period (or peak frequency) lies within zone Z1, then the 3rd natural frequency will be the resonating frequency for the stress response spectrum of that particular joint. In other words, that member will experience hydrodynamic responses dominated by inertia loads due to 2nd bending mode. Figure 3.14 illustrates an example of response spectrum. Similarly, for a sea-state with long wave peak period falling under zone Z5, the response will be purely quasi-static.

![Response Spectrum Corresponding to Wave with Peak Period Lying in Zone Z1](image)

Based on Marc Seidel’s approach, lumped sea-state parameters per wind speed class is known. So, the wind speeds are classified into the five zones defined in figure 3.13 based on their respective wave peak periods. A quick check reveals that wind speed ranging from 2 to 10 m/s, corresponds to lumped sea-states which fall in zone Z2. For 12 m/s wind speed, lumped sea-state lies within zone Z3 and likewise, for wind speed 14 m/s and onwards, lumped sea-states can be classified within zone Z4 (refer figure 3.15).
Looking at the individual hydrodynamic fatigue damage and cumulative damage in figure 3.12, when compared with reference damage, abnormalities in the estimated damage by Marc Seidel’s method is found for wind speeds between 10 and 20 m/s. By observing the transfer function in figure 3.13 in combination with the response behavior established in table 3.4, the erratic performance in fatigue damage is explained. For 10 m/s wind speed which falls in zone Z2, dynamic part of response is dominating at 1st and 3rd natural frequency i.e. 1st and 2nd bending mode. Since, Marc Seidel’s approach is based on the assumption that damage is directly proportional to the square root of spectral wave energy at first natural frequency. It does not consider resonance from higher modes such as the 2nd bending mode of the structure. This explains the difference in hydrodynamic fatigue damage for 10 m/s wind speed.

Similarly, for 12 m/s wind speed which falls in zone Z3, both dynamic ($f_1$ & $f_3$) and quasi-static ($f_p$) response are significant with quasi-static part being dominant at peak frequency of the lumped sea-state. Again, the influence of resonance due to 2nd bending mode is not accounted in wind-wave lumping by M. Seidel which results in different fatigue damage.

Lastly, for wind speed 14 m/s and onwards, the response is mainly quasi-static. Whereas, M. Seidel’s lumping methodology of predicting equivalent peak period is purely based on the assumption that resonant effect of 1st mode is dominating. Additionally, for quasi-static case, number of load cycles per time is inversely proportional to wave period which linearly increases the fatigue damage. Hence, a smaller equivalent peak period can cause higher fatigue damage.

Overall, cumulative hydrodynamic fatigue damage near mudline shows good result in comparison to the reference hydrodynamic fatigue damage. This is because the 1st bending mode has a significant contribution to the stress response of the joint for the entire wind speed distribution, which is successfully captured in Marc Seidel’s approach.

**Location: A013 (near interface)**

Similar to the location near mudline, a summary of stress response behavior and the transfer function of joint A013, near interface elevation is shown in table 3.5 and figure 3.16.

<table>
<thead>
<tr>
<th>Zone</th>
<th>Resonating frequencies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z1</td>
<td>$f_3$</td>
</tr>
<tr>
<td>Z2</td>
<td>$f_1$ &amp; $f_3$</td>
</tr>
<tr>
<td>Z3</td>
<td>$f_1$, $f_3$ &amp; $f_p$ ($f_1$ &amp; $f_3$ dominant)</td>
</tr>
<tr>
<td>Z4</td>
<td>$f_p$ &amp; $f_1$ ($f_p$ dominant)</td>
</tr>
<tr>
<td>Z5</td>
<td>$f_p$ (Quasi-static response)</td>
</tr>
</tbody>
</table>
3.5. Example method assessment

From the hydrodynamic fatigue damage results obtained in figure 3.12, for interface location, Marc Seidel’s approach fails to produce good results individually per wind speed class. By observing the transfer function along with the stress response behavior of the interface joint, reasoning can be established for the deviation in fatigue damage. For wind speeds ranging from 2\(\text{m/s}\) to 10\(\text{m/s}\) which falls in zone Z2, dynamic part of response is dominating at 1\(^{st}\) and 3\(^{rd}\) natural frequency i.e. 1\(^{st}\) and 2\(^{nd}\) bending mode. The same reason for deviation in hydrodynamic fatigue damage is valid here as well since, Marc Seidel’s approach does not consider resonance at higher modes such as the 2\(^{nd}\) bending mode of the structure.

In the same way, for 12\(\text{m/s}\) wind speed which falls in zone Z3, both dynamic (\(f_1\) \& \(f_3\)) and quasi-static (\(f_0\)) response are significant with dynamic response being the dominant one. Again, the influence of 2\(^{nd}\) bending mode is not accounted in wind-wave lumping by M. Seidel which results in different fatigue damage.

For 14\(\text{m/s}\) wind speed, both quasi-static (\(f_0\)) and dynamic (\(f_1\)) response are governing. Failure to capture the correct quasi-static response leads to different fatigue damage.

Lastly, for wind speed 16\(\text{m/s}\) and onwards, the response is mainly quasi-static with small dynamic contribution at the first natural frequency. Same as the case of mudline, since the response here is dominated by quasi-static, it contradicts with the assumptions made in Marc Seidel’s approach and thus underestimates total response.

Generally, the cumulative hydrodynamic fatigue damage near interface shows good result in comparison to the reference hydrodynamic fatigue damage. This is again because the 1\(^{st}\) bending mode has a significant contribution to the stress response of the joint for the entire wind speed distribution, which is successfully captured in Marc Seidel’s approach.

**Location: A010 (intermediate)**

Last of all, fatigue damage results for intermediate location within the substructure is analyzed for the lumped wind-wave data established by Marc Seidel’s approach. First, same
as above, a summary of stress response behavior and the transfer function of joint A010 at intermediate elevation is shown in table 3.6 and figure 3.17.

Table 3.6: Stress response behavior of joint at intermediate elevation

<table>
<thead>
<tr>
<th>Zone</th>
<th>Resonating frequencies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z1</td>
<td>( f_3 )</td>
</tr>
<tr>
<td>Z2</td>
<td>( f_1 ) &amp; ( f_3 )</td>
</tr>
<tr>
<td>Z3</td>
<td>( f_p ) (Quasi-static response)</td>
</tr>
<tr>
<td>Z4</td>
<td>( f_p ) (Quasi-static response)</td>
</tr>
<tr>
<td>Z5</td>
<td>( f_p ) (Quasi-static response)</td>
</tr>
</tbody>
</table>

![Figure 3.17: Transfer function of stress per unit wave height for joint A010 (intermediate)](image)

From the results in figure 3.12, at intermediate location, Marc Seidel's method largely overestimates the total fatigue damage by 29%. Such large difference can be explained by observing the transfer function. From figure 3.17, it can be seen that there is negligible dynamic amplification at the first natural frequency. And since, Seidel's method uses 1\( ^{st} \) eigenfrequency of the structure in the calculation of equivalent peak period, it assumes that damage is directly proportional to dynamic response due to 1\( ^{st} \) bending mode. In this case, since, the dynamic response due to 1\( ^{st} \) bending mode is not significant, the hydrodynamic fatigue damage is overestimated.

It can also be seen from table 3.6 that for wind speed ranging from 2\( m/s \) to 10\( m/s \) (zone Z2), dynamic response due to 1\( ^{st} \) and 2\( ^{nd} \) bending mode is governing. While for 12\( m/s \) wind speed and onwards (zone Z3 and Z4), only quasi-static response is recorded. Thus, validating the large difference observed in the estimated hydrodynamic fatigue damage at the intermediate location.
3.5. Example method assessment

Synopsis

The interface is typically located well above the submerged zone and therefore experiences hydrodynamic responses dominated by dynamics (inertia loads). Whereas, mudline location experiences hydrodynamic responses with both contributions, dynamic (inertia load) and static (direct hydrodynamic forces on the structure). Due to inaccurate estimation of dynamic response, we see large differences in cumulative hydrodynamic fatigue damage at intermediate and interface location, than at the mudline.

From the results and observations gained so far, it can be concluded that for the waves with long wave period which results in a response mainly dominated by quasi-static part, hydrodynamic fatigue damage is underestimated by Marc Seidel's approach. This is because Marc Seidel assumes the dynamic response due to 1\textsuperscript{st} bending mode to be dominating. Thus, undermining the quasi-static response of the structure. Alternatively, for waves contributing to both quasi-static and dynamic part of the response, it is not that straightforward to state if the fatigue damage will be underestimated or overestimated. For example, a decreased peak period corresponds to a decreased mean wave period $T$ leading to an increased number of load cycles per time, $n = 1/T$ which linearly increases the fatigue damage. On the other hand, decreasing wave period $T$ can lead to a decreasing dynamic response if the dynamic amplification factor also reduces.

3.5.4. Sensitivity analysis

To validate the large differences observed in the cumulative hydrodynamic fatigue damage between different lumping methods (refer figure 3.12b, 3.12d and 3.12f), a sensitivity analysis is done. In order to verify that the huge differences are not because of small (inadequate) damping, damage was calculated at joint A004, near mudline for wind speed class of 16m/s with an increased damping of 5%. Increase in damping ratio should eventually reduce the induced hydrodynamic fatigue damage in the structure, which might lead to a smaller error when compared to the reference case. Results of sensitivity analysis is shown in the figure 3.18.

![Figure 3.18: Normalized hydrodynamic fatigue damage with 2% and 5% damping for 16m/s wind speed](image)

It is clear from the results that an increase in damping leads to reduction in fatigue damage of the structure. This holds true for all approach. Percentage of reduction in fatigue damage is different for different approach, ranging from 0.5% to 17.5%. It is not possible to comment on the relationship of damping and fatigue damage at this point and is outside the scope of this thesis.

From figure 3.18, the general trend with increased damping can be observed but it does
not reflect on whether the differences in cumulative fatigue damage with different approach reduce or not and even if they do, then to what extent. To substantiate that, hydrodynamic fatigue damage with 5% damping is calculated using all the different wind-wave lumping approaches along with reference damage. Results from this analysis are presented in figure 3.19 at joint location A004, near mudline.

![Figure 3.19: Distribution of cumulative hydrodynamic fatigue damage with increased damping](image)

After a quick comparison of results in figure 3.19 with figure 3.12b, it can be established that the distribution of cumulative hydrodynamic fatigue damage remains the same with a slight damage reduction. Increase in damping does improve Marc Seidel’s result now to 3% error but for Martin Kühn and KCI method, error margin still remains big (81% and 91%). Nonetheless, this analysis proves that the error margin in between different approach remains comparable with different damping percentage. Thus, the above sensitivity analysis validates the obtained error margin. Although, it must be highlighted that a more detailed damping analysis is required to correctly establish correlation between damping and fatigue damage of the structure.

### 3.6. Conclusion

Lumping of correlated wind-wave data for the fatigue analysis of OWT has a direct influence on the accuracy of calculated design loads and fatigue damages and therefore has a direct consequence on the safety and economics of offshore wind turbine structures. The relevant design standards lack any guidance on the establishment of lumping methodology. There is a need for a method which allows for a good preservation of hydrodynamic fatigue damages at all locations in OWT.

Correlation of wind and wave data does not depend on structure properties. But, the general criteria for lumping of sea-states is to achieve approximately the same damage from lumped sea-state as from full scatter diagram. Hence, these lumping methods do depend on the response of the structure. The existing lumping methods studied in this thesis are more practical for monopiles and are based on either quasi-static assumptions or on the notion that damage is directly proportional to dynamic response due to 1$^{st}$ bending mode. Since, 1$^{st}$ bending mode is dominant mostly in case of slender structures like monopiles, these assumptions do not necessarily hold true for lattice type structure. Hence, the existing lumping methods are exercised for jacket support structure.

From the results obtained, it can be concluded that lumping methods for correlated wind-wave data suggested by KCI and Martin Kühn both fail to estimate the dynamic part of the response which results in an underestimation of fatigue damage. However, a better overall result is observed at mudline and interface with Marc Seidel’s approach as it captures both quasi-static as well as dynamic (resonant) response. At the same time, the distribution of
hydrodynamic fatigue damage is not preserved at intermediate location which leads to larger error in fatigue damage. Since, only 1-10 minute seed with rather large time step of 1 sec is used to simulate wave time series for this fatigue analysis, the time series will have low resolution. Bad resolution will also have an effect on the fatigue damage and will introduce certain error. Increasing the time series resolution by considering more number of seeds and a smaller time step should reduce the error or uncertainty introduced in the estimated fatigue damage. However, due to time constraint, this has not be exercised in this thesis.

M. Seidel’s approach is based on the idea of weighing on the basis of the spectral value at the first natural frequency (dominant first mode). In Jacket type structure, higher modes can also have a significant contribution to the response of the structure. Because Marc Seidel’s approach is based on the assumption that damage is directly proportional to the dynamic response due to 1st bending mode, the established lumping does not provide good results at all location.

3.7. Recommendation

From the studies performed on the existing lumping methodologies, it is very clear that further research is required to establish a method which can produce uniform results for all wind speeds and at every location within the structure. Based on the assessment, M. Seidel’s approach stands out from other existing lumping methods. Hence, further investigation to improve the results obtained through his approach is recommended.

Using modal analysis, different modes can be found which has a significant mass participation. Since, mass participation factor represents the amount of system mass participating in a particular mode, it provides a measure of the energy contained within each resonant mode. Thus, a mode with large mass participation is usually a significant contributor to the response of the structure. If a way can be found to consider the effects of all the significant modes and combine them with Marc Seidel’s method, lumping of correlated wind-wave data can be improved significantly.

Another promising damage equivalent wind wave correlation method is proposed by Patrik Passon [31, 32]. This new method claims to preserve the fatigue damage at all locations. Patrik Passon’s method was investigated for the present scenario but due to the lack of detail metocean data, lumped sea-state parameters could not be obtained. Further investigation is recommended to delve deeper into the lumping techniques of correlated wind-wave data.
Reduction of design load cases (DLCs)

4.1. Introduction

According to IEC 61400-3 [4], in total there exists 27 ultimate strength design load cases (DLCs) and 7 fatigue DLCs. While designing an offshore wind turbine (OWT) support structure, all the above mentioned DLCs must be analyzed to allow it to successfully withstand different loading conditions.

Each of these DLCs comprise of many load combinations which can consist of wind and wave directions, yaw misalignments, wind speeds, different seeds, current model and design water depth. In general, a foundation designer can end up with more than 15000 load combinations corresponding to all the DLCs. Analysis of an OWT support structure with +15000 load combinations in time domain requires enormous computing power and simulation time. Design cycle of an OWT support structure typically goes through at least 3 to 4 iterations. Thus, simulating +15000 load combinations for multiple iterations becomes very time consuming, cumbersome and are not very economical.

Hence, there is a need to reduce the computational effort required for a typical design cycle of OWT support structure. Reduction of computational time can be achieved by reducing the number of DLCs for which the support structure actually needs to be analyzed.

In this thesis, load combinations within various DLCs and wind time series data has been provided by MHI Vestas which was used in Moray Firth – Pre-Feed project [30] by KCI. It consists of the following DLCs:

- DLC 1.2 – Normal power production
- DLC 1.3 – Power production with extreme turbulence
- DLC 1.6 – Power production with severe sea state
- DLC 3.2 – Start up with Extreme operating gust
- DLC 6.1 – Parked (standing still or idling)

For this thesis, main focus is on fatigue analysis, which is why DLC 1.2 is investigated for reduction analysis. In this design situation, the OWT is running and connected to the electric load. As per IEC 61400-3 [4], DLC 1.2 embodies the requirements for loads resulting from atmospheric turbulence (NTM – normal turbulence model) and stochastic sea states (NSS – normal sea state) that occur during normal operation of an OWT throughout its lifetime. A single value of significant wave height is considered for each relevant mean wind speed from wind-wave correlation. The significant wave height, peak spectral period, wave direction and water level for each normal sea state are considered together with the associated mean wind speed based on the joint probability distribution.

It has to be highlighted that the data obtained from Vestas is already a reduced version of design load cases with in total 1524 load combinations. For DLC 1.2, nacelle and wave
direction is considered to be in the same direction with a yaw misalignment of ±6. To ensure that the number and resolution of the time series considered are sufficient to account for the fatigue damage associated with the full long term distribution of metocean parameters, 12-10 minute seeds are created per direction with 6 directions in total within each wind speed class. For example, within 4m/s wind speed class, nacelle and wave directions considered are 0°, 30°, 60°, 90°, 120° and 150°. Each of the mentioned 6 direction consists of 12-10 min seeds. Hence, in total, 72 load combinations (or seeds) are considered within one wind speed class. A sample load case table of DLC 1.2 for 4m/s wind speed is shown in figure 4.1.

<table>
<thead>
<tr>
<th>File name</th>
<th>Wind speed</th>
<th>Turbulence intensity</th>
<th>Wind seed</th>
<th>Wind dir.</th>
<th>Nacelle dir.</th>
<th>Yaw misalignment</th>
<th>Wave seed</th>
<th>Wave dir.</th>
</tr>
</thead>
<tbody>
<tr>
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<td>0.189</td>
<td>1</td>
<td>-6</td>
<td>0</td>
<td>6</td>
<td>0.25</td>
<td>3.99</td>
</tr>
<tr>
<td>12_A00_12B</td>
<td>4</td>
<td>0.189</td>
<td>2</td>
<td>6</td>
<td>0</td>
<td>-6</td>
<td>0.25</td>
<td>3.99</td>
</tr>
<tr>
<td>12_A00_12C</td>
<td>4</td>
<td>0.189</td>
<td>3</td>
<td>-6</td>
<td>0</td>
<td>6</td>
<td>0.25</td>
<td>3.99</td>
</tr>
<tr>
<td>12_A00_12D</td>
<td>4</td>
<td>0.189</td>
<td>4</td>
<td>6</td>
<td>0</td>
<td>-6</td>
<td>0.25</td>
<td>3.99</td>
</tr>
<tr>
<td>12_A00_12E</td>
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<td>0.189</td>
<td>5</td>
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<td>0</td>
<td>6</td>
<td>0.25</td>
<td>3.99</td>
</tr>
<tr>
<td>12_A00_12F</td>
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<td>0.189</td>
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<td>6</td>
<td>-6</td>
<td>0</td>
<td>0.25</td>
<td>3.99</td>
</tr>
<tr>
<td>12_A00_12G</td>
<td>4</td>
<td>0.189</td>
<td>7</td>
<td>6</td>
<td>-6</td>
<td>0</td>
<td>0.25</td>
<td>3.99</td>
</tr>
<tr>
<td>12_A00_12H</td>
<td>4</td>
<td>0.189</td>
<td>8</td>
<td>-6</td>
<td>0</td>
<td>6</td>
<td>0.25</td>
<td>3.99</td>
</tr>
</tbody>
</table>

A new hypothesis is proposed in the next section for the reduction of DLCs based on the available wind time series. Key idea is to guessestimate fatigue damage by just analyzing the loads acting on the structure. The proposed analysis for reduction of DLCs is performed on wind speed class 4m/s. Later on, for validation purpose, detailed fatigue analysis and reduction analysis is executed on wind speed class 10, 16 & 24m/s.

### 4.2. Analysis Philosophy

The rudimentary approach that is followed during the reduction of DLCs is to identify seed numbers that can correspond to the maximum (critical) fatigue damage. By identifying critical seeds, we can essentially sort seeds in an order relevant to the incurred fatigue damage.
This way, seeds resulting in minimum fatigue damage can be ignored and fatigue analysis can be performed only on the seeds which are more critical to the structure.

Since wave parameters remain same for every mean wind speed (from lumping) and jackets are hydrodynamically less sensitive, the resulting wave response will not be dominant. Hence, the current reduction is proposed solely based on wind load time series.

The following assumptions are made for this reduction analysis:

- Wind loads are governing for fatigue damage when compared to wave loads
- Response is assumed to be linear-quasi-static with a constant turbulence intensity. Therefore, the stress ranges are considered proportional to the standard deviation of wind speed and thus mean wind speed.

### 4.2.1. Analysis steps

Reduction of DLCs is proposed in the following two steps:

- Identify critical seeds which will result in maximum fatigue damage for the selected joint
- Devise a selection criterion to optimally choose different seeds which will result in fatigue damage with least error

To identify critical seeds, an approach very similar to the methodology used in estimation of fatigue damage is proposed. To calculate fatigue damage by Palmgren-Miner rule [28], summation is performed on the individual fatigue damage ratios caused by each stress cycle or stress range block. And to obtain stress range, a standardized procedure called Rainflow counting [27] performed on stress time history. Since we don't have a stress time history to begin with, fatigue damage cannot be calculated without a dynamic analysis of the OWT support structure.

For the proposed reduction analysis of DLCs, we don't need to calculate the actual fatigue damages but just an estimation of them. Hence, based on the assumptions made earlier, by performing Rainflow counting on wind load time history, we can calculate load amplitude and the load cycle corresponding to each load time history (seed). Based on linear-quasi-static response, stress cycle will be same as that of load cycle and stress range can be calculated from load range.

To perform rainflow cycle counting, a MATLAB toolbox RAINFLOW developed by Niesony A. [5] is used where the algorithm code has been written according to the ASTM standard [3].

By following the above hypothesis, quasi-static fatigue damage due to wind load corresponding to different seeds can be estimated. As mentioned in section 4.1 we have 72 load combinations or seeds for each wind speed class. 10-step reduction analysis is performed for 4m/s wind speed class as explained in below steps as well as in the flow chart presented in figure 4.2:

- **Step 1:** For each wind speed, 72 seeds are segregated into 6 sections depending on the direction of loading, i.e. for each direction we end up with 12 seeds.
- **Step 2:** By performing time domain simulation, total fatigue damage is calculated for different joints in *Base model*. Based on the damage, seeds are identified for which major number of joints show critical fatigue damage. This will form as a reference for comparison later on in step 8.
- **Step 3:** Rainflow counting is performed by using MATLAB toolbox RAINFLOW [5] on 72 seeds within the chosen wind speed class and load cycle and amplitude is obtained.
- **Step 4:** Wind time history consists of loads in the form of forces and moments in X, Y and Z direction, i.e. FX, FY, FZ, MX, MY and MZ. Example format of wind load input provided by WTM can be referred in Appendix C. Rainflow counting is executed on all 6 loads after which, the load amplitudes are converted to stress amplitudes.
• **Step 4.1:** To convert load amplitude to stress amplitude, a dummy member is selected. In this case, member properties of Joint TW00 (at the interface level) is chosen since, loads are transferred at the interface level. Any other member can be selected as this will not change the main outcome of the analysis. To convert loads to stresses, the following formulas are used:

\[ \sigma_F = \frac{F}{A} \quad \text{and} \quad \sigma_M = \frac{M}{I} \]

• **Step 5:** Using Miner rule of linear damage accumulation, fatigue damage is calculated in MATLAB corresponding to FX, FY, FZ, MX, MY and MZ for each seed.

• **Step 6:** Total fatigue damage is calculated by taking summation of damage corresponding to FX, FY, FZ, MX, MY and MZ for each seed.

• **Step 7:** Lastly, the 12 seeds are sorted in decreasing order of estimated fatigue damage within each direction. Thus, identifying seeds which will result in critical fatigue damage due to wind load. For this analysis, the sorted seeds are termed as *damage sequence/order*.

• **Step 8:** Verification is done to check if the damage sequence from rainflow counting is able to identify the critical seed from reference case.

• **Step 9:** If yes, perform reduction of DLCs by removing non-critical seeds from step 2.

• **Step 10:** Check if the error introduced by reduction analysis is within acceptable range and then apply a safety factor to compensate the error.

![Figure 4.2: Flow chart presenting rainflow reduction analysis](image-url)

Next section presents all the steps and the results obtained by performing the introduced reduction analysis on wind speed class \(4 \text{m/s}\).
4.3. Results
Rainflow counting reduction method is performed on 4m/s wind speed by following 10-step procedure as proposed earlier. Step by step results are shown in this section.

Step 1
As recommended, 72 seeds are compartmentalized into 6 sections corresponding to 6 different directions as shown in table 4.1.

Table 4.1: Seeds corresponding to different wind direction

<table>
<thead>
<tr>
<th>Wind Direction [°]</th>
<th>Seed Number [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1-12</td>
</tr>
<tr>
<td>30</td>
<td>13-24</td>
</tr>
<tr>
<td>60</td>
<td>25-36</td>
</tr>
<tr>
<td>90</td>
<td>37-48</td>
</tr>
<tr>
<td>120</td>
<td>49-60</td>
</tr>
<tr>
<td>150</td>
<td>61-72</td>
</tr>
</tbody>
</table>

Step 2
Based on the results from time domain fatigue analysis of Base model, critical wind seeds are identified out of 12 seeds for every direction. Results for the same are presented in figure 4.3.
4.3. Results

Figure 4.3: Critical wind seeds for every direction

Figure 4.3 shows the number of joints which are critical (having maximum fatigue damage) corresponding to different seeds in each direction. As an example, from figure 4.3a, it can be inferred that 22 joints are having maximum fatigue damage for seed number 1. Similarly, 3 joints shows critical fatigue damage for seed number 7 and few other joints were critical for seed number 3, 6 and 10. Rest of the seeds did not result in critical damage for the chosen joints. From figure 4.3, it is clear that for $0^\circ$, seed number 1 will result in maximum fatigue damage for most joints and should not be ignored. Similar interpretations can be made from other figures in other directions.

Step 3
Next, rainflow counting is executed on the wind load history by using MATLAB toolbox RAINFLOW [5]. Information obtained is in the form of load cycle, amplitude and its mean value. Example of the result obtained from rainflow counting on force in X direction for seed number 1 is shown in figure 4.4.
4.3. Results

Step 4 and 5
Load amplitudes are converted to stress amplitudes for interface level member. For the calculation of expected fatigue damage, S-N curve parameters for T-curve are used with slope $m$ equal to 3. Using the equation 4.1 which is based on the Palmgren-Miner rule (refer equation 2.2.1 of DNVGL-RP-0005 [14]), damage is calculated for each seed.

$$D_{FX} = \sum \left( \frac{N_{FK}}{N_K} \right) \left( \frac{\sigma_{FX}}{\sigma_L} \right)^m$$  \hspace{1cm} (4.1)

where:

- $D_{FX}$: damage accumulated due to wind force in X direction
- $N_{FK}$: number of stress (load) cycles
- $N_K$: Number of cycles for $m_1 = 3$ as per [14] = $1E+6$
- $\sigma_{FX}$: Stress cycle amplitudes (converted from load amplitudes)
- $\sigma_L$: Limiting stress at $10^7$ cycles as per [14] = 52.63Mpa

Step 6
Similarly, fatigue damage due to wind load in other direction is calculated and total fatigue damage for each seed is obtained by taking the summation of damage in all the direction.

$$D = D_{FX} + D_{FY} + D_{FZ} + D_{MX} + D_{MY} + D_{MZ}$$  \hspace{1cm} (4.2)

Step 7
Based on the cumulative damage, seed numbers are sorted in descending order as per the total quasi-static fatigue damage, for each direction. Results with normalized damage are shown in figure 4.5:
4.3. Results

Figure 4.5: Damage sequence obtained based on quasi-static damage calculation

Step 8
Figure 4.5 gives an indication towards the critical wind seeds which can contribute to high fatigue damages. When compared with the reference data (refer figure 4.3), it is observed that damage order is not exactly matching with the critical seed identified in figure 4.3. This is because the estimated fatigue damage is resulting from a static calculation on a dummy member. It fails to undertake the dynamic behavior of OWT support structure. Hence, dynamic response of the structure cannot be mapped in the stress time series. Although, this methodology is able to capture critical wind seed in top 5 seeds of damage order, except for the case with direction 120° (refer Table 4.2).
Table 4.2: Comparison of critical seed with identified damage order

<table>
<thead>
<tr>
<th>Wind Direction [°]</th>
<th>Critical Seed number as per Base model</th>
<th>Identified top 5 critical seed number by Rainflow counting</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
<td>8, 1, 11, 2, 10</td>
</tr>
<tr>
<td>30</td>
<td>15</td>
<td>24, 14, 13, 15, 17</td>
</tr>
<tr>
<td>60</td>
<td>34</td>
<td>32, 34, 33, 28, 25</td>
</tr>
<tr>
<td>90</td>
<td>47</td>
<td>45, 47, 48, 39, 42</td>
</tr>
<tr>
<td>120</td>
<td>56</td>
<td>60, 52, 58, 51, 49</td>
</tr>
<tr>
<td>150</td>
<td>68, 61</td>
<td>61, 64, 68, 65, 67</td>
</tr>
</tbody>
</table>

Based on rainflow counting method, 4 out of 5 times, most critical seed can be identified within the top 5 seeds w.r.t. fatigue damage. This ensures that the seed number which is critical for majority of joints is successfully captured by following the reduction analysis.

**Step 9**

Now that the most critical seed has been identified and the approximate damage order is known, reduction of DLCs can be performed by neglecting the non-critical seed numbers. Combined fatigue damage (wind + wave) of 3 joints has been evaluated for the reduced DLCs. Joints that are considered is A004 (near mudline), A010 (intermediate) and A013 (near interface). Originally, there are 12 seeds per direction. Number of seeds are reduced one at a time until only one is remaining and combined fatigue damage is calculated for each of them.

Combined fatigue damage is known for 12 seeds per direction (total of 72 seeds in 4m/s wind speed class) from time domain fatigue analysis. As a next step, 11 seeds are considered by deserting the last seed number which was identified in figure 4.5, i.e. for 0° - seed number 4, for 30° – seed number 19, for 60° – seed number 30 and so on. Combined fatigue damage is evaluated with only 11 seeds in all the 6 directions and cumulative fatigue damage is calculated for the 3 joints. Similarly, combined fatigue damage is calculated after reducing 1 seed, and this step is repeated till only 1 seed is remaining. Figure 4.6 presents the results in terms of error obtained in total fatigue damage for 4m/s wind speed at 3 joints due to the reduction in number of seeds.

![Figure 4.6: Error (%) in fatigue damage due to reduction of seeds – 4m/s wind speed](image)

First observation is that with the above reduction method, error is always positive i.e.
the calculated fatigue damage is on the conservative side. Second observation which also complies with the standards/codes is that reducing the number of seeds results in reduction in resolution of time series which indeed results in an increasing error. For less number of seeds per direction, i.e. 4 or less, the resolution weakens which results in an unpredictable error pattern.

**Step 10**

For the present case, in DLC 1.2, 16 different mean wind speeds are considered with each wind speed having total of 72 seeds (12 seeds per direction x 6 direction). From figure 4.6, it is observed that if the number of seeds are reduced to 6, even then the introduced error in design fatigue damage is less than 15%, for the three joints. This in itself is a significant reduction in DLCs by 50% which can help save enormous amount of computation time and effort, when simulating in time domain.

Next section introduces validation exercise that is carried out to authenticate the proposed reduction method.

### 4.4. Validation

To validate the results obtained from reduction analysis, detailed fatigue analysis was performed for wind speeds 10, 16 and 24 m/s after which, the above mentioned reduction analysis was implemented on the design load cases. Mentioned wind speeds are selected based on the power curve and the relative distribution of wind speed (refer figure 4.7). 10 m/s wind speed is chosen because it is near to the rated output speed. Similarly, 24 m/s wind speed is selected because it is the close to the cut-out speed. Furthermore, 16 m/s wind speed is randomly chosen in between rated and cut-out wind speed.

![Power curve - Rotational frequency Vs Wind speed](image1.png)

![Relative distribution of wind speed](image2.png)

**Figure 4.7: Power curve of rotor and distribution of wind speed**

For clarity, only final result in terms of “error in design fatigue damage” is presented (refer figure 4.8) for the mentioned wind speed. Same 10-step procedure as defined earlier is followed to obtain these results.
As a general observation, first, there is no definitive trend for the introduced error in design fatigue damage. In case of 4m/s wind speed (refer figure 4.6), error has a clear trend where it keeps on increasing for all the three joints until the resolution is compromised beyond repair. When compared to the results in figure 4.8, error has a rather unpredictable development for different wind speeds.

Secondly, it can also be observed that the reduction of DLCs is introducing negative error. There are two possible reasons for this. In case of 4m/s wind speed, for each direction, i.e. out of 12 seeds there was only one critical seed number which was having maximum fatigue damage for majority of joints. While performing the reduction analysis on 10, 16 & 24m/s wind speed, there were more than one critical seed number per direction, i.e. for every seed there were many joints which had maximum fatigue damage. Refer figure 4.9 for an example of more than one seed being critical. Thus, by following the reduction analysis, it was not possible to identify all the critical seeds per direction. Second reason for negative error is that the identified critical seed number, for which maximum number of joints are having maximum fatigue damage, is not necessarily a critical seed for the chosen three joints.
For 4 m/s wind speed (refer figure 4.6), reducing seed numbers by 50% introduced around +15% error for all the three joints, which is an acceptable error margin. For validation purpose, results obtained from reduction of DLCs on different wind speeds is compared for reduced DLCs with 6 seeds per direction. Refer table 4.3 which provides error (%) in fatigue damage corresponding to reduction of DLCs by 50% evaluated at three joints for the respective wind speeds.

Table 4.3: Error (%) in combined (wave+wind) fatigue damage introduced due to 50% reduction in DLCs

<table>
<thead>
<tr>
<th>Joint</th>
<th>Wind speed class</th>
<th>4m/s</th>
<th>10m/s</th>
<th>16m/s</th>
<th>24m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>A004 (near mudline)</td>
<td>9%</td>
<td>-5%</td>
<td>1%</td>
<td>-1%</td>
<td></td>
</tr>
<tr>
<td>A010 (Intermediate)</td>
<td>12%</td>
<td>-1%</td>
<td>35%</td>
<td>-1%</td>
<td></td>
</tr>
<tr>
<td>A013 (near interface)</td>
<td>13%</td>
<td>5%</td>
<td>2%</td>
<td>-10%</td>
<td></td>
</tr>
</tbody>
</table>

Near rated and cut-out speed, i.e. 10 and 24 m/s, all the three joints shows less than 10% error. This is good when compared to 4 m/s wind speed results. Although, the introduced error is both negative and positive which means that the joint design can be conservative or nonconservative with reduced DLCs. Since, this reduction method fails to maintain a positive error and it is difficult to identify for which wind speed, which joint will have positive/negative error, this makes it even harder to apply a safety factor globally.

Even though for 16 m/s, all the three joints show positive error, intermediate joint (A010) shows a rather high error of 35%. On the other hand, joints near mudline and interface level only show an error of 1% and 2% respectively. This proves that it is not wise to apply one safety factor to all the joints as the error varies with different joint and different wind speed.

In the next section, conclusion is drawn on the reduction analysis based on the results obtained so far.

4.5. Conclusion

As per IEC 61400-3 [4], for better resolution of time series, more number of seeds are required. Hence, while performing any kind of reduction on DLCs, one has to compromise on the resolution which would invariably introduce a certain percentage of error in the final result. To have a viable reduction in DLCs, the only thing that can be ensured is to not have a huge error in the final damage results.

The presented reduction analysis on wind load time series provides good insight on the reduction capabilities of DLCs. It gives an understanding about the extent to which we can reduce the DLCs without compromising too much on the final result and the led error along with it. As a bare minimum, IEC 61400-3 [4] suggests to use at least an hour long time series to be able to capture the stochastic nature of the loads. From the results obtained through reduction analysis, 6 – 10 min seeds (1 hour) results in an acceptable error margin of under 15% for joints near mudline and interface in general, which is in close agreement with the prescribed minimum seed number. Hence, just by analyzing wind load history, approximate estimation can be made towards the critical load cases.

The proposed reduction analysis has some drawbacks. This method does not consider dynamic properties of the OWT support structure. Hence, it fails to account the dynamic response of the structure due to the acting loads. Since, the wind load time history is directly applied at the interface level of the support structure, performing rainflow counting on the wind time series and using Miner’s rule of damage accumulation gives very precise damage sequence for interface joint only. Trying to replicate this damage sequence to other joints in the support structure does not produce good results. This is because the damage sequence which holds true for interface joint will not necessarily be true for other joints in the support structure. For jacket type structure with complex geometry, dynamic response of different joints will be different from each other for a given load time history which will result in
4.6. Recommendation

Following up on the conclusions made earlier, some recommendation for the reduction analysis is advised below:

- If dynamic response of the structure can be accounted for in presented approach, then it can lead to more accurate results. Although, this would mean analyzing the load time series for each interested joint separately. This will make the entire reduction procedure very bulky.

- In the reduction analysis, wave loads were completely ignored for jacket support structure. Monopiles are more sensitive to hydrodynamic loads. Hence, to predict correct dynamic response of the structure, interaction of both wind and wave loads should be considered.

- After the damage sequence has been identified, it is still not clear on how to handpick different seeds to get least error in total fatigue damage. An improved version of selection criteria should be sought out which can lead to minimal damage error.

- This reduction analysis proves that fatigue analysis in time domain requires dynamic simulation for a bare minimum amount of time. That minimum length of time series is necessary for an accurate representation of the stochastic nature of loads. Hence, it is recommended to explore frequency domain method for fatigue analysis of OWT support structure. Frequency domain approach is presented in next chapter.
Fatigue analysis of OWT support structure in frequency domain

5.1. Introduction

There are various fatigue calculation methods proposed in offshore industry. Due to its diversity, it is difficult to standardize one method. It has also to do with the practical and contractual point of view. Usually, there is a division between the offshore contractor and the wind turbine manufacturer which restricts information sharing. Typically, offshore contractor is responsible for the design of foundation and transition piece, while the tower and rotor-nacelle assembly falls under the wind turbine manufacturer. Even though this partition is practically justified, the structure does not feel this interface and will act dynamically from top of the wind turbine to bottom of the foundation pile.

An example of industry practice for a typical design cycle of an OWT support structure is illustrated in figure 5.1. First, the foundation designer estimates the foundation geometry based on preliminary information. Next, the dynamic properties of the support structure in terms of mass, stiffness, damping and load vector matrix is calculated, typically at interface level. These matrices acts as an input to the wind turbine manufacturer who then uses this input at the interface to calibrate wind time series producing aerodynamic loads for each of the wind time series. Finally, the foundation designer receives these aerodynamic loads in the form of wind time series which are used to design and optimize the support structure.
5.2. Frequency domain approach: Single DOF system

For every design cycle, one has to perform transient analysis for up to 15,000 design load cases. Based on the complexity of the support structure and FE software used, dynamic analysis of one load case (600s) in time domain can take up to 1-2 hours. Time domain analysis of more than 15,000 design load case requires considerable computational time and memory storage. Typically, every offshore structure goes through 3 to 4 design loops (refer figure 5.1) due to changes in input and or optimization of support structure.

An efficient way to solve this problem would be to create a new method for fatigue assessment of the support structure in the frequency domain. Frequency domain approach [35] can be executed with the help of traditional methods, software and experience, and it makes the optimization of support structure very easy. Additionally, it offers the advantage of providing the offshore designer with clear information on environmental and structural properties. The biggest gain with frequency domain is the fast computation time which speeds up the design process, making this approach very efficient for use.

In the next section, basics of dynamics and frequency domain method are analyzed for a linear one degree-of-freedom (DOF) system.

5.2. Frequency domain approach: Single DOF system

The frequency domain can be described as simply another domain in which to view a time signal; the x-axis now represents frequency instead of time. To convert a time signal into the frequency domain, we effectively split it up into a number of discrete sinusoidal waves of varying amplitude, frequency and phase. When these are added together they form the original time signal [19].

An elementary system can be represented as a system which when subjected to an external input produces an output. Figure 5.2 shows a graphical illustration of system.

![Figure 5.2: Simple representation of a system](image)

Let the system be defined as a mass-spring-dashpot with harmonically varying external load as input and the displacement of the mass as the output. Refer figure 5.3 for graphical representation. This system has only one degree of freedom which is the displacement $x$.

![Figure 5.3: Single degree of freedom system](image)

Now, the system behavior can be described by its equation of motion:

$$m\ddot{x} + c\dot{x} + kx = F$$  \hspace{1cm} (5.1)
Let the input load be a harmonic with \( F(t) = \hat{F} \cos(2\pi ft) \). Then, for a linear system the output \( x \) will also be harmonic with an amplitude \( \hat{x} \) and a phase angle \( \phi \). The system can be solved analytically. Since, we know that output will also be a harmonic, \( x \) can be assumed to be of the form:

\[
\begin{align*}
  x &= \hat{x} \cos(2\pi ft - \phi) \\
  \dot{x} &= -2\pi f \hat{x} \sin(2\pi ft - \phi) \\
  \ddot{x} &= -4\pi f^2 \hat{x} \cos(2\pi ft - \phi)
\end{align*}
\]

(5.2)  
(5.3)  
(5.4)

By substituting above equations in equation of motion, only two unknowns are left: the amplitude \( \hat{x} \) and the phase angle \( \phi \). Then, the resulting equation can be easily solved by resolving the equation in an in-phase (\( \cos(2\pi ft) \)) and an out-of-phase (\( \sin(2\pi ft) \)) part. Figure 5.4 shows 3 different input signals and their corresponding system’s output.
In order to calculate transfer function of a system, impulse response of the system is calculated. Impulse response of a system is the response when given impulse as input. For example, a system is defined with impulse response \( h(t) \), input \( x(t) \) and output \( y(t) \). In time domain, the relation between input, output and impulse response is given by [11]:

\[
y(t) = x(t) * h(t)
\]

(5.5)

Where, output of a system is equivalent to input convolution with impulse response. Since, convolution in time domain becomes multiplication in frequency domain, the relation in frequency domain becomes:

\[
Y(\omega) = X(\omega)H(\omega)
\]

(5.6)

\[
H(\omega) = \frac{Y(\omega)}{X(\omega)}
\]

(5.7)

Where, \( Y(\omega) \) and \( X(\omega) \) are the Fourier transforms of output and input. And \( H(\omega) \) is called the transfer function of the system which is the Fourier transform of impulse response, also given by ratio of output transform by input transform.

Figure 5.5 depicts the system’s transfer function of displacement per unit force per frequency.

The transfer function provides a direct relationship between input and output for each frequency. It is a characteristic property of the system and does not depend on the input. As the system is fully linear, a random excitation made up from combined multi-harmonic excitation can be transformed into random response harmonics by adding the separate single harmonic responses. Figure 5.6 shows the random input excitation made up by combining signal 1, 2 and 3 and the resulting random response output by combining the separate harmonic responses.
To transform the time series shown in figure 5.6 to the frequency domain, both the random input and the output can be translated into a spectrum through Fast Fourier Transforms (FFT). Similarly, to flip back into time domain, Inverse Fast Fourier Transform can be used.

The transfer function can either be derived analytically or numerically from the equation of motion or by taking the ratio of output over input. Figure 5.8 shows the derivation of transfer function in frequency domain.
5.3. Hybrid frequency domain approach: OWT support structure

Commonly, spectral frequency domain analysis is utilized to calculate fatigue damage where the random loading and response are categorized using Power spectral density (PSD) functions and transfer function. Figure 5.9 illustrates the process of frequency domain fatigue analysis based on PSD. In the next step, using the loading and response PSD, transform function is estimated which creates dependency of responses on the frequencies in particular range. For derivation of transfer function based on PSD, refer figure 5.11. Based on PSD and transform function, it is then possible to define response spectrum corresponding to loading. Lastly, various methods are available for performing fatigue analysis from PSDs. Dirlik method is one of the most widely used method which gives best comparable results with the traditional time domain approaches.

In this thesis, a novel-hybrid frequency domain method is proposed. In this section, application of hybrid frequency domain method for offshore wind turbine fatigue calculation due to wind loads is shown. A general comparison between fatigue results due to wind loads between time domain and frequency domain method is studied.

An overview of steps required to carry out fatigue calculation using hybrid frequency domain method is depicted in figure 5.10. These steps are applicable for a system with a complete separation between turbine and support structure. This method consists of two parts. The primary step is the derivation of location specific transfer function between loading and stress response using impulse response function of the system. Once the transfer functions are determined, stress response spectrum is calculated corresponding to different load input. Using inverse Fast Fourier Transform (iFFT), stress response is transformed back to time domain, which becomes the input of the second part. In the second part, stress response time history is rainflow counted (RFC) to find the stress range variation histogram. Based on material properties, S-N curve is selected which is then utilised along with Palmgren-Miner rule to determine fatigue damage, $D_{fat}$. 

![Figure 5.9: Spectral frequency domain fatigue analysis](image)
5.3. Hybrid frequency domain approach: OWT support structure

5.3.1. Derivation of transfer function

The Power spectral density (PSD) is the most common approach for representation of loading or response in frequency domain [7]. PSD is a normalized density plot describing the mean square amplitude of each sinusoidal wave with respect to its frequency. A typical relation of transfer function using PSD plot is shown in figure 5.11.

![Figure 5.11: Frequency Domain representation using PSD](image-url)
Generally, a time signal can be regenerated by performing Inverse Fourier Transform on the frequency domain results. When working with PSD, this method becomes inappropriate because PSD does not contain any original phase information. Since, the phase information is lost, it becomes difficult to regenerate a statistically equivalent time history. For this reason, PSD representation of frequency domain is not used in this thesis.

Instead, transfer function is derived by using impulse response of the system based on equation 5.7. This way, both the amplitude and the phase information is retained in frequency domain. Impulse response of the system is obtained by performing transient analysis for a unit load at time, \( t=0 \) in a FE software called ANSYS\(^1\).

In this project, wind load which is acting at the interface level is provided by the WTM in the form of 600 seconds time series corresponding to 6 degrees of freedom (refer figure 5.12). Sample wind load time series for a random seed is presented in Appendix C. For the given wind load, fatigue damage is calculated by using time signals in the form of stress. For tubular members, ANSYS is capable of calculating linearized stress output in the form of axial and bending stress components. Maximum stress is then calculated by using superposition principal for combined loads. Figure 5.13 illustrates the form of stress output. For the given input and the desired output, transfer function is calculated and is assessed at three locations within the support structure as shown in figure 5.12.

\(^1\)Note that 3-legged jacket support structure model has been migrated from SACS to ANSYS because SACS is incapable of producing response time series for any dynamic analysis.

Figure 5.12: Form of wind load and interested location within the support structure

Figure 5.13: Form of output stress
In this project, wind load is provided in the form of forces and moments corresponding to different degrees of freedom and the design (maximum) stress is calculated from axial/direct and bending stress components. Hence, bookkeeping becomes rather important in frequency domain as more than one transfer function is derived for a particular location. In total, 18 transfer functions are calculated for one location. Number of transfer functions as per the input and output is illustrated in figure 5.14.

Using the principal depicted in figure 5.8, transfer function is calculated from impulse response of the system. Impulse response for damped system corresponding to 6 DOF loads near mudline, intermediate and interface locations are shown in Appendix D. Figure 5.15, shows amplitude plot of transfer function of axial stress per unit force FX at location near mudline, for damped and undamped system. In figure 5.15, amplitude plot is illustrated for the frequency range of 0 to 2Hz, which is of interest here. Refer Appendix E for amplitude and phase plots of different transfer function at other locations.
The transfer function provides clear information regarding the system behaviour. The first two peaks observed in figure 5.15 corresponds to the first and the second bending mode of the system. Which means that the system is prone to undergo resonance when large energies are present at those frequencies. For wind loads, most of the input energy lies within 1P and 3P range.

In traditional fatigue life calculation, the gravity loads of structures are assumed to only contribute to the mean stress of the structures. However, due to the large deformation of the offshore structures subjected to environmental loads, the centers of gravity for structures vary with time. The gravity loads then exert additional actions on the structures and change their stiffness known as P-Delta effects. In addition, the stress "stiffening/softening" effects induced from the gravity will also tune the structure’s stiffness, thus changing its natural frequencies and alter the response. Since a slight variation of stress amplitude may cause a significant change in the fatigue damage, the gravity effects can have significant influence on the calculation of fatigue damage [22].

In FEM software ANSYS, application of gravity to the system is not so straightforward. ANSYS applies gravity as an external gravitational field which excites the whole system, thus, introducing vibration of its own. This was tested on both, the jacket support structure as well as a SDOF system. Hence, due to software limitation, effect of gravity on the stiffness and the natural frequencies of the system were not evaluated. Thus, in this case, all the transfer functions derived are without the effect of gravity. Although, it has to be stressed out that with the right tools available, effect of gravity should be considered in any system.

### 5.3.2. Determining stress response under wind loading

Now that the transfer functions of the system have been established, it can be used to determine stress response due to wind loading. By multiplying Fourier Transform of wind load time series with its respective transfer function, we find the stress response spectrum for the location under consideration. By performing inverse Fourier Transform on the stress response spectrum, stress time history is obtained. To validate the method, this stress time history is compared with the stress time series found from a full time domain simulation in ANSYS. Figure 5.16 shows the frequency domain method to calculate the stress response time history due to wind loading.

![Figure 5.16: Flowchart illustrating frequency domain calculation to determine stress response time series](image)
As an example, axial stress component corresponding to wind load FX (for a random seed) at location near mudline is shown in figure 5.17. The figure shows a comparison of stress response time series obtained through frequency domain method and through time domain simulation. Comparison of response time series from time domain and frequency domain analysis for individual stress components corresponding to different DOF wind loads are referred in Appendix F.

![Figure 5.17: Comparison of axial stress response time series due to FX @ near mudline](image)

The top time series in figure 5.17 is resulting from dynamic analysis in time domain and the bottom one is obtained from frequency domain analysis. First observation is the transient effect in the initial part of time series. This is generally introduced when simulating in a software because the variables that define the system behavior are still changing in the beginning of time series before reaching steady state. While calculating fatigue damage, this part of the time series has to be ignored.

Secondly, it was observed that the mean of time series is different in frequency domain. For the example stress time series shown in figure 5.17, mean obtained through frequency domain analysis is $-1.328 \text{Mpa}$, while mean found through time domain analysis is $-1.342 \text{Mpa}$. This is resulting because of the transformation from time domain to frequency domain. While performing Fast Fourier Transform, some data is lost which results in a different mean. Extensive research on different frequency domain transformation methods is recommended. While, the difference in mean is not an issue when it comes to calculating fatigue damage for steel, but for materials where mean of the stress time series matters, frequency-domain transformation methods should be further investigated.

Except for the first two observations, remaining part of the time series is an exact match with that of the time domain simulation. This is an initial validation of frequency domain method. Next step is to obtain maximum design stress resulting due to all 6 DOF wind loads by combining axial and bending stress components. Since, in this thesis, we are only dealing with a linear system, superposition principal holds. As the stresses obtained due to different wind load components are acting along the same direction, they can be added together. Figure 5.18, 5.19 and 5.20 illustrates the maximum design stress (for a random seed) obtained at three locations within the support structure, namely, near mudline, intermediate and interface.
5.3. Hybrid frequency domain approach: OWT support structure

Figure 5.18: Comparison of maximum design stress time series at mudline location calculated through Time domain and Frequency domain method

Figure 5.19: Comparison of maximum design stress time series at intermediate location calculated through Time domain and Frequency domain method
5.4. Frequency domain analysis for multiple loads

To this point, only the response resulting from a single random load input is considered. Traditionally, dynamic analysis of OWT support structure will consist of multiple random load inputs. However, the concept of frequency domain method can be applied here as long as the phase relationship information between the load input is preserved. Since, the system is linear, combined response can be calculated by a simple linear superposition of response spectra due to multiple load inputs.

An OWT support structure has to be assessed for both wind and wave excitation. The previous section presented a method to derive the stress response spectrum in support structure due to wind loads acting at the interface level. If the response due to wind and wave excitation are assumed to be fully independent, the combined response can be determined by adding their respective response spectra [35]. The flow chart in figure 5.21 illustrates the steps to obtain total stress response spectra due to wind and wave loads.

---

1 For clear presentation, the mean from each of these time series has been subtracted
When calculating wave loads on a structure, Morison’s formula is generally applied which introduces a non-linearity through the drag term. This would make it difficult to solve for hydrodynamic forces in frequency domain. For monopile structures typically used for offshore wind turbines, it is generally assumed that the inertia term is dominant over the non-linear drag term. However, for deep water structures, the non-linear drag term can increase the response of the structure considerably. Many offshore design software offers several options for linearization, but these are not discussed in this thesis. For more information on linearization technique used to linearize the forcing function in the frequency domain, refer [18].

5.5. Comparison of frequency domain results with time domain

From figure 5.18, 5.19 and 5.20, it was concluded that frequency domain analysis gave very good results when compared to the results of time domain simulations. Using the frequency domain method, maximum stress response corresponding to two different seeds are obtained for the three locations under consideration. Before moving on to the fatigue damage, first the statistical properties of the stress response time series from different analyzing technique are compared. Results of the same are shown in table 5.1 and 5.2 in the form of mean and standard deviation of stress time series.

Table 5.1: Comparison of mean$^1$ of stress response time series obtained through Time domain and Frequency domain analysis

<table>
<thead>
<tr>
<th></th>
<th>Mean - $\bar{\mu}$ [Mpa]</th>
<th>TD</th>
<th>FD</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Case 1</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mudline</td>
<td>-14.00</td>
<td>-13.86</td>
<td></td>
<td>-1.01%</td>
</tr>
<tr>
<td>Intermediate</td>
<td>-19.88</td>
<td>-19.68</td>
<td></td>
<td>-1.00%</td>
</tr>
<tr>
<td>Interface</td>
<td>-19.74</td>
<td>-19.54</td>
<td></td>
<td>-1.00%</td>
</tr>
<tr>
<td><strong>Case 2</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mudline</td>
<td>-11.55</td>
<td>-11.44</td>
<td></td>
<td>-1.00%</td>
</tr>
<tr>
<td>Intermediate</td>
<td>-17.34</td>
<td>-17.17</td>
<td></td>
<td>-1.00%</td>
</tr>
<tr>
<td>Interface</td>
<td>-16.83</td>
<td>-16.66</td>
<td></td>
<td>-1.00%</td>
</tr>
</tbody>
</table>

$^1$Results are produced after removing transient effect from initial part of time series
5.5. Comparison of frequency domain results with time domain

Table 5.2: Comparison of standard deviation\(^1\) of stress response time series obtained through Time domain and Frequency

<table>
<thead>
<tr>
<th>Case</th>
<th>TD</th>
<th>FD</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Mudline</td>
<td>2.39</td>
<td>2.37</td>
</tr>
<tr>
<td></td>
<td>Intermediate</td>
<td>1.78</td>
<td>1.76</td>
</tr>
<tr>
<td></td>
<td>Interface</td>
<td>1.86</td>
<td>1.84</td>
</tr>
<tr>
<td>Case 2</td>
<td>Mudline</td>
<td>1.95</td>
<td>1.94</td>
</tr>
<tr>
<td></td>
<td>Intermediate</td>
<td>1.16</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td>Interface</td>
<td>1.11</td>
<td>1.10</td>
</tr>
</tbody>
</table>

Case 1 and case 2 represent two different random wind time series. Stress time series determined through frequency domain method compares really well with that of time domain result. The difference in mean and standard deviation is negligible between the two techniques. Assuming that the load time history is taken from a Gaussian random process then according to the central-limit proposition, for linear systems, response time history will also have a Gaussian distribution. But to avoid making assumptions about the distribution of data, probability density function (PDF) of time series from case 1 is estimated using Kernal distribution. For comparison, PDF of time series obtained through time domain and frequency domain analysis is shown in figure 5.22 near mudline.

From figure 5.22, it follows that the probability distribution of stress time history from frequency domain method is a close match. The negligible differences observed here is due to the difference in calculated mean and standard deviation, which can occur due to the numerical accuracy of the software and due to the transformation from time domain to frequency domain and vice versa.

Now that the stress time series has been analyzed, fatigue damage can be calculated. To complete the fatigue analysis, the following steps has to be executed:

- Count the stress variations
- Compare stress variations with the S-N curve

\(^1\)Results are produced after removing transient effect from initial part of time series
• Calculate the fatigue damage $D_{fat}$ using Miner sum

Commonly in time domain, stress range, amplitude and number of cycles are calculated by performing rainflow counting. Alternative to time domain rainflow counting method, several frequency domain counting methods exist. These methods (Rayleigh, Rice and Dirlik) can be used to calculate fatigue damage from the frequency domain response directly. Many research papers have shown that the empirical Dirlik method uses most information from the spectra and hence shows the best comparison with the time domain rainflow counting method. Now, using different counting methods will introduce certain error of their own in the calculated fatigue damage. Since, the objective of using frequency domain analysis is to show that it can determine the same stress response time series as that of time domain analysis, frequency domain counting algorithms are not used here. Instead, the frequency domain response is transformed back to time domain and then rainflow counting is used same as shown in figure 5.10.

Results in terms of fatigue damage $D_{fat}$ calculated for a design life of 25 years is shown in table 5.3 for case 1 and case 2.

Table 5.3: Comparison of fatigue damage using time domain and frequency domain method for 2 cases

<table>
<thead>
<tr>
<th>Fatigue damage ($D_{fat}$) for a service life of 25 years</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>Case 1</td>
</tr>
<tr>
<td>Mudline</td>
</tr>
<tr>
<td>Intermediate</td>
</tr>
<tr>
<td>Interface</td>
</tr>
<tr>
<td>Case 2</td>
</tr>
<tr>
<td>Mudline</td>
</tr>
<tr>
<td>Intermediate</td>
</tr>
<tr>
<td>Interface</td>
</tr>
</tbody>
</table>

The different analyzing techniques show remarkably good comparison. It needs to be highlighted that fatigue damage determination with bi-linear S-N curves having slopes of 3 and 5 differ radically for small changes in stress ranges. Apart from minor variation in mean and standard deviation, small differences are also introduced due to the rainflow counting algorithm. Nonetheless, the calculated fatigue damage of the OWT support structure using frequency domain approach gives very accurate results.

So far, the results show that frequency domain method is a good alternative for time domain simulations. To check if the frequency domain approach is effective in terms of computational time, a small comparison is performed. Computation time required to perform time domain simulation mainly depends on the system, length of the time series and time step. Larger the length of time series and smaller the time step, more computational time is required. The time series we are working with is 600 s long, with a time step of 0.04 s. On a computer with 2.6 GHz processor and 8 GB RAM, dynamic analysis in time domain with post processing took close to 70 min for 1 seed. On the other hand, frequency domain method was able to complete the analysis in less than a minute. To see the differences on large scale, comparison of computational time required for calculating fatigue damage for one wind speed class, consisting of 72 seeds is shown in table 5.4.

---

1 Results are produced after removing transient effect from initial part of time series
Table 5.4: Comparison of computation time\(^1\) required for dynamic analysis of 72 seeds through TD and FD analysis

<table>
<thead>
<tr>
<th></th>
<th>Computational time [min] for 72 seeds</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TD</td>
</tr>
<tr>
<td>Reading load input</td>
<td>4320</td>
</tr>
<tr>
<td>Dynamic analysis</td>
<td>1</td>
</tr>
<tr>
<td>Post Processing</td>
<td>720</td>
</tr>
<tr>
<td><strong>Total time</strong></td>
<td><strong>5040</strong></td>
</tr>
</tbody>
</table>

From the results, it can be concluded that frequency domain method is much more effective and will help in bringing down the computational time, effort and cost involved in the fatigue design of OWT support structure.

5.6. Conclusion

Time domain and frequency domain methods are two different analyzing techniques, which are entirely interchangeable for a linear system. Difference being that every time domain characterization is a specific realization of a stochastic process, whereas the frequency domain characterization entails all stochastically possible realizations.

Traditionally, frequency domain analysis is performed by means of spectral method using power density spectral (PSD) functions. Disadvantage of using PSD functions is that the phase information is lost. To preserve phase information, linear transfer functions are derived in this thesis using frequency domain representation of input and output. According to the nonlinearity of turbine responses to different mean wind speed, a wind-induced stress transfer function varies with different mean wind speed [23]. Which means, one has to derive different stress transfer functions corresponding to different mean wind speed, which increases the computational time. Another drawback of using spectral methods is that it is based on the Rayleigh distribution assumption that can only be applied to the narrow band random process [16]. While the response of the offshore structures is always a very complicated stochastic and wide-banded process [21]. In contrast, the time-domain analysis considers the stochastic characteristics and nonlinear influences and is assumed to be more accurate. However, it is more time-consuming.

In this chapter, a hybrid frequency domain approach has been presented for the calculation of fatigue damage of an OWT support structure. This method consists of two parts. The first part is carried out in the frequency domain, whereas the second part in time domain. First, using the impulse response function of the system and with the help of Fast Fourier Transform, both the input and the output have been characterized in frequency domain. Afterwards, location specific transfer functions are calculated which are then used to derive stress response spectra. The stress response spectra which is the output of the first part is then converted to the response time history, which will be the input of the second part. After that, statistical analysis of the response time history is conducted by using rainflow counting algorithm. By means of stress ranges and their corresponding occurrence frequencies, fatigue damage is determined based on the Palmgren-Miner rule and the chosen S-N curves. Thus, the first part which provides the response time history avoids the complex coupled analysis in time domain, and the second part assess the fatigue damage by avoiding the Rayleigh distribution assumption. And since, transfer functions are derived using impulse response of the system, it remains same for different mean wind speed.

The results obtained in this chapter proves that fatigue damage of OWT support structure can be determined effectively in the frequency domain. After removing the transient effect from response time series, fatigue damage calculation using different analyzing technique compares well. With the introduced approach, phase information of input and output is well preserved. Frequency domain method does introduce a difference in the mean and standard deviation of stress response. Difference in mean can be ignored as it is very small and doesn't play a role in damage. There is a further need for investigation into the avail-

\(^1\)Required time shown in the table for FD is valid when the transfer functions has already been derived
able transformation techniques from time domain to frequency domain and vice versa. To summarize, this chapter presents an approach in which the response corresponding to the load time series obtained from WTM can be calculated rapidly in frequency domain without the need of sensitive information sharing. This concludes that by using frequency domain approach, fatigue damage can be assessed very quickly without loss of accuracy compared to time domain calculations.
Offshore wind farm installations have been growing in size and number over the past few decades. The main challenge for future offshore wind farm installations is to bring down the cost of energy. Generally, fabrication and installation of foundations are major cost drivers and can amount to approximately 20% of the overall project costs. Therefore, apart from ensuring safety and reliability, the importance of cost optimized offshore wind turbine foundation design is strongly related to the economic feasibility and further advancement of offshore wind energy.

The design of support structure is driven by cyclic loads due to wind and waves, which excite the structure. Thus, fatigue analysis of OWT support structure becomes an important design criterion. Therefore, study in this thesis is focused on fatigue design aspects of the structure. In the context of the desired cost reductions for offshore wind energy, the main objective of this thesis is set to – "reducing computational effort and time required for a typical fatigue design cycle of OWT support structure".

Reduction of computation time can be achieved through various methods, e.g. by either using a reduced foundation model or by alternatively using an integrated design approach. But in this thesis, the main emphasis is on more practical challenges faced in an industry from the perspective of a foundation designer. During the thesis, it was observed that common industrial practice is to condense the wind-wave data which is in the form of scatter diagrams, in order to reduce the computation time. By doing so, one has to compromise on the accuracy of the estimated hydrodynamic fatigue damages. In most scenarios, foundation designer has to blindly follow the lumped sea states provided by the metocean engineer, without being able to check its reliability. Thus, in chapter 3 of this thesis an investigation is performed on existing lumping methods for correlated wind and wave data and a comparison for accuracy in fatigue results is exemplified.

The complete fatigue load assessment of an OWT is often based on time-domain simulations, which is a comprehensive and time-consuming task. Therefore, an important question asked in this thesis is, whether it is possible to reduce the number of load cases while retaining a high level of accuracy in fatigue results. An experimental approach is presented in chapter 4, investigating the reduction in design load cases. Another analyzing technique which has proved very efficient for fatigue design of OWT support structure is the frequency-domain approach. Frequency-domain approach is computationally so fast that it negates the need of lumping and reducing in any form. Chapter 5 presents frequency-domain application for wind loads acting on OWT support structure.

This chapter summarizes the conclusions of the explored topics together with recommendations for future work.
6.1. Conclusions

Lumping methodology for correlated wind-wave data

Establishment of wind and wave correlation does not depend on the structure properties. However, the general criteria of lumping of scatter diagram is to achieve similar damage from lumped sea-states when compared to all sea-states. Thus, the lumping methods are indirectly dependent on the dynamic response of the structure. For this reason, these existing lumping methods are validated for jacket type structure.

KCI and Martin Kühn’s lumping approach is independent of any structural property. Damage assessment through these methods proved that they highly underestimate the fatigue damage when it comes to jacket type structure. From illustration in various research papers [15, 32], it appears that Martin Kühn’s approach works well with monopile foundations. This thesis research is not sufficiently adequate to comment on the applicability of Martin Kühn’s method for different type of support structure. But, it is clear that all lumping methods do not give accurate results for different types of support structures.

Marc Seidel’s lumping approach was able to produce reasonably accurate cumulative fatigue damages but was not able to preserve hydrodynamic fatigue damage at all locations within the OWT support structure. The assumptions made regarding quasi-static and resonance part of response and its relation with the fatigue damage holds good for a monopile support structure. The assumption that fatigue damage is directly proportional to dynamic response due to $1^{st}$ bending mode is very structure specific. As was demonstrated in this thesis, for jacket structure the dynamic response was governed by $2^{nd}$ bending mode. Hence, for locations where dynamic response due to $2^{nd}$ bending mode was dominant, Marc Seidel’s approach produced inaccurate fatigue damage results.

Nonetheless, M. Seidel’s lumping approach produced the most accurate results out of the three. This study also validated that for lattice type structure, where higher modes are having a significant contribution to the dynamic response of the system, only M. Seidel’s method gave comparable results.

Reduction of design load case (DLCs)

The proposed reduction analysis was performed to help reduce computational time and memory storage required for time domain simulations. The analysis provided insightful information regarding the reduction capabilities of DLCs in time domain. It showed that reduction in design load cases effects the resolution of time series, which in turn effects the accuracy of final fatigue damage calculation results. As this method is based on linear-quasi-static assumptions, dynamic part of response is ignored, which leads to inaccuracy in the final damage results. It was also proven that it is difficult to generalize the reduction criteria as, different location within the support structure responds differently to a given load due to inertia effects.

Wave loads were ignored in this reduction analysis due to the fact that lattice type structures are hydrodynamically less sensitive. For slender structures with large dimensions like monopile, which are more sensitive to wave loads, it is recommended to account for the combined effect of wind and waves.

For fatigue calculations in single environmental state, a duration of 1 hour with different random seeds for each state, generally produced fatigue damages with an acceptable error margin of ±15% at all the investigated locations within the support structure. This is in close agreement with the minimum cap suggestion in IEC 61400-3 [4].

It was concluded that for preliminary stages, this reduction approach can be used to save valuable computation time. The reduction will always introduce error in the design fatigue damage and hence, use of contingency factor is advised. Choice of safety factor is open to the foundation designer but should be at least 15%.

Frequency domain fatigue analysis

In chapter 5, it was shown that frequency domain is a different way of representing a signal. When dealing with a simple signal, it is very easy to identify its properties in time domain. But when the signal is stochastic in nature, it becomes difficult to extract information. On
the other hand, when the same stochastic signal is transformed to frequency domain, its properties become much more transparent.

Because of this advantage, frequency domain method naturally becomes a more effective tool to assess fatigue damage for linear systems. Analysis from chapter 5 showed very accurate results. A small difference in the mean and standard deviation of stresses was observed. Difference in mean does not have any influence on the fatigue damage. Although, the small difference in standard deviation does affect the fatigue damage. Due to the S-N curve slope, small changes in stresses can have large influence on the fatigue damage. In this case, since, the difference in standard deviation of stress is less than 1%, it does not have a significant effect on the fatigue damage.

Obtained fatigue damages from hybrid frequency domain method compared really well with the fatigue damage through time domain simulation. Hence, it was concluded that frequency domain method is a very good alternative to time domain simulations not only in preliminary phase but also in detail design phase and the damages can be evaluated very quickly and very accurately.

6.2. Outlook

This thesis was directed on researching the possibility of a number of different methods to reduce the computational time required to perform fatigue analysis of an OWT support structure. The results look promising, but in the process many simplifications were made and many hurdles were encountered which limited the scope of this thesis. This leaves many opportunities for further research, of which a few are highlighted here.

One of the primary interactions between the turbine and the support structure is the aerodynamic damping. With increasing turbine size and weight, and the fact that offshore wind farms are moving into deeper waters, the natural frequency of the support structure will decrease, thus making the support structures more prone to wave induced fatigue loads. Presence of operating wind turbine introduces aerodynamic damping which significantly damps the resonant behavior, ergo fatigue damage. Due to the lack of information transfer between WTM and foundation designer, calculation of aerodynamic damping was not possible in this research. Hence, development of a calculation method for aerodynamic damping to improve the fatigue calculation is recommended.

The ambition for making the design procedure of offshore structures more cost effective is pushing the researchers to come with up with more ways to condense or lump design steps while preserving the accuracy of the analysis. One such important step followed in the design is the lumping of wind and wave data. From the different existing lumping methods which were investigated in this thesis, it is observed that due to the extensive use of monopiles in offshore world, the formulas developed for lumping are based on assumptions which are mostly true for slender structures like monopiles. This is one of the reasons why the response based on these lumping methods were not very accurate for jacket support structure. Hence, a promising research opportunity can be to explore cases where these lumping assumptions are not valid.

The frequency domain method presented in this thesis was carried out with a series of different programs to generate transfer functions and stress responses. To make use of this method in real life projects, development of a well organized and sturdy calculation tool will be needed.

Comparison of response from time domain and frequency domain showed minor differences in the mean and the standard deviation. Possible reasons which could have influenced the results are:

- Numerical accuracy of softwares used for different analysis
- Information lost while using transformation technique used to transform from time domain to frequency domain and vice versa

To build confidence in the frequency domain analysis for fatigue design of OWT support structures, further research should be performed in explaining the reason behind the observed differences.
General Arrangement - Jacket structure
Mode shapes with natural frequencies

1\textsuperscript{st} tower side to side:  
\( f_1 = 0.239\text{Hz} \)

1\textsuperscript{st} tower fore-aft:  
\( f_2 = 0.243\text{Hz} \)

2\textsuperscript{nd} tower side to side: 
\( f_3 = 0.776\text{Hz} \)
2\textsuperscript{nd} tower fore-aft: $f_4 = 0.782Hz$

3\textsuperscript{rd} tower side to side:
$f_5 = 1.36Hz$

3\textsuperscript{rd} tower fore-aft:
$f_6 = 1.555Hz$

1\textsuperscript{st} tower torsion: $f_7 = 1.575Hz$
Example wind load time series
Stress Impulse response of Jacket

Impulse stress response due to wind load near mudline
Impulse stress response due to wind load at intermediate location
Impulse stress response due to wind load near interface
Transfer function between wind load and stress response

Amplitude of Transfer function for $\sigma_{dir}$ per unit wind load, near mudline
Amplitude of Transfer function for $\sigma_{DLF}$ per unit wind load, at intermediate location

Amplitude of Transfer function for $\sigma_{DLF}$ per unit wind load, near interface
Amplitude of Transfer function for $\sigma_{My}$ per unit wind load, near mudline

Amplitude of Transfer function for $\sigma_{My}$ per unit wind load, at intermediate location
Amplitude of Transfer function for $\sigma_{My}$ per unit wind load, near interface

Amplitude of Transfer function for $\sigma_{Mz}$ per unit wind load, near mudline
Amplitude of Transfer function for $\sigma_{Mz}$ per unit wind load, at intermediate location

Amplitude of Transfer function for $\sigma_{Mz}$ per unit wind load, near interface
Phase angle of Transfer function for $\sigma_{\text{dir}}$ per unit wind load, near mudline

Phase angle of Transfer function for $\sigma_{\text{dir}}$ per unit wind load, at intermediate location
Phase angle of Transfer function for $\sigma_{\text{dir}}$ per unit wind load, near interface

Phase angle of Transfer function for $\sigma_{\text{My}}$ per unit wind load, near mudline
Phase angle of Transfer function for $\sigma_{My}$ per unit wind load, at intermediate location

Phase angle of Transfer function for $\sigma_{My}$ per unit wind load, near interface
Phase angle of Transfer function for $\sigma_{M_2}$ per unit wind load, near mudline

Phase angle of Transfer function for $\sigma_{M_2}$ per unit wind load, at intermediate location
Phase angle of Transfer function for $\sigma_{MZ}$ per unit wind load, near interface
Comparison of response time series for time domain and frequency domain for different stress components

Axial stress ($\sigma_{D(r)}$) time series due to wind load, near mudline
Axial stress ($\sigma_{\text{Dir}}$) time series due to wind load, at intermediate location

Axial stress ($\sigma_{\text{Dir}}$) time series due to wind load, near interface
Bending stress ($\sigma_{M_y}$) time series due to wind load, near mudline

Bending stress ($\sigma_{M_y}$) time series due to wind load, at intermediate location
Bending stress ($\sigma_{M_y}$) time series due to wind load, near interface

Bending stress ($\sigma_{M_z}$) time series due to wind load, near mudline
Bending stress ($\sigma_{M_z}$) time series due to wind load, at intermediate location

Bending stress ($\sigma_{M_z}$) time series due to wind load, near interface
[1] DMS: 0045-5699. Site specific loads for preliminary offshore foundation design for Moray Firth with the V164-8.0 MW HH107m. 2014.


