Project funded by the European Commission under the 6th (EC) RTD Framework Programme (2002- 2006) within the framework of the specific research and technological development programme "Integrating and strengthening the European Research Area"

Project UpWind

Contract No.:
019945 (SES6)

"Integrated Wind Turbine Design"

Design solution for the Upwind reference offshore support structure

Deliverable D4.2.5

(WP4: Offshore Foundations and Support Structures)

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Document Information

DOCUMENT TYPE	Deliverable Report
DOCUMENT NAME:	UpWind_WP4_D4.2.5_Design solution Upwind reference offshore support structure
REVISION:	3
REV.DATE:	26 October 2010
CLASSIFICATION:	R0: General Public
STATUS:	S0: Approved / Released

Acknowledgement

This report is the result of work jointly carried by several participants in the Task 4.2 “Concepts for deep-water sites”. Contributions from Universität Stuttgart (T. Fischer), Risø-DTU (J.D. Sørensen), Delft University of Technology (W. de Vries), DONG A/S (N.J. Tarp-Johansen), GEGR-E (P.W. Cheng), Garrad Hassan (Andrew Cordle, T. Camp), GL Wind (B. Schmidt, K. Argyriadis), Vattenfall (K. Roed-Jensen), Fraunhofer IWES (F. Vorpahl), NREL (J. Jonkman) and Shell (A. Ploeg) are kindly acknowledged.

STATUS, CONFIDENTIALITY AND ACCESSIBILITY							
Status			**Confidentiality**			**Accessibility**	
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PL: *Project leader* **WPL:** *Work package leader* **TL:** *Task leader*

Summary

The jacket foundation support structure study documented in this report has been carried out with in the ‘Foundation and Support Structure’ work package as part of the EU Upwind project (SES6 No 019945 UPWIND). This study consists of a preliminary design phase and a final design phase.

In preliminary design phase, a preliminary assessment of the jacket design has been carried out from given loads for the NREL 5.0 MW turbine. The dimension of the preliminary design jacket bottom width is chosen such that the requirements from the support structure 1st natural frequency and total optimal cost of the structural steel. The preliminary design jacket dimensions are optimized based on requirements to the structural steel utilization ratios and minimum fatigue lives.

In the final design phase, the preliminary jacket has been used by GH Bladed to generate the wind loads for the detailed design phase. The jacket structure is optimized with respect to the natural frequency, extreme event and fatigue conditions, i.e. the natural frequency of the overall structure is within the allowed range and all member and joint utilizations as well as the fatigue lives are within the allowable limits.

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Preface

This work has been carried out within the ''Foundation and Support Structure'' work package on as part of the EU Upwind project (SES6 No 019945 UPWIND). This report consists of a general description of the jacket model and obtained design results from the detailed design study of an exemplary jacket foundation design for the Upwind project.

The report presents six chapters including introduction, design requirements, concept selection, design methodology, design procedure, conclusions and recommendations.

N.K. Vemula (Rambøll) worked on the detailed design and optimization of the jacket foundation structure including eigen-frequency, extreme and fatigue analyses in the preliminary design and final design phase with input from other members of WP4. Furthermore, the chapters regarding design requirement, design methodology, design procedure as well as conclusions and recommendations are carried out by N.K. Vemula (Rambøll).

The chapter on the introduction is carried out by W. de Vries (TU Delft). The section regarding the design load case implementation is carried out by T. Fischer (Universität Stuttgart), A. Cordle (Garrad Hassan) and B. Schmidt (Germanischer Lloyd).

1. Introduction

1.1 Purpose of this study

The jacket foundation structure described in this report is intended to demonstrate a design solution for a support structure for an offshore wind turbine in 50m of water depth. This water depth is chosen to represent a deep water site as it exceeds the largest water depth that any wind turbine on a fixed support structure has been installed in to this date. It is not necessarily the ultimate solution for deep water; alternative solutions that prove to be more cost-effective might be conceivable. Instead it is intended to be a reference which can be used for the following:

- Comparison with other support structure concepts
- Demonstration of the effectiveness of design improvements
- Demonstration of the sensitivity to various design parameters

By carefully documenting the model dimensions and the design approach as well as all input data used for generating the reference structure a valuable reference is created. Hereby the need to rely on actual projects for which data is generally not in the public realm is partially eliminated. While it is conceded that measurements from the field are important, this fictive support structure can be of use by allowing a clear and consistent comparison of design approaches, parameter studies and support structure concepts.

Several jacket support structures have already been installed. Therefore the jacket support structure concept cannot be considered an entirely new concept. However, there are several reasons why the jacket concept is a good subject for the purpose of this study:

- Within UpWind Work Package 4 (WP4) the main focus is on fixed steel structures.
- Knowledge about state-of-the-art in steel fixed structures is present among WP4 members.
- In preliminary comparisons of jacket and tripod support structures, the jacket appears to be the more economic solution for the site conditions considered in this report.
- Rough comparisons of the reference design presented in this report with actual jacket designs are possible.

1.2 Approach

Before the actual design of the reference structure can be commenced a consistent set of design data is required. To this end a design basis has been created [1] in which all relevant site data and environmental data are collected. The aim is to have a document that can live up to industry standards. A preliminary geometry has been defined by Rambøll on the basis of an allowable natural frequency range. A load document has been set up reporting wind turbine loads, calculated by Garrad Hassan on the basis of simulations using the UpWind reference turbine, an equivalent support structure with the same natural frequency as determined for the preliminary structure. Subsequently Rambøll has checked the preliminary structure for the extreme event and the fatigue limit states using the loads from the load document, adjusting the dimensions of the structure where necessary.
With suitable dimensions for the structure known the final design phase is entered. In this phase a more detailed set of turbine loads has been generated using the preliminary geometry determined in the previous phase. These loads are combined with wave loads for the extreme event and fatigue analyses. An optimisation of the geometry finally leads to the structure dimensions presented in this report.

The report roughly follows the steps in the design process as described above. Chapter 2 presents the most important design requirements and limitations on the structural dimensions. In the subsequent chapter the design methodology is explained for the preliminary design phase as well as the final design phase. Chapter 4 describes the design load cases and the loads for the preliminary and final design phases. In Chapter 5, the results in terms of structural dimensions are presented for both phases. This is followed by the description of the final design procedure. The results of the natural frequency analysis, the extreme event analysis and the fatigue assessment are reported. In the final chapter the conclusions regarding the reference structure design and recommendations for further analysis are given.

2. Design Requirements

In this section the design requirements for the jacket structural design are explained, including the allowable natural frequency for the structure in relation to the UpWind reference turbine. Also material factors for various design situations are given.

2.1 General

The 1st eigenfrequency of the entire structure must be located in the range of 0.22 Hz-0.310 Hz according to the design basis [1]. The penetration of the jacket pile into the soil is determined under consideration of the plastic soil capacity while the design of the pile steel is carried out under consideration of characteristic soil conditions. The design requirement for the jacket members and joints is that the maximum steel utilization ratio is below 1. The minimum fatigue life for all jacket members and joints has to be above 20 years. Soft soil conditions stated in [1] have been used for the design.

The design is carried out for a water depth of 50.0 m w.r.t. MSL. The interface level and hub height are set at 20.15 m and 90.55 m w.r.t. MSL [1]. The concrete transition piece dimensions estimated and used in this study are 9.6*9.6*4. Information regarding turbine parameters and tower geometry is provided in the design basis. The pile and jacket steel utilizations are also checked with the hard soil profile provided in design basis in order to confirm whether steel utilization ratio is below 1.

2.2 Material safety factors

The load-carrying capacity of piles shall be based on strength and deformation properties of the pile material as well as on the ability of the soil to resist pile loads. For the requirements in extreme event analysis, the piles are designed as geotechnical elements by assuming the material safety factors as stated in Table 2 1 and the jacket elements are designed in the elastic ultimate limit state with material safety factors equal to unity.

Geotechnical Design

Material factors for the soil parameters are shown in Table 2–1 for the design of the pile as geotechnical element to consider for the plastic soil conditions.

Table 2–1: Material safety factors for pile as geotechnical element [5]

Material Parameters	Material safety factor for the plastic soil conditions
Angle of internal friction φ	1.15
Undrained shear strength c_u	1.25
Axial load-carrying capacity	1.25

For this analysis, equilibrium has to achieve between the load carrying capacity of the soil and the pile loads. Normally this design practice is crucial for the calculation of the necessary pile length and as well as pile diameter.

Elastic Pile Design

This analysis is based on the characteristic soil strength, i.e. soil strength parameters with material safety factors equal to unity. The purpose of this analysis is the verification of the capacity of the steel structure where the soil reaction acts as a boundary condition. Material safety factors for the steel in accordance to [5] are shown in **Error! Reference source not found.**. Normally this analysis is dimensioning for the wall thicknesses of the pile as required from the extreme event conditions.

Table 2–2: Partial material factors for structural steel design [5]

	ULS
Steel strength	1.15
Modulus of elasticity	1.00

3. Design Methodology

The design of a jacket structure is based on three analyses, namely natural frequency, extreme events and fatigue. In general, the penetration and the diameter of the piles are designed by the extreme event, while the wall thicknesses are determined by fatigue analyses. The jacket legs are designed by either the extreme events or fatigue analyses.

In the following sections, the design methodology for the applied jacket concept is described. First some general considerations regarding the jacket configuration are presented, followed by a description of the structural model and the main dimensions of the structure. Subsequently the design procedure for preliminary design phase and the final design phase is explained

3.1 Jacket Concept

3.1.1 General

The jacket foundation concept is characterised by a number of legs, which are stiffened by braces. The legs are supported by piles – either main piles, skirt piles or a combination of these. For the present design a four legged jacket is applied with four levels of –braces, a horizontal brace and main piles. Furthermore, the main pile concept, i.e. the legs are located inside the pile top and consequently be grouted, is applied. Ideally, the piles in a jacket should carry the loads exclusively by axial tension and compression. This is normally secured by placing the mud brace close to the mud line and therefore minimizing the moments building up in the piles.

The X-bracings are designed in such a way that the angle between the brace and leg exceeds 30 degrees in accordance to the NORSOK recommendations [7]. Requirements from NORSOK [7] regarding the minimum gap between braces at tubular joints (50 mm) and minimum distance between the brace-chord weld and the end of the can (the maximum of one fourth of the chord diameter or 300 mm) are fulfilled. Due to the large water depth (50 m) at this site, four levels of X-braces are implemented in order to comply with the requirement of the minimum angle between chord and brace.

The Timoshenko beam model is applied ROSA [3]. Moreover, a simple local joint flexibility (LJF) model is included; i.e all braces are calculated as simple T and Y joints, where the flexibility for each brace is calculated as if no other braces were present at the joint. Note that braces are automatically cut off at the brace centreline intersection with the chord wall, so the stiffness will be reduced.

A concrete block transition piece is applied as a connection between the tower and the jacket structure. The material for the transition piece has been chosen as reinforced concrete rather than steel, which is based on a cost benefit evaluation. The concrete transition piece has the weight as a disadvantage. However, it is neither as susceptible to fatigue damage nor as labour intensive compared to a steel transition piece.

However, it is recommended that further studies are carried out on alternative transition piece, grouted connection and total cost reduction possibilities. A detailed finite element analysis would be necessary to check whether the transition piece can withstand the interface loads.

Furthermore, a detailed finite element analysis is necessary in order to verify that the grouted connection between the jacket and the piles is designed sufficiently for the transfer of axial loads and bending moments.

In general, jacket steel is more expensive than the pile steel (due to high yield strength of the steel). Hence, it is recommended to minimize the jacket steel mass by transferring mass into the pile so the total foundation cost will be reduced.

3.1.2 Structural Jacket Model

This section describes the overall jacket concept applied at 50 m water depth. Figure 3–1 shows a 3D-model of the jacket foundation and the superstructure, i.e. tower and rotor-nacelle-assembly (RNA). Secondary steel such as two boat landing bumpers, anodes and J-tubes are also shown in below figure. The background for the jacket design is presented in the following sections.

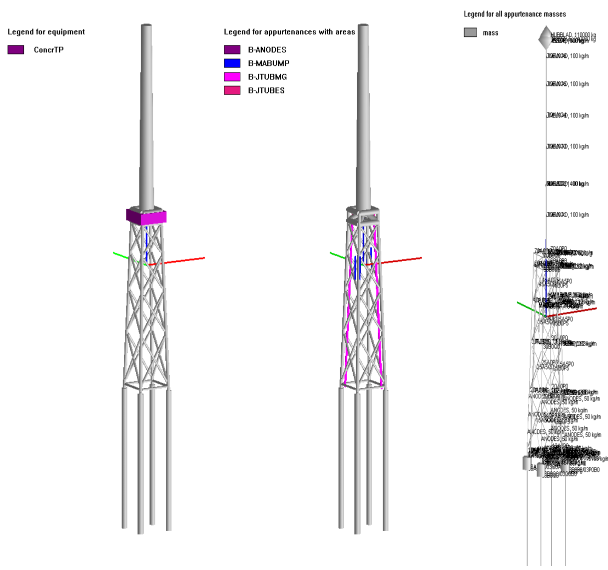


Figure 3–1: Jacket foundation model

The jacket FE-model consists of general non-linear beam and pile elements, and the load transfer from the concrete TP to the jacket legs is modelled by a stiff frame of fictitious elements resembling the stiffness of the reinforced concrete.

Figure 3–2 shows how the jacket leg is located inside the pile while the mud braces are located in a certain elevation above the mud line. The distance between the mud brace and mudline for this design is 6.0 m and the distance between the bottom of the grouted connection and the mud line is 0.5 m. The link elements on the right side of the figure below indicate the top and bottom of the grouted connection.

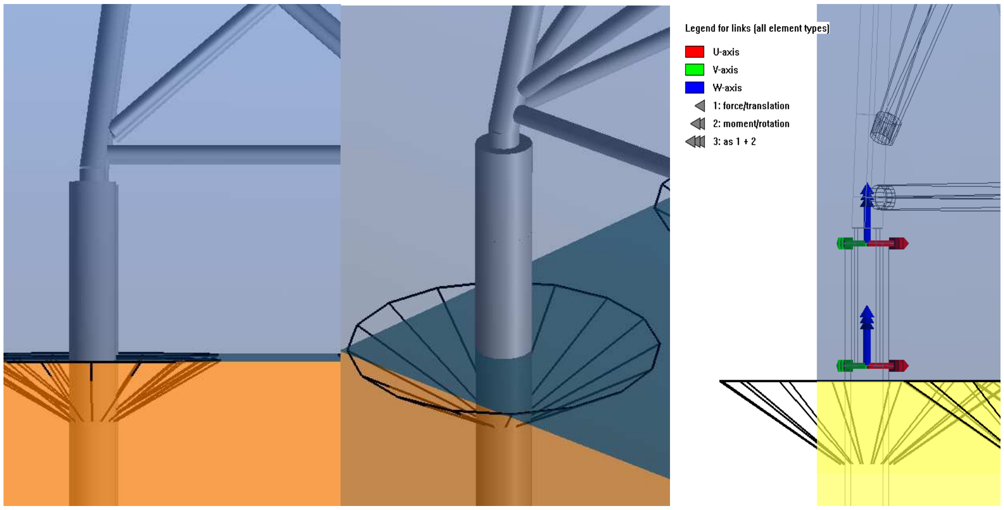


Figure 3–2: Pile and grouted connection

3.1.3 Main dimensions

Figure 3–3 shows the thicknesses, diameter over thickness (D/t)-ratios, material names and applied corrosion allowance for extreme event analysis in the splash zone. Note that the thicknesses and D/t-ratios have been adjusted for corrosion, whereas the steel amounts presented in chapter 5 corresponds to the uncorroded structure. It can be seen that the wall thicknesses in the vicinity of the tubular joints are locally increased by can sections in order to increase fatigue life and punching shear capacities. The D/t ratio is a key parameter for local buckling in the jacket structure. It is should be noted that the bottom part of the jacket legs are designed with high wall thicknesses and lower D/T-ratios in order to secure steel utilization ratios below 1.0. The fictitious material 'NOW' has no weight which is used for the fictive beam framework elements resembling the stiffness of the reinforced concrete TP. 'NOW' elements are therefore not checked with respect to the stresses.

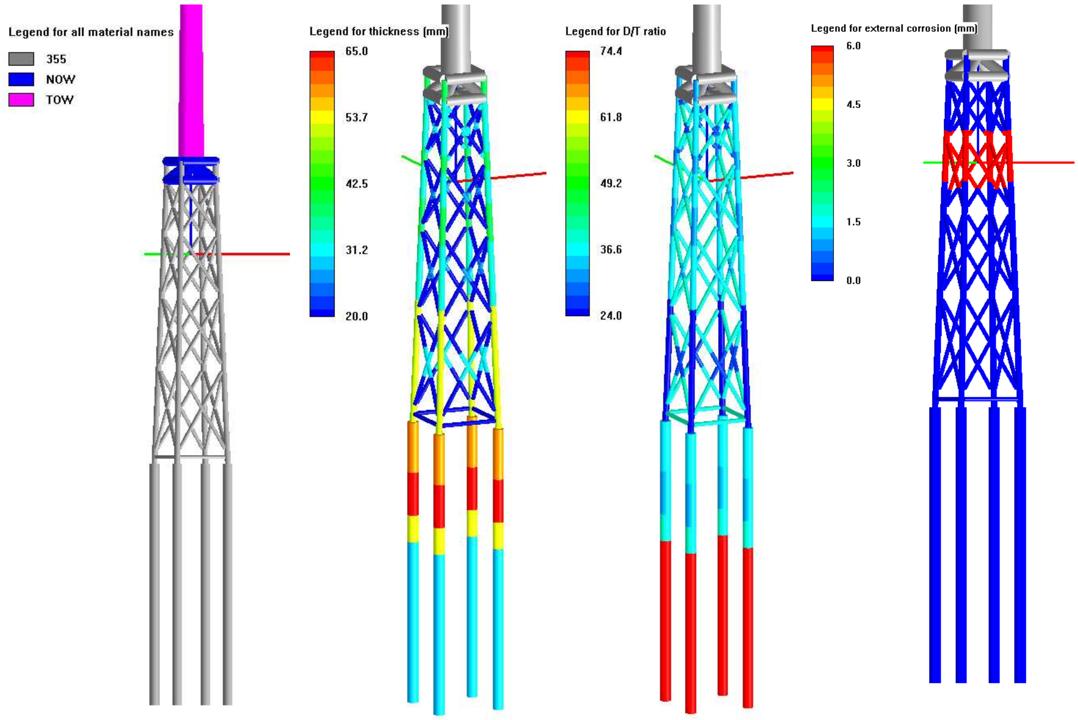


Figure 3–3: Material names, thicknesses, D/t-ratios and corrosion allowance for extreme event analysis in splash zone.

3.2 Design tools

This section gives a brief description of the design tools used in the detailed design study of the jacket foundation support structure.

ROSAP [3] is the name of the Rambøll Offshore Structural Analysis Programme Package. It has been developed as a tool to solve the problems commonly arising in analyses of fixed offshore steel platforms. During recent years the programme package has been extended to solve problems regarding offshore wind turbine support structures.

The programmes used in the present design study are:

<u>ROSA</u>: Static and dynamic analysis of space frame structures

ROSA determines the deformations and sectional forces in the entire structure. Environmental loads due to gravity, buoyancy, wave and current loads are generated automatically. Furthermore load time series and accelerations (to estimate inertia forces) (from e.g. FLEX5) can be imported in the programme and applied to the structure. In this study load time series excluding accelerations are applied at interface level.

<u>STRECH</u>: Member stress check

The programme performs stress check of beam and pile elements according to a user specified code.

<u>FATIMA</u>: Fatigue analysis programme

The programme performs damage and fatigue life calculations of joints, beam, pipe and pile elements defined in ROSA according to a user specified code.

FATCOM: Fatigue damage combination programme

The programme performs fatigue damage combination of damages stored in damage files from the programme FATIMA or other sources.

WAVGEN: Wave generation programme

The programme generates velocities, accelerations and excess pressures in a rectangular grid for waves and current. Several wave theories and spectra types are available.

ELLA: Damage equivalent moments programme

The programme calculates the damage equivalent moments based on damages calculated by FATIMA and FATCOM.

TUBJOI: Tubular joints analysis programme

The program is used to perform the punching shear analysis of tubular joints in fixed offshore steel structures.

SIDSEL & STPLOT: Structural plot programmes

The programme SIDSEL is used to generate the structural geometry plots, mainly for fatigue details and fatigue lives.

The programme STPLOT is used to generate the structural geometry plots, mainly for steel utilization ratios.

3.3 Preliminary design phase

In this phase, a preliminary assessment of the jacket design has been carried out for given loads in section 4.1 for the Upwind reference turbine, which is an update of the NREL 5.0 MW turbine [11] with an industry-standard controller [13].

A parameter study has been performed in order to determine a feasible configuration for the jacket dimensions, pile penetration and diameter. The intention of designing an optimal jacket structure is achieved by varying the base width at top and bottom of the jacket. The minimum pile diameter is chosen from the various jacket configurations. The well known behaviour of four legged structures has led to the selection of this base concept.

The following parameters have been considered.

- Two soil profiles (soft soil and hard soil, used for steel checks only).

- Six different jacket bottom base widths are chosen for the analysis. The final base width of 8.0 m at the top/interface has been chosen in accordance to the required tower bottom diameter (5600 mm) and transition piece dimensions (4*9.6*9.6).

- Three different pile diameter variations are chosen. The final pile diameter (1829 mm) has been chosen as an optimal pile diameter.

- Pile penetration variation from 40.0 m to 48.0 m. The optimal pile penetration 48.0 m is chosen.

The purpose of the extreme event analysis is to ensure that the jacket structure is able to withstand and transfer the loads to the piles. The jacket design is dependent on the applied loads which will influence the required base width at the bottom.

By varying the base width of the jacket, it is possible to determine the optimal diameter & penetration. The optimum pile diameter & penetration is chosen with respect to allowable utilization ratios and total weight of the structure from all above combinations.

The extreme wind loads provided in section 4.1.2are applied along and across (45°) the jacket structure together with the corresponding extreme wave as explained in section 4.1 for the preliminary design phase.

For fatigue analysis the damage equivalent loads provided in section 4.1.1 for an inverse slope of the S-N curve of m=5 and a reference number of cycles of $N_{ref} = 10^7$ are applied at the interface level together with the wave loads.

3.4 Final design phase

The design procedure for natural frequency, extreme event and fatigue analyses in the final design phase is explained in this section.

3.4.1 Natural Frequency Analysis

The natural frequency analysis is important to check whether the eigenfrequencies for the integrated structure (foundation + tower + RNA) are within the allowable limits according to the turbine characteristics. The eigenfrequencies must be outside the operational frequencies of the wind turbine (1P and 3P frequency bands including safety margin) in order to minimize dynamic amplifications especially from the aerodynamic loads. The 1P frequency refers to the rotor frequency and the 3P frequency refers to blade passing frequency (the set of three blades passing the tower).
Bottom mounted support structures are typically designed between the 1P and 3P frequencies of the turbine including an additional safety margin of 10%. This result in an allowable frequency band of 0.22 Hz – 0.31 Hz for the present design as stated in the design basis [1].

For offshore wind turbines on jacket foundations the eigenfrequency design in general tends to be more efficient when performed on the basis of tower variations rather than on basis of variations in the jacket structure, since jacket type foundations are relatively stiff and have relatively low masses compared to tubular steel towers. Especially, an increased tower length e.g. by an increased hub height while keeping the interface level unchanged can efficiently be used to reduce the eigenfrequency as shown in Figure 3–4 for the structure under consideration in this report.

The natural frequency analyses are based on characteristic soil conditions, i.e. partial safety factors for soil are set to unity. It should be noted, that the particular type of a concrete transition piece applied for this design has a significant influence on the modal properties while e.g. conical steel transition pieces are significantly softer and also less heavy.

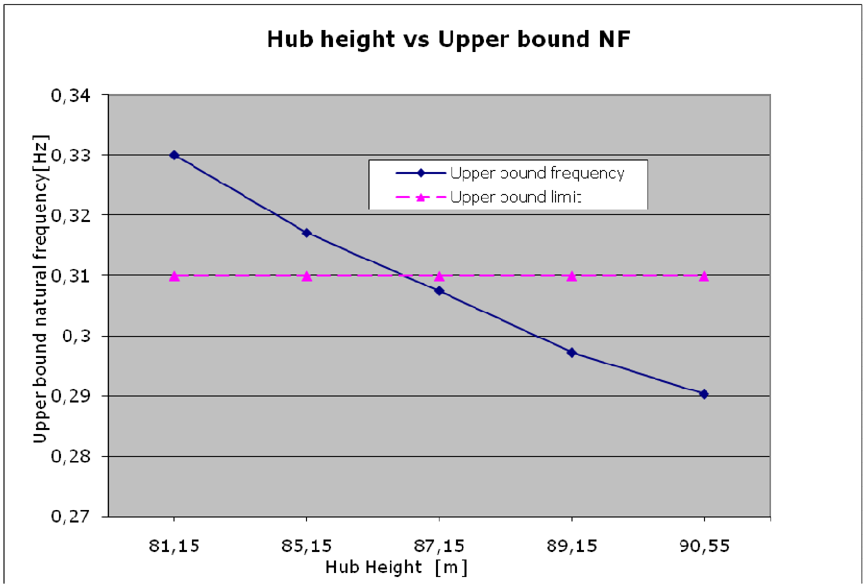


Figure 3–4: Natural frequency variation with the tower top

3.4.2 Extreme Event Analysis

General

The Extreme Event Analysis has been carried out using ROSAP **Error! Reference source not found.**.

The following steps briefly describe the analysis procedure. Each step is further described in the following sections.

- **Computer Model Geometry**

  Establish a three-dimensional space-frame computer model representing the jacket support structure, tower and RNA. Introduce secondary items acting as load carrying appurtenances.

- **Soil-pile Interaction**

  Establish the non-linear soil-curves (p-y, t-z, q-w) in compliance with API [9].

- **Load Generation**

  Determine the basic load cases and combine these in compliance with GL [2].

- **Static Analysis**

  Perform the static extreme event analysis with ROSA. This analysis is non-linear due to the non-linear soil behaviour. The results of the analysis are nodal displacements as well as sectional forces and moments in the entire structure.

- **Stress Check**

  Perform the steel stress check in accordance with NORSOK [7] for the steel members by the use of STRECH. The soil capacity is checked via the soil curves in accordance with the requirements in API [9].

Computer Model Geometry
The ROSA model of the jacket structure, transition piece and tower is modelled as one 3D-structure for the soil profiles as specified in [1]. In Figure 3–5, the model of the structure is shown in ROSA.

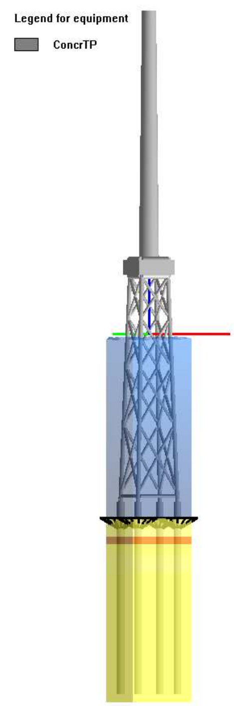


Figure 3–5: Integrated model of foundation and tower structure in ROSA

The tower structure has been modelled to obtain the correct stiffness and mass distribution for the global model, but no stress checks have been performed for the tower. Nacelle, rotor, tower accessories and secondary steel on the foundation have in general been modelled as appurtenances contributing with masses and wave load areas respectively.

Soil-pile Interaction
The soil-pile interaction is described in chapter 2.

Load Generation
The methods of introducing the various kinds of loading are described in the following.

Permanent Loads
The permanent loading on the structure has been modelled as self generated weight for all tubular elements of the jacket, transition piece and tower structure. All other masses have been applied as appurtenances.

Wind and Wave loads
The wave loads on the support structure are computer generated and based on Morison's equation and appropriate wave kinematics. Aerodynamic load time series provided by GH Bladed [14] are used for design purposes. The wave loads generated by ROSA including wave dynamics are combined with wind load time series in static analysis.

From the combined extreme loads ROSA searches for governing loads for each individual element in the structure.

Static Analysis
The ROSA program generates all relevant loads except the wind loads which are provided by bladed from the GH and based on the properties of the soil layers, the embedded part of the jacket pile is subdivided into a suitable number of elements. The soil-pile interface is described as a coupling between the nodes of these elements and the surrounding soil in terms of soil curves. In each node the non-linear lateral (p-y) and axial skin friction (t-z) curves are generated based on the properties of the pile and the actual soil layer. Furthermore, the tip resistance is generated using (q-w) curves.

ROSA finally determines the force distribution in the soil and displacements/rotations in all nodes and sectional forces/moments in all structural members.

Stress Check
Soil capacity and structural steel strength have been verified as described in Chapter 2.

Jacket structures transfer the overturning moments from the interface into the soil mainly by tension and compression in the piles. The extreme event analysis comprises investigations on the capacity of the structure and soil to withstand extreme loads. Basically it consists of two different analyses:

1) Analysis of pile-soil interaction using plastic soil conditions. This analysis evaluates the soil capacity.
2) Analysis of the pile steel using characteristic soil conditions. This analysis verifies the capacity of the pile steel.

3.4.3 Fatigue Analysis

The fatigue design approach is based on separate simulations of stochastic wind and stochastic waves under consideration of the aero-elastic interactions between both. These interactions are represented by an aerodynamic damping that influences the wave response.

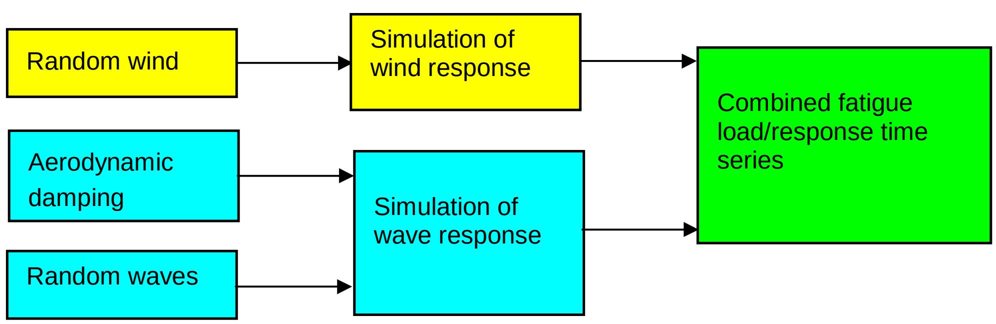


Figure 3–6: Fatigue analysis approach

Wind Loading
The wind loads are provided as load time series at the interface level for the given wind load combinations. The calculation approach is based on wind load time series inclusive of dynamics (inertial loads) determined by GH Bladed. The time series have been provided for different wind directions for the power production condition (DLC1.2, V=4-24 m /s) and for the idling conditions (DLC6.4, V=2; 26, 28 m/s).

Wave Loading

General
Time series for the wave loads are generated in ROSA, [3], according to the representative wave situations.

Generation of Surface Elevation Time Series
A time-series realisation of each selected scatter group sea state is performed assuming that the spectral density of the wave elevation can be described by the JONSWAP wave spectrum defined in DNV, [5].

By discretising the wave spectrum, free surface elevation time series are generated. The spectrum is discretised into a number of harmonic components in the frequency range 0-5 Hz. The discretisation is performed with a constant frequency interval Δf, which allows for application of the Fast Fourier Transform (FFT) technique. For each discrete frequency the corresponding harmonic wave amplitude is determined. In order to simulate an irregular sea surface, each harmonic component is assigned with a random phase. One time series is generated for each analysed scatter group. The duration of this time series introduces an initialization time sufficiently long to allow for transient vibrations to be damped out followed by simulation time of 100 times T_P.

Determination of Wave Kinematics
The calculation of the velocities and accelerations is performed in the frequency domain by means of transfer functions applied on the free surface spectrum. Based on linear wave theory (Airy), the velocities and accelerations for each harmonic component are calculated in discrete points from mudline to mean sea level. Time series of velocities and accelerations are generated by inverse FFT of the kinematic spectra. The discrete grid ranging from mudline to MWL, containing the kinematic components is afterwards modified by Wheeler stretching to cover the full interval between mudline and the actual free surface.

Wind wave directional combination
The jacket foundation structure is analysed with loading from different directional combinations of wind and waves.. illustrates the considered wind & wave directional combinations in the fatigue analysis. There are only 6 wave directions 12 wind directions due to symmetry.

Table 3–1: Wind wave directional combination

Wind /wave direction	N -0	NNE -30	ENE -60	E -90	ESE -120	SSE -150	S -180	SSW -210	WSW -240	W -270	WNW -300	NNW -330
N-0	X	X							X	X	X	X
NNE-30	X	X	X							X	X	X
ENE-60	X	X	X	X							X	X
E-90	X	X	X	X	X							X
ESE-120	X	X	X	X	X	X						
SSE-150		X	X	X	X	X	X					
S-180			X	X	X	X	X	X				
SSW-210				X	X	X	X	X	X			
WSW-240					X	X	X	X	X	X		
W-270						X	X	X	X	X	X	
WNW-300							X	X	X	X	X	X
NNW-330	X							X	X	X	X	X

Fatigue from combined wind and wave

Wind and waves are combined according to the directional wind - wave combinations as described above. The wind response time series and the wave response time series, both including dynamics, are superimposed and subsequently post-processed to determine the total fatigue damage during the simulated period of time. Based on the annual and directional probabilities of occurrence, the fatigue damage from the combined wind and wave simulation is scaled to annual damages.

The fatigue damage is determined using an S-N curve approach combined with appropriate stress concentration factors (SCFs, e.g. for the joints) calculated according Efthymiou [6]. The cumulative damage is determined on basis of Miner’s linear damage accumulation hypothesis. Eight equally spaced stress points around the circumference of the tubular section are considered. Nominal stresses due to axial forces, in-plane and out-of-plane bending moments are calculated based on Timoshenko beam theory. Variations of the nominal bending stresses along the circumference of the tubular section are considered to follow a cosine variation.

Hot spot stresses at each of the stress points are obtained by multiplying the above nominal stresses by SCFs.

3.4.4 Damping

For power production case where wind and wave are aligned (0°), the applied total damping is 4.5%, i.e. 4% of aerodynamic damping and 0.5% of structural damping. In case of 90° wind and wave misalignment, the applied total damping value is 0.5%. As an engineering approach, a cosine profile variation is reveals the aerodynamic damping values for the remaining misalignment. Applied total damping values for different wind and wave misalignment in fatigue analysis are shown in Table 3–2. For the idling case total damping value of 0.5% is applied i.e. only the structural damping value.

Table 3–2: Applied damping values for power production case

Wind & wave misalignment	Aerodyamic damping [%]	Structural damping [%]	Total damping [%]
-30	3.46	0.5	3.96
0	4.00	0.5	4.5
30	3.46	0.5	3.96
60	2.00	0.5	2.5
90	0.00	0.5	0.5
120	2.00	0.5	2.5

3.4.5 Hydrodynamic coefficients

The hydrodynamic coefficients are calculated in ROSA [3]] for the individual elements dependent on the instantaneous Reynolds number (Re) and Keulegan - Carpenter number (KC). The maximum coefficients including marine growth are shown in Table 3–3 for the extreme and fatigue cases

Table 3–3: Hydrodynamic coefficients

Fatigue		Extreme	
Cd	Cm	Cd	Cm
0.65	2.0	0.65	2.0

4. Design Load Cases

In this section, the implemented design load cases for the pre-design and final design phase are explained and the aerodynamic loads for the preliminary design phase are shown.

In the different phases of the support structure design process, different approaches are followed with respect to the load analysis. As a first step, a preliminary design of the jacket is done in order to identify a first set of structural dimensions. In a second step, the structure will be analysed in more detail. For both cases the implemented design load cases are different. In the following the implemented load cases according to current standards are described and results for the turbine loads are shown. In all cases the used turbine is the Upwind reference turbine, which is new model based on a baseline design of NREL [11] and an industry-standard controller [12].

4.1 Wind loads for preliminary design phase

In the preliminary design phase of the jacket design, damage equivalent loads are applied to the structural design tool as described in section 3.1.2.
The generation of these preliminary turbine loads are done with the aid of an equivalent turbine model in the design water depths of 50m. For the load calculations, a stiff monopile will be used, which has the target eigenfrequency of 0.29Hz. This approach is valid here, as no hydrodynamic loading will be present (calm sea). By using a standard tubular steel tower of 68m and a vertical offset in the nacelle of 2.4m on top of the transitions piece with a elevation of 14.8m above sea level, the support structure design results in a hub height of 85m.

Based on the IEC-61400-3 standard [10], different load cases are simulated. As the generated loads shall deal as an input for the turbine loads only, no hydrodynamic effects are included (calm sea). Furthermore only a reduced number of design-driving load cases is simulated. The simulated load cases are (the detailed descriptions of the load cases can be found in Appendix X - Load case description for preliminary design phase):

- dlc1.2 Power production + normal turbulence (Fatigue)
- dlc6.4 Idling + normal turbulence (Fatigue)

- dlc1.3 Power production + extreme turbulence (Extreme)
- dlc1.4 Power production + extreme coherent gust with change of direction (Extreme)
- dlc6.2 Idling with loss of electrical network, incl. dlc6.1 for wind direction=0° (Extreme)

The aero-elastic simulations are performed by the GH Bladed code [13]. All load simulations include:

- tower shadow
- 2 side-to-side and 2 fore-aft tower modes and 6 out of plane and 5 in plane blade modes
- three dimensional Kaimal turbulent wind field
- idling with pitch angle of 90°

The loads are given at the top of the transition piece, where the tower is mounted to the sub-structure. The coordinate system is defined as shown below.

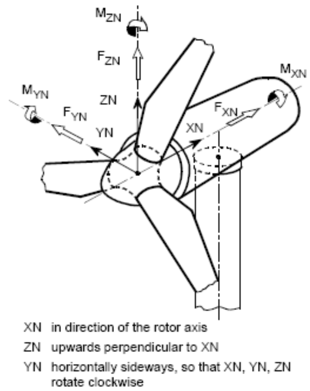


Figure 4–1: Definition of coordinates [2]

4.1.1 Fatigue loads

The fatigue loads are based on the wind speed distribution shown in the design basis [1]. Based on this distribution, an occurrence per wind speed bin is derived.

In the following, damage equivalent loads (DEL) are used to equate the fatigue damage represented by rain flow cycle counted data to that caused by a single stress range repeating at a single frequency. The method is based on Miner's rule, which gives the damage equivalent stress by the following equation:

$$L_N = \sqrt[m]{\frac{\sum L_i^m n_i}{N}}$$

with L_N - equivalent stress for N cycles
L_i - stress range bin i
n_i - number of rain flow cycles at stress range bin i
m - negative inverse of the slope on the material's Wöhler curve
N - number of cycle repetitions in the turbine lifetime

The stress, L_i, depends upon geometry of the structure under consideration. It is assumed that stress is proportional to load, therefore it is quite acceptable to use load instead of stress in the above equation. For simplicity, L_i and n_i have been derived from the one-dimensional table with no correction to account for the fatigue damage due to mean stresses. The equivalent loads (in kNm and kN) are presented overleaf for each load component assuming different frequencies, hence cycles in turbine lifetime. The turbine lifetime is set to be 20 years. The values are given for the integration of all design load cases, which are in this case dlc1.2 and dlc6.4 according to IEC-61400-3 [10].
Transient fatigue load cases (such as start, stop and faults) are not taken into account, as they are mainly important for components with large S-N-slopes (such as m=10 for blades) due to their large loading amplitudes. As the following design loads are for the support structure only, with its small S-N-slopes of 4, this approach is valid. Furthermore, the fatigue loads due not take turbine wake effects into account. However, this will be part of the simulations for the final result, where higher turbulence intensities are defined.
In the following, the results for the overturning moment, My, are shown exemplary. The resulting tables can be found in Appendix XII - Fatigue loads (as DEL) for preliminary design.

Table 4–1: Lifetime weighted equivalent loads: Support structure at My at 14.75m MSL

	f [Hz]						
	0,0158	0,1	0,27	0,29	0,31	1	10
m \ N	1.0 e7	6.312 e7	1.704 e8	1.83 e8	1.957 e8	6.312 e8	6.312 e9
3	31687,2	17146,4	12313,6	12023,8	11759,4	7958,65	3694,08
4	26263,4	16569,8	12926,4	12697,5	12487,5	9317,88	5239,83
5	24291,2	16804,4	13776,8	13581,3	13401,4	10602,8	6689,94
6	23592,4	17354,6	14706,9	14532,8	14372,2	11823,6	8055,31
7	23468	18037,3	15651,2	15492,3	15345,4	12981,2	9342,37
8	23627,4	18767,2	16576	16428,6	16292,2	14073,4	10553,6
9	23930,5	19500,6	17463	17324,9	17197	15098,6	11690,3
10	24304	20214,6	18303,2	18172,9	18052,1	16057	12754,5
11	24707,8	20897,5	19093,2	18969,6	18854,9	16950,6	13749,1
12	25119,3	21544,1	19832,7	19715	19605,7	17782,6	14677,9

4.1.2 Extreme loads

The following extreme load overview illustrates the maximum and minimum value of each load along with the contemporaneous value of associated loads. Within these values, the safety factors have already been applied. The following extreme load histograms are presenting the maximum absolute load for each load case.

Table 4–2: Ultimate loads: Support structure at +14.75m MSL

			Fx	**Fy**	**Fz**	**Mx**	**My**	**Mz**
		Load case	**kN**	**kN**	**kN**	**kNm**	**kNm**	**kNm**
Fx	**Max**	**1.3cb_2**	1400.3	-16.6	-8100.2	7517.0	84230	-1433.7
Fx	**Min**	**6.2g_2**	-799.6	-428.1	-6394.0	27640	-50257	1935.4
Fy	**Max**	**6.2a_3**	355.2	739.0	-6029.2	-45262	8992.6	-2097.3
Fy	**Min**	**6.2e_1**	-227.8	-1892.2	-6362.2	115619	-20267	8705.8
Fz	**Max**	**6.2b_1**	425.5	-724.0	-5808.5	40957	11863	8747.5
Fz	**Min**	**1.3ec_3**	139.8	-56.3	-8397.8	11696	-4314.7	10602
Mx	**Max**	**6.2e_1**	-248.7	-1890.9	-6371.5	115715	-22115	8509.1
Mx	**Min**	**6.2a_3**	355.2	739.0	-6029.2	-45262	8992.6	-2097.3
My	**Max**	**1.3ba_3**	1342.4	-42.3	-8054.9	9750.7	87523	1118.6
My	**Min**	**6.2g_1**	-796.4	-38.8	-6504.8	2649.6	-54566	607.1
Mz	**Max**	**1.3ec_3**	356.3	-132.0	-8260.1	13015	18097	13786
Mz	**Min**	**1.3ea_3**	405.4	62.4	-7757.7	488.8	12473	-17417

4.2 Wind loads for final design phase

In the following the load case assumptions for the final design phase are discussed. The design process itself is described in section 3.4. The focus is on reducing the full set of required load cases according to standards to a range of cases, which will dominate the design for such a deep-water jacket design. Due to the non-rotational symmetry of the space frame jacket structure, wind and wave orientation influence the overall design. For fatigue design two methodologies may be applied:

1) Simplified method considering reduced directionality, but two support structure orientations

   The two support structure orientations (0° and 45°) are defined with regard to the rotor axis, while the rotor axis is assumed collinear with the wind direction. For conservative simplicity it is assumed that the rotor axis points north. The support is oriented accordingly to N (0°) or NE (45°). Figure 4–2 illus trates the 0° and 45° orientation by giving top views on the jacket.

Figure 4–2: Different orientation w.r.t. environmental loading

2) Consideration of site environmental conditions for directional wind and wave distribution and directional load analysis

In order to reduce the sets of load cases, the for the extreme load calculation the first approach will be followed by using a reduced set of wind-wave-misalignments but by taking two different support structure orientations into account. For the fatigue load analyses, the more detailed second approach is done, where site-specific directionalities are taken into account.

In the following the definitions for both – the fatigue and extreme load analysis are explained.

4.2.1 Fatigue load analysis

As described, the fatigue load analysis assumes wind-wave misalignment. Thus, it more precisely represents the site conditions while it reduces the amount of conservativeness by increasing the computational effort. Furthermore a technical availability of 100% is considered, which tends to be conservative for jackets [14]. This means that within the power production regions (4-24m/s), dlc1.2 according to IEC-61400-3 [10] is taken, and idling (dlc6.4) below cut-in and above cut-out. As for the preliminary design, transient fatigue load cases (such as start, stop and faults) are not taken into account, as they are mainly important for components with large S-N-slopes (such as m=10 for blades) due to their large loading amplitudes. As the following design loads are for the support structure only, with its small S-N-slopes of 4, this approach is valid.
The settings for the load cases include the general assumptions as given in Table 4–3 (the detailed description can be found in Appendix XI - Load case description for final design phase):

Table 4–3: General assumptions for fatigue load analysis

Conditions	
Wind speed	10-min turbulent wind, incl. wake effects (6 seeds)
Yaw	+ 8° yaw error
Misalignment	Wind directions iterated from 0-330° a nd wave directions relative to wind from -30°-120°
Support structure orientation	Structure pointing with 2-legs North (0°)

Due to the setup of iterating the wind around the whole structure (and the wave directions accordingly), the simulation of two different support structure orientations can be avoided. For the occurrences per simulated wind speed bin, the values of the design basis are taken [1].

4.2.2 Extreme load analysis

As already stated above, for the extreme load simulation some simplifications are defined in order to reduce the set of simulations. In a first approach, the generally design driving load cases are taken from the standard. Nevertheless, dlc6.1 and dlc6.2 are of high computational effort. Unfortunately, the effort is mandatory as dlc6.1 and dlc6.2 are assumed to be the main design drivers for the jacket structure.

For all other design load cases proposed (dlc2.2; dlc1.6; dlc2.3) the IEC standard states wind aligned with waves if both approach from the worst case direction regarding loads. Thus, no wind wave misalignment but two support structure orientations shall be analysed for those load cases. The simulated cases can be summarized as:

- dlc1.6 Power production + severe sea state
- dlc2.2 Power production + turbine fault case + occurrence of an extreme wind gust
- dlc2.3 Power production + extreme turbulence (Extreme)
- dlc6.1 Idling + extreme maximal waves
- dlc6.2 Idling + extreme reduced waves

Assumptions for dlc1.6

The load case dlc1.6 represents power production in turbulent wind conditions and a 50-year sea state. For conservative reason in this reduced set of cases, an embedded wave with a maximum 1-year wave height ($H_{max,1}$) may be assumed. The settings for the load case include the general assumptions as given in Table 4–4 (the detailed description can be found in Appendix XI - Load case description for final design phase):

Table 4–4: General assumptions for dlc1.6

Conditions	
Wind speed	1-min turbulent wind (6 seeds)
Yaw	+ 8° yaw error
Misalignment	Wind and waves aligned
Support structure orientation	Two structural positions (0° and 45°)

Assumptions for dlc2.2

The setup is comparable to dlc1.2 for the fatigue analysis. In the here conducted reduced set of cases, safety system fault computations may be limited to one significant pitch fault. It is conservatively proposed to assume all blades turn to fine (with a reasonable average pitch rate) until the safety system is activated again by reaching the safety system overspeed limit. The settings for the load case include the general assumptions as given in Table 4–5 (the detailed description can be found in Appendix XI - Load case description for final design phase):

Table 4–5: General assumptions for dlc2.2

Conditions	
Wind speed	1-min turbulent wind (6 seeds)
Yaw	+ 8° yaw error
Misalignment	Wind and waves aligned
Fault case	Collective pitch error
Support structure orientation	Two structural positions (0° and 45°)

Assumptions for dlc2.3

The load case dlc2.3 represents a load situation of a turbine in power production during a one-year gust (EOG1) that looses the generator torque due to a generator cut-out from the grid. The grid loss shall be considered at the three time instants, lowest wind speed, highest gust acceleration, maximum wind speed (according to [2]). Furthermore, the rotor start positions shall vary from 0 deg to 90 deg (in 30 deg steps). The settings for the load case include the general assumptions as given in Table 4–6 (the detailed description can be found in Appendix XI - Load case description for final design phase):

Table 4–6: General assumptions for dlc2.3

Conditions	
Wind speed	Steady wind with 1-yrs extreme gust
Yaw	+ 8° yaw error
Rotor positions	Four different start positions (0°, 30°, 60°, 90°)
Misalignment	Wind and waves aligned
Fault case	Grid loss
Wind extreme event	Gust at three different times of the grid loss (beginning, mid and end of the gust)
Support structure orientation	Two structural positions (0° and 45°)

Assumptions for dlc6.1

Here the turbine is idling in storm conditions with active controls (i.e. the yaw system is still able to move the nacelle into the wind). The conditions coincide with an extreme sea state and the 50yrs extreme wave. However, the cases are split up into three sub-groups (a-c) according to the combination of wind and wave conditions. In order to reduce the amount of simulation efforts, the worst wind and wave conditions are brought together.

Here a turbulent wind with a minimum longitudinal turbulence intensity of 11 % shall be considered in combination with at least 6 seeds for wind and sea states, according to the IEC standard. Wind-wave misalignment shall include site-specific values derived during fatigue analysis. The 50yrs maximum wave is simulated as constrained wave within the 50yrs irregular sea state. The settings for the load case include the general assumptions as given in Table 4–6 (the detailed description can be found in Appendix XI - Load case description for final design phase):

Table 4–7: General assumptions for dlc6.1

Conditions	
Wind speed	1-min turbulent wind (6 seeds)
Yaw	+ 8° yaw error
Misalignment	Misalignment for 0°-150° (in 30° steps), where wind is always action from North (0°) on the turbine
Support structure orientation	Two structural positions (0° and 45°)

Assumptions for dlc6.2

This load case is similar to dlc6.1, but no grid access is available. This means that the nacelle position cannot be changed anymore. The result is that the wind can face the idling rotor from any possible direction.

As for dlc6.1, a reduced set of runs for dlc6.2 is proposed. It will be assumed that dlc6.1 identifies the worst support structure positions in terms of incoming waves. Therefore the support structure position and wave direction with the highest loads in dlc6.1 will be kept constant in dlc6.2. This set is then used to simulate all possible incoming wind directions due to the grid loss (stuck nacelle position). The settings for the load case include the general assumptions as given in Table 4–8 (the detailed description can be found in Appendix XI - Load case description for final design phase):

Table 4–8: General assumptions for dlc6.2

Conditions	
Wind speed	1-min turbulent wind (6 seeds)
Yaw	Yaw error fault for 0°-180° (in steps of 30°)
Misalignment	Worst misalignment according to dlc6.1
Support structure orientation	Worst structural position according to dlc6.1

5. Design Results

In this chapter the results for the preliminary design phase and the final design phase are presented. For the preliminary design phase the dimensions following from this phase and the reasoning to come to these dimensions are discussed. The discussion of the final design phase is more extensive. The final dimensions are presented and the various steps in the design, including the natural frequency analysis, the extreme event analysis and the fatigue analysis are described.

5.1 Selection of the Preliminary Design Structural Dimensions

An investigation has been carried out on support structure 1st natural frequency with varying jacket bottom width and remaining dimensions of the jacket foundation are kept constant.
Table 5–1 summarizes the found 1st natural frequencies values for different jacket bottom widths.

Table 5–1: 1st natural frequencies for different jacket bottom base widths

Jacket bottom base width [m]	1st Natural frequency [Hz]
11.0	0.2762
12.0	0.2838
13.0	0.2903
14.0	0.2960
15.0	0.3009
16.0	0.3051

The total structural cost of the jacket foundation has been estimated for different jacket bottom base widths and remaining dimensions of the jacket foundation are kept constant. The following factors have been applied on the single component masses for the cost estimates of the individual foundations:

Jacket : 4.00 Euro/kg
Piles : 2.00 Euro/kg

Table 5–2 summarizes weight and cost distribution for different jacket bottom base widths.

Table 5–2: Total mass and cost for different jacket bottom base widths

Jacket bottom base width [m]		11.0	12.0	13.0	14.0	15.0	16.0
Piles	Mass [tons]	380.04	380.04	380.04	380.04	380.04	380.04
Jacket		569.87	575.42	581.00	586.63	592.32	598.07
Total [piles+jacket]		949.91	959.46	965.04	970.67	976.36	982.11
Piles	Cost Estimate [kEuro]	760.08	768.08	768.08	768.08	768.08	768.08
Jacket		2279.48	2301.68	2324.00	2346.52	2369.28	2392.28
Total [piles+jacket]		3039.56	3069.76	3092.08	3114.6	3137.36	3160.36

The 1st natural frequency and total cost variation for different jacket bottom base width is shown in Figure 5–1. From this, it can be concluded that the preliminary design jacket bottom base width of 12.0 m should be chosen in order to meet the requirements from the support structure 1st natural frequency and total optimal cost of the structural steel.

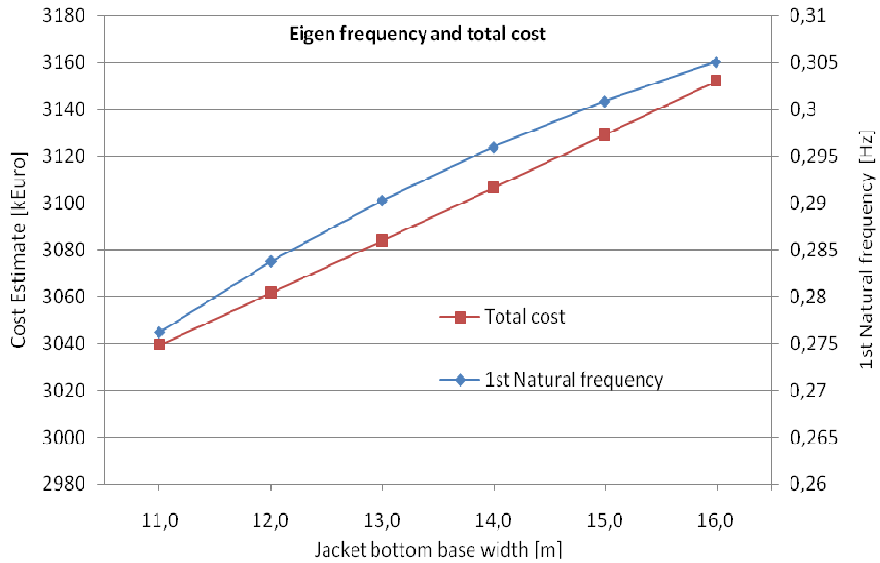


Figure 5–1: Eigen frequency and total cost for different jacket bottom base widths

The overall design summary of the preliminary design jacket foundation structure is presented in Table 5–3.

Table 5–3: Preliminary design jacket design summary with 50.0m water depth at soft soil conditions

Base width		Pile Diameter	Pile Penetration	Jacket only Weight (excl. piles)	All 4 piles	Total Jacket Weight (incl. piles)
Bottom	Top					
12 m	8 m	1829 mm/74''	48 m	576 tons	380 tons	956 tons

The requirements of the structural steel utilization and minimum fatigue lives are at the acceptable limit for the preliminary design phase. Detailed explanation on design results for natural frequency, extreme and fatigue analyses can be found in the following sections.

5.2 Final Design Structural Dimensions

The overall design summary of the jacket foundation structure for final design is presented in Table 5–4 for 50m water depth w.r.t to MSL for the soft soil conditions according to the Design Basis [1]. Node and element denotations of the FE model are shown in Appendix III - Node and elements names.

Table 5–4: Final design jacket design summary with 50.0m water depth at soft soil conditions

Base width		Pile Diameter	Pile Penetration	Jacket only Weight (excl. piles)	All 4 piles	Total Jacket Weight (incl. piles)
Bottom	Top					
12 m	8 m	2082 mm/82''	48 m	545 tons	438 tons	983 tons

The wall thicknesses of the piles are 65 mm in the upper part of the piles and 28 mm at the lower part as shown in Appendix IV - Structural drawings for the jacket.

According to DNV recommendations [5], it is reasonable to assume local scour depth of 1.3 times the pile diameter for sand if no detailed scour information is available. In this study a local scour of 1.3 times the pile diameter and no global scour is considered for the jacket design. The obtained 1st natural frequency of the entire structure is 0.291 Hz which is in the allowable range. The corresponding analysis results are shown in Appendix II - Natural frequency analysis.

All structural dimensions of the primary steel are summarized in Appendix IV - Structural drawings for the jacket.

From previous experience on jacket design, the estimated concrete transition piece weight 666 tons is considered in this design. Table 5–5 shows the maximum utilization ratios (ULR) as well as minimum fatigue lives for members and joints. The utilization ratios are within the allowable limit for both, the soil capacity and the steel capacity. The maximum utilization ratios are found at the bottom X-braces for the members and joints, where as the minimum fatigue lives are found at the top of the jacket joints. The steel member utilization ratios and tubular joints utilization ratios are shown in the Appendix V - Utilization ratios for members and joints. Fatigue lives for the members and tubular joints are shown in Appendix VI - Fatigue lives for members and joints.

Table 5–5: Maximum utilization ratios (ULR) and minimum fatigue lives for members, joints

	Members	Joints
Steel Utilization ratios	0.99	0.91
Minimum fatigue lives	152	23

5.2.1 Natural Frequency Analysis

The natural frequency analysis (NFA) has been carried out for inflexible foundation with the jacket legs flooded and without consideration of corrosion and marine growth in order to get the upper bound eigenfrequencies. The NFA has been carried out also for the "rigid foundation" (Structure clamped at the bottom) as a reference case to quantify the major influence of the superstructure on the modal properties in comparison to the inflexible configuration.

Table 5–6 summarizes the eigenmode values for the rigid structure and for the inflexible configuration. The corresponding top view mode shapes are shown in Figure 5–2 and the side view mode shapes are shown in Appendix II - Natural frequency analysis. It can be seen that the modal displacements of the tower are large, while the jacket only deforms slightly for the first 2 eigenmodes.

Table 5–6: Eigenfrequency values for rigid and inflexible foundation

Tower mode	Natural frequency [Hz]	
	Rigid foundation	With marine growth, LJF, added mass, flooded jacket leg members, inflexible foundation
1st Tower Fore-Aft	0.310	0.291
1st Tower Side-to-Side	0.308	0.290
2nd Tower Fore-Aft	1.104	0.813
2nd Tower Side-to-Side	1.088	0.806
3rd Tower Fore-Aft	2.622	2.001
3rd Tower Side-to-Side	2.375	1.936
1st Tower Torsion	1.291	1.038

2nd Tower Torsion	3.187	2.040

From Table 5–7, it can be seen that the resulting natural frequencies for rigid foundation are relatively close to the natural frequencies of the inflexible foundation case for the first eigen mode, indicating a dominating influence of the tower and rotor-nacelle-assembly properties on the modal properties.

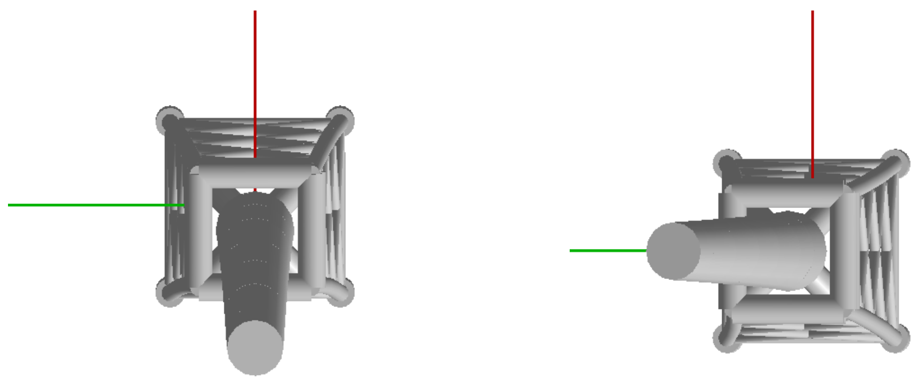
The reason for having lower natural frequencies for inflexible foundation in Table 5–7 as compared to the rigid foundation is due to the fact that the foundation has no flexibility at all (Structure clamped at the bottom) and therefore the eigenfrequencies are higher.

A mass comparison has been done for inflexible foundation without local joint flexibility (LJF) and with local joint flexibility. The corresponding masses for the support structure are shown in Table 5–7 where lower weight is observed for jacket with LJF. This is due to the fact that in LJF, the braces are cut off at the chord surface.

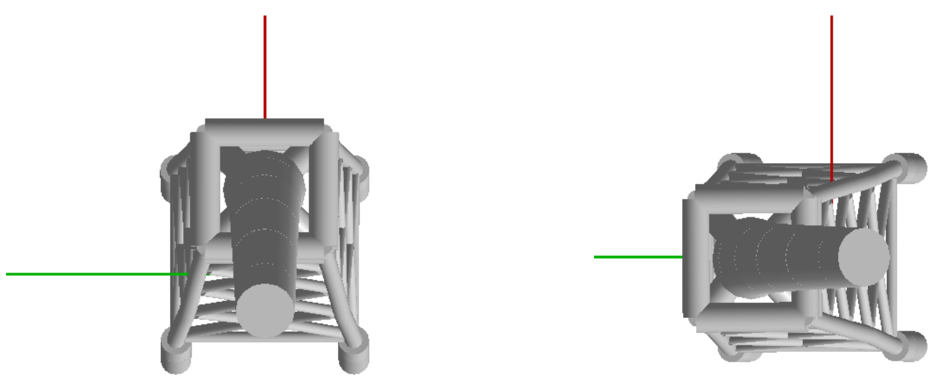
Table 5–7: Jacket structural masses

	Without LJF [tons]	With LJF [tons]
Jacket	583.6	545
Pile	438.2	438
Tower	215.5	216
Transition Piece	666.0	666
Appurtenances	345.7	346

Post processing results from ROSA from the natural frequency analysis are shown in the Appendix II - Natural frequency analysis.



(a) 1st tower fore- aft (b) 1st tower side to side



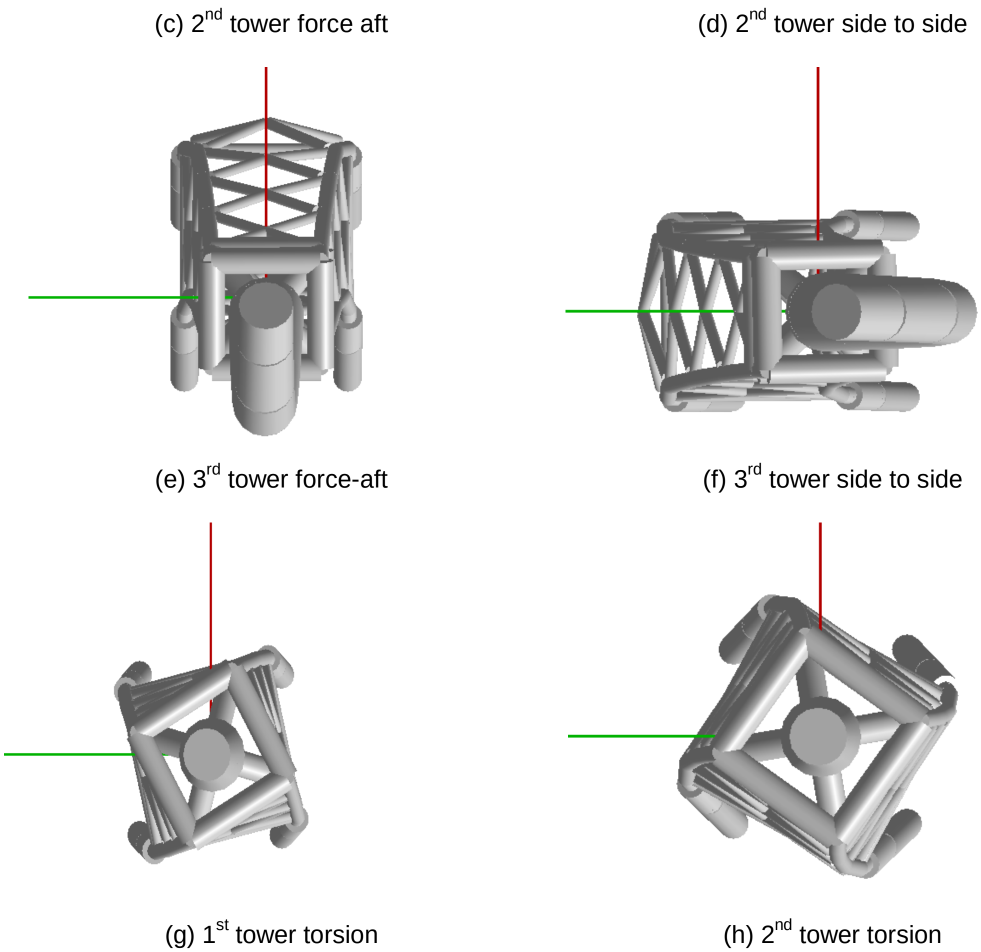


Figure 5–2: First 5 Eigen mode shapes for 50 m of water depth

5.2.2 Extreme Event Analysis

Design load cases

The design load cases are implemented as described in section 4.2. The considered load cases for extreme and fatigue are shown in Table 5–8. These reduced set of extreme and fatigue load cases are expected to drive support structure loads.

Table 5–8: Design load cases used for extreme and fatigue analysis (ULS/FLS)

	Description	Type
DLC1.6	Power production in 50 year sea state	ULS
DLC2.2	Safety system fault	ULS
DLC2.3	Generator cut-out	ULS
DLC6.1	Idling in storm	ULS
DLC6.2	Idling in storm during grid loss	ULS
DLC1.2	Power production	FLS
DLC6.4	Idling	FLS

Table 5–9 presents the design extreme load case DLC6.1 used for the combined wind and wave load calculations for final design. A description regarding combined wind and wave load calculations for design load cases is shown in the Appendix XI - Load case description for final design phase.

Table 5–9: Combined wind and wave conditions used for extreme load DLC6.1

Design load case (DLC):	6.1					
Operating condition:	Idling					
Wind conditions:	Extreme wind model (turbulent) ($V_{hub} = V_{50}$)					
Sea conditions:	Extreme sea state ($H_s = H_{s50}$), extreme current model (50yr return), EWLR					
Type of analysis:	Ultimate					
Partial safety factors:	Normal					
Description of simulations:						
	Wind conditions		Wave conditions			
	Mean wind speed (m/s)	Longitudinal turbulence intensity (%)	Significant wave height (m)	Peak spectral period (s)	Yaw error	Wind/wave misalignment
6.1a1-6	42.73	11.00	9.40	13.70	8 deg	0 deg
6.1b1-6						30 deg
6.1c1-6						60 deg
6.1d1-6						90 deg
6.1e1-6						120 deg
6.1f1-6						150 deg
Comments:	Three dimensional three component Kaimal turbulent wind field (10 min sample). First 20s of output discarded to allow initial transients to decay Six turbulent wind seeds per wind speed bin (indexed 1-6) Simulations run with support structure at 0deg and 45deg orientation from North Wind gradient exponent (exponential model), $\alpha = 0.11$ Extreme sea state with irregular waves defined using Jonswap spectrum with $\gamma = 3.3$ Extreme current with 50-year return period of 1.2 m/s applied 50-year extreme water level (HSWL) of 53.29m Constrained extreme non-linear wave included in irregular wave history: - Constrained wave height = H50 = 17.48m - Constrained wave period = T50 = 10.87s - Time of constrained wave crest: 100s The characteristic loads for each load case group are calculated as the mean of the maxima from each of the six seeds.					

From the extreme analysis it can be concluded that the load case DLC6.1 is the governing load for the jacket design.

Combined load cases

Extreme wind loads provided from GH Bladed [14] are combined with corresponding sea states for each of the extreme load case (e.g. DLC 6.1, 6.2 etc.) using ROSA [3]. The maximum absolute wind forces, moments defined at interface are added to loads from irregular waves and the combined loads have subsequently been used to search for the governing loads in all individual elements in the structure.

In total, 72 different wind load combinations are used for DLC6.1 and all the combinations are shown in Appendix IX - DLC6.1 Load combinations.

Governing load case

The governing load combinations of DLC6.1 for most of the jacket members with respect to wind and wave directions are shown in
Table 5–10. For a four legged jacket, the smallest pile capacities can typically be found in a diagonal direction, which means that the highest utilizations of the jacket legs can be found in the diagonal direction. This is why the load case DLC6.1 with wind from 45° and waves from 195° results in the most severe load. The locations of the appurtenances do also have a significant influence on the governing load direction.

For load case DLC6.2 despite of support structure orientation and wind-wave misalignment, the remaining load setup is same as DLC6.1 as described earlier. It is assumed that DLC6.1 identifies the worst support structure positions in terms of incoming waves. The support orientation shall be determined for DLC6.2 by the support orientation for DLC6.1 that resulted in maximum loads. See a full description on governing design load cases in design basis.

The support structure orientation of 45° (wind from 45°) and wind-wave misalignment of 150° (with wind from 45° and waves from 195°) results in maximum loads for DLC6.1. The governing wind and wave load direction from DLC6.1 is used to calculate the maximum loads for DLC6.2.

Table 5–10: Governing load occurrence on jacket member for DLC6.1

Compass Direction		Rosa Direction		Amount	
Wind	Wave	Wind	Wave	Combined load case no.	Occurrence on jacket member
450 (NNE-ENE)	1950(S-SSW)	1350	3450	332	48
450(NNE-ENE)	1650 (SSE-S)	1350	150	73	40
450(NNE-ENE)	1350 (ESE-SSE)	1350	450	66	35

The extreme event analysis showed that the governing loads for most of the jacket elements and joints result from DLC6.1. This is summarized in more detail together with individual element utilizations and their corresponding governing load cases in Appendix VIII - Elements utilization ratios for DLC load cases

The maximum shear force and overturning moment at interface are shown in Table 5–11. The extreme event analysis comprises investigations on the capacities of the structure and soil to withstand extreme loads. Analyses of pile-soil interactions are performed on the basis of plastic soil conditions and analyses of the pile steel and jacket steel are performed on the basis of characteristic soil conditions. In addition, the pile steel and jacket steel utilization ratios are also checked with the hard soil profile provided in the Appendix V - Utilization ratios for members and joints. However, the largest steel utilizations occur for the soft soil conditions.

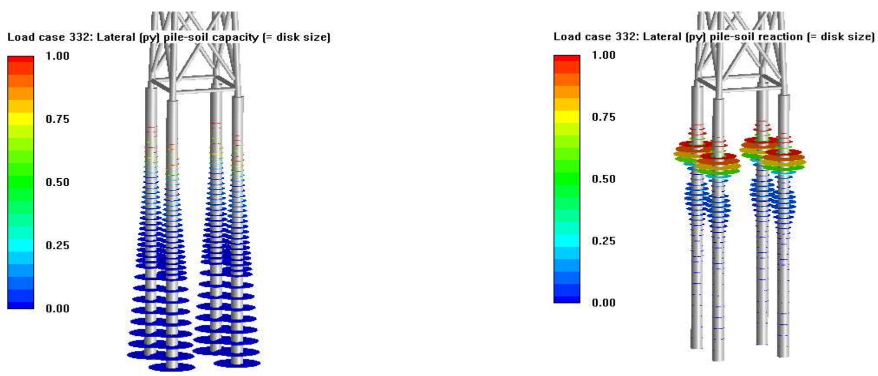
Table 5–11: Maximum resultant shear force and moment at interface.

	DLC6.1	
Level	Maximum shear force [kN]	Maximum moment [kNm]
Interface	900	57146

Design results

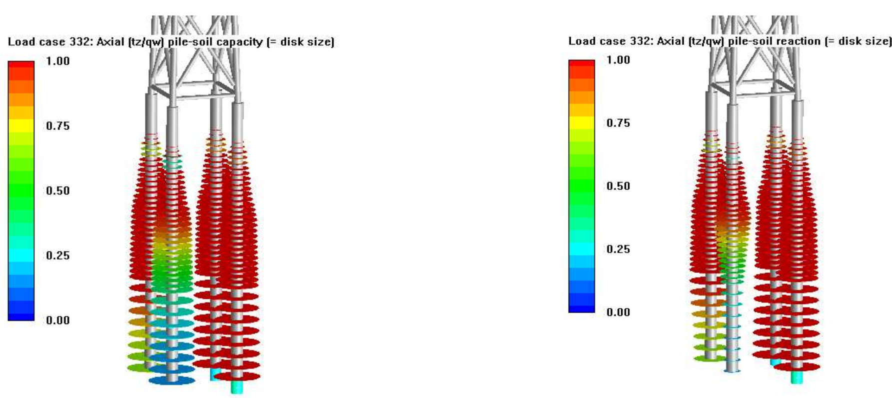
The capacities of the piles in the soil are checked under consideration of plastic soil conditions as stated in section 2.2. Steel stresses in the jacket structure are checked under consideration of characteristic soil conditions. Furthermore punching shear stresses are checked for all tubular joints by using Rambøll's in- house program TUBJOI [3].

In Figure 5–3 & Figure 5–4, lateral and axial soil capacities and reactions are shown for the worst load condition (DLC6.1, load combination 332). The size of each disk represents the reaction in the soil while the colour represents the utilisation of that particular soil layer. It can be seen that the soil just below mudline is fully utilized due to rather high deformations and low capacities of the corresponding layers.



(a) (b)

Figure 5–3: Lateral soil (a) capacities and (b) reactions for worst load combination 332



(a) (b)

Figure 5–4: Axial soil (a) reactions and (b) capacities for worst load combination 332

The steel utilization plots for members, joints and piles are shown in Figure 5–5. Individual steel member utilization ratios and tubular joint utilization ratios can be found in Appendix V - Utilization ratios for members and joints.

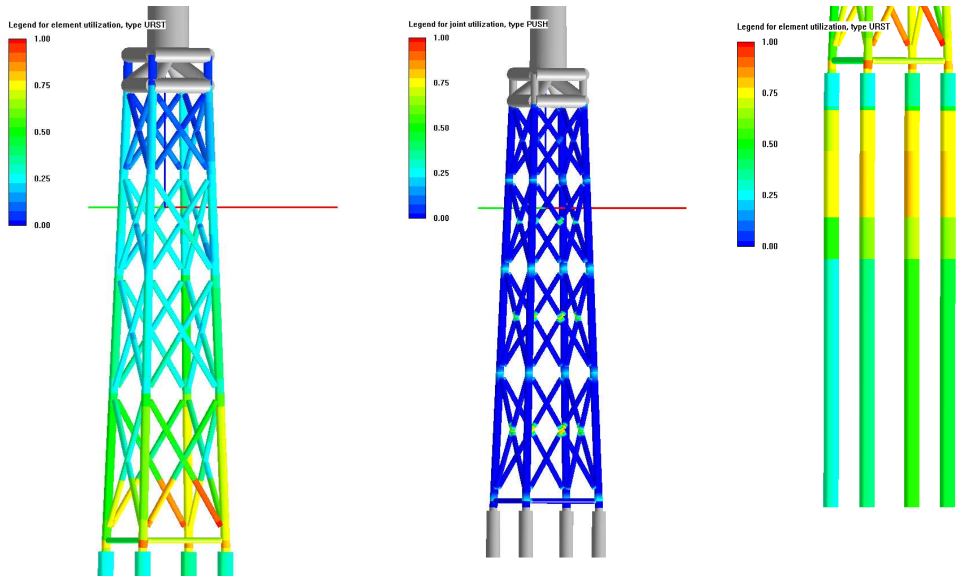


Figure 5–5: Maximum utilization ratios for elements, tubular joints and piles

5.2.3 Fatigue Analysis

Design results

A fatigue analysis is performed for the tubular joints, elements and attachments in the jacket structure. Fatigue lives are improved by increasing the can section thickness at middle X-braces. The tubular joints, upper parts of the jacket legs and bracings are optimized with respect to the fatigue loads. The tubular joints, elements and circumferential welds are analysed on basis of a GL-90 curve [2], i.e. without weld toe grinding. Boat landing attachment fatigue lives are analysed under consideration of a mean stress reduction factor of 0.77 on basis of GL [2] for GL- 63 curve. The fatigue analysis shows that the fatigue lives are above the minimum fatigue life of 20 years.

It is important to check the fatigue lives for both joints as well as for members in order to extract minimum fatigue lives in entire structure. The minimum fatigue lives are observed for joints at top x brace where the chord and brace are met. The maximum damages can be seen where wind and wave coming from SSW. The maximum fatigue lives for joints are observed at bottom x brace. The maximum and minimum fatigue lives for joints are shown in Figure 5–6 and Figure 5–7 respectively.

Fatigue lives for individual members and joints are shown in Appendix VI - Fatigue lives for members and joints.

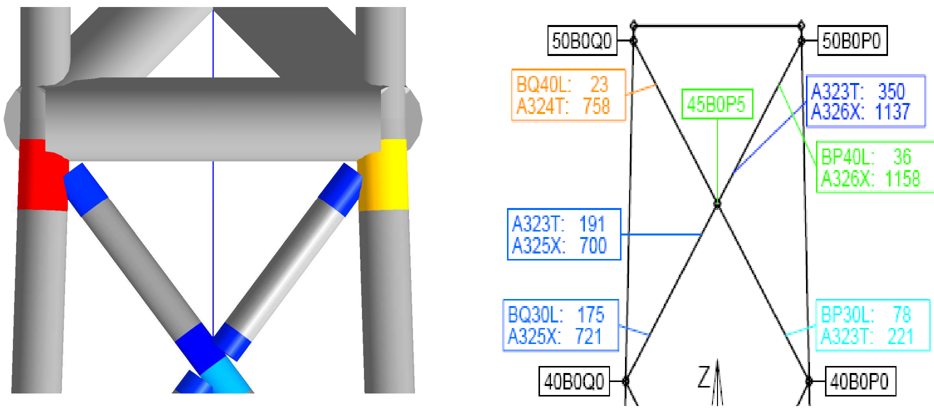


Figure 5–6: Maximum joint fatigue lives at top x-brace

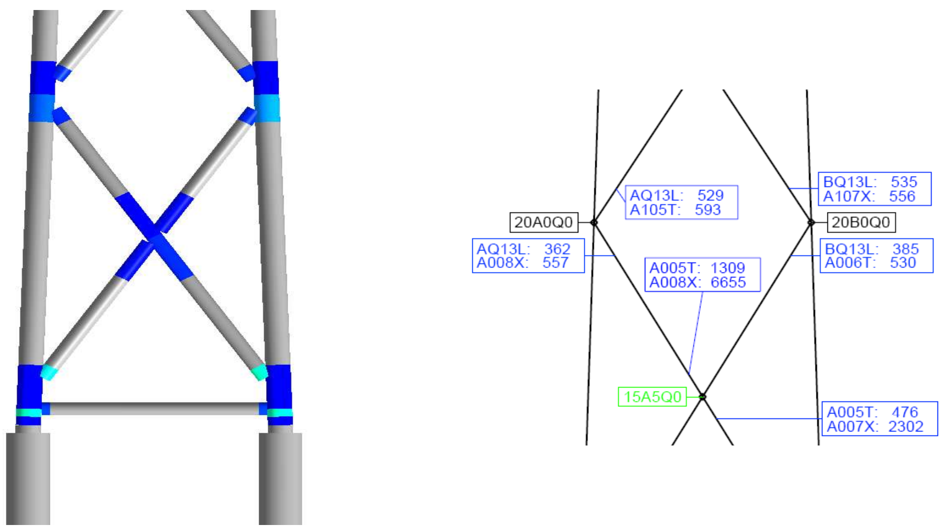


Figure 5–7: Maximum joint fatigue lives at bottom x-brace

Damage equivalent loads

The damage equivalent moment (DEM) provided in Table 5–12 for the preliminary design phase and from the final design phase are shown in the below table. The damage equivalent moment provided for preliminary design phase is, however, valid for a monopile structure and therefore not comparable with the damage equivalent moment for the jacket structure in the final design phase. However, it can be concluded that the monopile DEM (wind only) is very close to jacket DEM (wind & wave).

Table 5–12: Damage equivalent moments

	Final design-jacket (combined wind & wave loads)	Preliminary design (From **Error! Reference source not found.**)
	M_{eq} [kNm], (m=5, N_{eq}=10^7)	M_{eq} [kNm], (m=5, N=10^7)
Interface	23471	24291

6. Conclusions and outlook

In general jacket foundations are found to be less prone to wave loads and introduce higher stiffness and lower soil dependency. Therefore such foundations are well suited to deeper water sites with soft soil condition such as the site under consideration within this report. The jacket structure is optimized with respect to the natural frequency, extreme event and fatigue conditions i.e. the natural frequency of the overall structure is within the allowed range and all member and joint utilizations as well as the fatigue lives are within the allowable limits.

Naturally, not all jacket members and joints can be designed optimally, i.e. fully utilized in terms of fatigue lives and limit states for the extreme events. This results in member diameters and thicknesses that are in some cases fully utilized and in other cases conservative.

Jacket type foundations are relatively stiff and have relatively low masses compared to monopiles. With increased hub height the eigenfrequency can efficiently be reduced. The interface level and hub height are set at 20.15 m and 90.55 m w.r.t. MSL The transition piece dimensions are estimated and used in this study are 9.6*9.6*4. Due to the large water depth (50 m) at this site, four levels of X-braces are implemented in order to comply with the requirement of the minimum angle between chord and brace.

The jacket structure is modelled with simplified local joint flexibility (LJF) assumptions i.e. all braces are calculated as simple T and Y joints, where the flexibility for each brace is calculated as if no other braces were present at the joint. The braces are automatically cut-off at the brace centreline intersection with the chord wall, so the global stiffness is reduced. A mass comparison has been done for inflexible foundation without LJF and with LJF. The lower weight is observed for the jacket with the LJF assumption.

In the preliminary design phase, provided preliminary design extreme loads are applied at interface along and across the jacket foundation in order to extract the governing loads for jacket foundation design. A parameter study has been performed with variations of the jacket bottom width and remaining dimensions of the jacket foundation are kept constant. A jacket bottom base width of 12.0 m is chosen in order to meet the requirements from the support structure 1st natural frequency and total optimal cost of the structural steel. For the final design phase the procedure regarding applied wind and wave loads is explained in chapter4.2.

It should be noted that, the type of transition piece may have an influence on the modal properties e.g. conical steel transition pieces are significantly softer, but less heavy than the concrete transition piece used in this study. The transition piece is considered a major cost item for the jacket type foundation. Moreover, installation of such heavy concrete transition piece adds additional cost to the foundation. Hence, various transition piece solutions should be discussed and tested for offshore wind turbines with jacket foundations.

It is recommended that further studies should be carried on grouted connection and total cost reduction possibilities. A detailed finite element analysis is necessary to check whether the transition piece will withstand the interface loads as well as to verify that the grouted connection between the jacket and the piles is designed sufficiently for the transfer of axial loads and bending moments.

In general, jacket steel is more expensive than the pile (due to high yield strength of the steel). Hence, it is recommended to minimize the jacket steel mass by transferring mass into the pile so the total foundation cost will be reduced.

From section 5.2.3, it can be concluded that there are no significant dynamics introduced from the hydrodynamic excitations, therefore the monopile DEM (wind only) is similar to the jacket DEM (wind-wave) at interface.

It is worth mentioning that the applied wind loads at interface are without accelerations i.e. neglecting the influence of the foundation inertia loads on the total dynamic response. However, especially in case of large masses connected to the foundation, such as the transition piece in this example, the overall fatigue lives might significantly be influenced by the foundation inertia loads as explained in detail in [8].

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Appendix I Soil profiles

DESIGN PARAMETERS FOR SOIL US50 WITH PILE A0P0P

depth (m)	Soil type	γ' (kN/m³)	I_p (%)	q_u (MPa)	φ (deg)	c_u (kPa)	E (MPa)	ε_{50} (%)	t_c (kPa)	t_t (kPa)	q (MPa)
2.7	SAND	10.00			36.0		1.6		0.0	0.0	0.0
3.0	SAND	10.00			36.0		4.3		1.2	1.2	0.1
5.0	SAND	10.00			33.0		6.1		8.4	8.4	0.5
7.0	CLAY	10.00				60.0	24.0	0.7	28.2	28.2	0.5
10.0	SAND	10.00			37.0		22.8		39.1	39.1	3.4
12.9	SAND	10.00			35.0		20.8		51.4	51.4	3.7
15.0	SAND	10.00			35.0		23.1		63.7	63.7	4.6
17.9	SAND	10.00			37.5		35.6		83.4	83.4	7.5
50.0	SAND	10.00			37.5		49.7		103.0	103.0	10.1

scale changed

Soft soil

γ' : Submerged unit weight
I_p : Plasticity index
q_u : Unconfined compression strength
φ : Characteristic angle of internal friction
c_u : Characteristic undrained shear strength
E : Modulus of elasticity
ε_{50} : Strain which occurs at one-half of the maximum stress in laboratory undrained compression test
t_c : Unit skin friction, compression
t_t : Unit skin friction, tension
q : Unit tip resistance, compression

Design code: API
Partial coefficient on angle of internal friction: 1.00
Partial coefficient on undrained shear strength: 1.00
Partial coefficient on axial bearing capacity: 1.00

Scour: Local scour: 2.7 m
Global scour: 0.0 m
Scour angle: 30.0 deg

Pile tip depth: 47.90 m
Pile tip diameter: 2082.0 mm
Pile tip thickness: 28.0 mm

Ramboll Oil & Gas

Subject: Upwind - Soft Soil US50 | Program: ROSA 4.40
Prepared: | Checked: | Approved: | Date: 2009-11-27

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PEN_OLD_CHSL_610_OD82_P48M_ro_A0P0P.PS

DESIGN PARAMETERS FOR SOIL UH50 WITH PILE A0P0P

depth (m)	Soil type	γ' (kN/m³)	I_p (%)	q_u (MPa)	φ (deg)	c_u (kPa)	E (MPa)	ε_{50} (%)	t_c (kPa)	t_t (kPa)	q (MPa)
2.7	SAND	10.00			38.0		2.2		0.0	0.0	0.0
3.0	SAND	10.00			38.0		5.6		1.3	1.3	0.1
5.0	SAND	10.00			35.0		8.0		9.1	9.1	0.7
7.0	SAND	10.00			38.0		20.4		24.8	24.8	2.3
10.0	SAND	10.00			38.0		26.2		40.6	40.6	3.8
12.9	SAND	10.00			42.0		54.5		67.1	67.1	9.5
15.0	SAND	10.00			42.0		60.8		83.1	83.1	11.4
17.9	SAND	10.00			42.5		70.9		100.4	100.4	12.0
50.0	SAND	10.00			42.5		99.1		112.3	112.3	12.0

scale changed

Hard soil

γ' : Submerged unit weight
I_p : Plasticity index
q_u : Unconfined compression strength
φ : Characteristic angle of internal friction
c_u : Characteristic undrained shear strength
E : Modulus of elasticity
ε_{50} : Strain which occurs at one-half of the maximum stress in laboratory undrained compression test
t_c : Unit skin friction, compression
t_t : Unit skin friction, tension
q : Unit tip resistance, compression

Design code: API
Partial coefficient on angle of internal friction: 1.00
Partial coefficient on undrained shear strength: 1.00
Partial coefficient on axial bearing capacity: 1.00

Scour: Local scour: 2.7 m
Global scour: 0.0 m
Scour angle: 30.0 deg

Pile tip depth: 47.90 m
Pile tip diameter: 2082.0 mm
Pile tip thickness: 28.0 mm

Ramboll Oil & Gas

Subject: Upwind - Hard Soil UH50 | Program: ROSA 4.40
Prepared: | Checked: | Approved: | Date: 2009-11-24

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hardsoil_check_CHSL_610_OD82_P48M_ro_A0P0P.PS

Appendix II Natural frequency analysis

Rigid Foundation

E I G E N V A L U E   S O L U T I O N

Number of iterations used 10

Number	Cyclic frequency (rad/s)	Frequency (1/s)	Period (s)	Modal mass (kg-m2)	Modal participation factors X-direction (kg-m2)	Modal participation factors Y-direction (kg-m2)	Modal participation factors Z-direction (kg-m2)	Relat. effective masses X-dir.	Relat. effective masses Y-dir.	Relat. effective masses Z-dir.
1	1.9398	0.3087	3.2391	4.5780E+05	-5.1082E+02	5.6636E+05	-1.4204E+00	0.0000	0.3164	0.0000
2	1.9479	0.3100	3.2256	4.5780E+05	-5.6946E+05	-3.9064E+02	-1.8053E+03	0.3198	0.0000	0.0000
3	6.8400	1.0886	0.9186	4.5780E+05	-3.3309E+03	6.2761E+05	1.6865E+01	0.0000	0.3885	0.0000
4	6.9365	1.1040	0.9058	4.5780E+05	6.2994E+05	2.9340E+03	-3.4389E+03	0.3914	0.0000	0.0000
5	8.1171	1.2919	0.7741	4.5780E+05	-2.3410E+00	-1.3910E+04	4.4430E-02	0.0000	0.0002	0.0000
6	14.9194	2.3745	0.4211	4.5780E+05	1.8802E+01	1.7766E+05	4.9202E-01	0.0000	0.0311	0.0000
7	16.4773	2.6224	0.3813	4.5780E+05	1.6391E+05	1.8336E+01	1.3377E+04	0.0265	0.0000	0.0002
8	20.0259	3.1872	0.3138	4.5780E+05	-9.1477E-02	-1.5470E+02	3.1431E-03	0.0000	0.0000	0.0000
9	33.1479	5.2757	0.1895	4.5780E+05	-1.6728E+04	-6.3441E+02	8.3390E+05	0.0003	0.0000	0.6858
10	34.3948	5.4741	0.1827	4.5780E+05	-2.4998E+04	-2.7614E+05	-6.0086E+03	0.0006	0.0752	0.0000

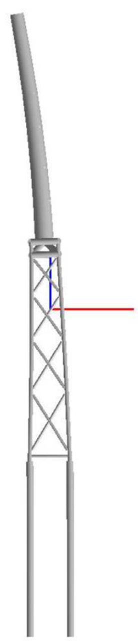
Inflexible Foundation

E I G E N V A L U E   S O L U T I O N

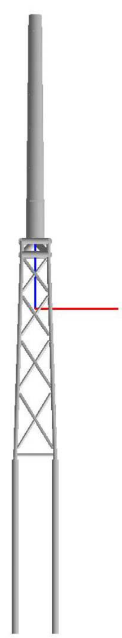
Number of iterations used 7

Number	Cyclic frequency (rad/s)	Frequency (1/s)	Period (s)	Modal mass (kg-m2)	Modal participation factors X-direction (kg-m2)	Modal participation factors Y-direction (kg-m2)	Modal participation factors Z-direction (kg-m2)	Relat. effective masses X-dir.	Relat. effective masses Y-dir.	Relat. effective masses Z-dir.
1	1.8247	0.2904	3.4435	4.7434E+07	-4.7940E+03	6.6020E+06	-3.6071E+02	0.0000	0.3237	0.0000
2	1.8309	0.2914	3.4318	4.7434E+07	-6.6357E+06	-3.5526E+03	-1.6253E+04	0.3270	0.0000	0.0000
3	5.0668	0.8064	1.2401	4.7434E+07	4.9555E+04	-7.9614E+06	-6.3257E+02	0.0000	0.4707	0.0000
4	5.1091	0.8131	1.2298	4.7434E+07	7.9680E+06	4.9621E+04	-2.6182E+04	0.4715	0.0000	0.0000
5	6.5243	1.0384	0.9630	4.7434E+07	1.6364E+04	-1.2933E+05	1.1577E+01	0.0000	0.0001	0.0000
6	12.1680	1.9366	0.5164	4.7434E+07	-5.5383E+04	-4.3498E+06	4.0397E+03	0.0000	0.1405	0.0000
7	12.5755	2.0015	0.4996	4.7434E+07	4.5564E+06	-4.8583E+04	5.6232E+04	0.1542	0.0000	0.0000
8	12.8190	2.0402	0.4901	4.7434E+07	-5.2699E+03	1.4057E+05	-1.9273E+02	0.0000	0.0001	0.0000
9	16.0982	2.5621	0.3903	4.7434E+07	9.8306E+02	-2.2328E+06	1.0767E+04	0.0000	0.0370	0.0000
10	17.3334	2.7587	0.3625	4.7434E+07	-1.6282E+06	8.3895E+03	1.5695E+05	0.0197	0.0000	0.0002

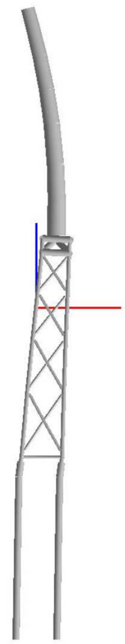
Side view of mode shapes



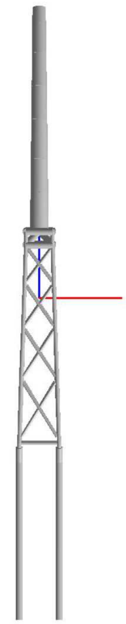
(a) 1st tower fore- aft



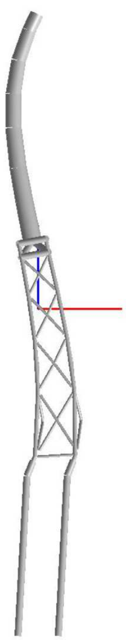
(b) 1st tower side to side



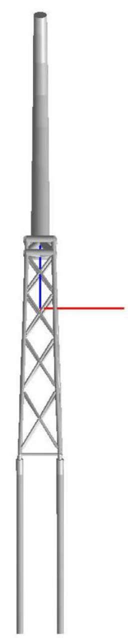
(c) 2nd tower force aft



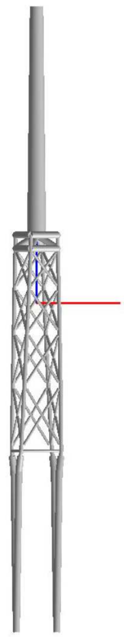
(d) 2nd tower side to side



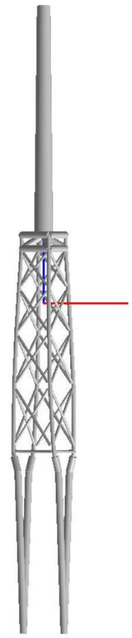
(e) 3rd tower force-aft



(f) 3rd tower side to side

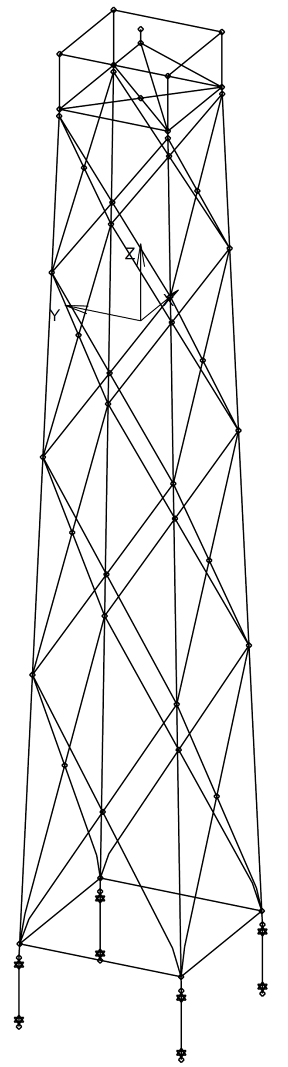


(g) 1st tower torsion



(h) 2nd tower torsion

Appendix III Node and elements names



General view of Jacket

Ramboll Oil & Gas

Direction:	X: 0.816	Y: 0.408	Z:-0.408
Limits:	X(-6.000, 6.000)	Y(-6.000, 6.000)	Z(-50.000, 21.150)
Plot 1:	Date: 2009-11-27	Time: 06:51:15	Job QQGVRA (STPLOT 4.4)

COMPANY: E.U Research Project
PROJECT: UpWind Jacket design-Deep water loacation
SUBJECT : GEOMETRY PLOTS

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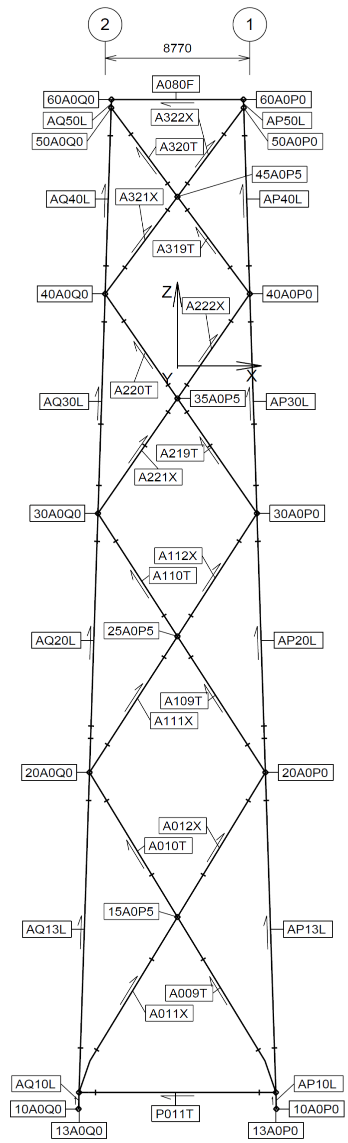
NFA_FATBEM_BD12_OD82_P48M_pl03_A___04.PS



Node and Element Names
Jacket Row A

Ramboll Oil & Gas

Direction:	X: 0.000	Y: 0.999	Z: 0.033
Limits:	X(-6.000, 6.000)	Y(4.000, 6.000)	Z(-45.000, 16.150)
Plot 2:	Date: 2009-11-27	Time: 06:51:15	Job QQGVRA (STPLOT 4.4)

COMPANY: E.U Research Project
PROJECT: UpWind Jacket design-Deep water loacation
SUBJECT : GEOMETRY PLOTS

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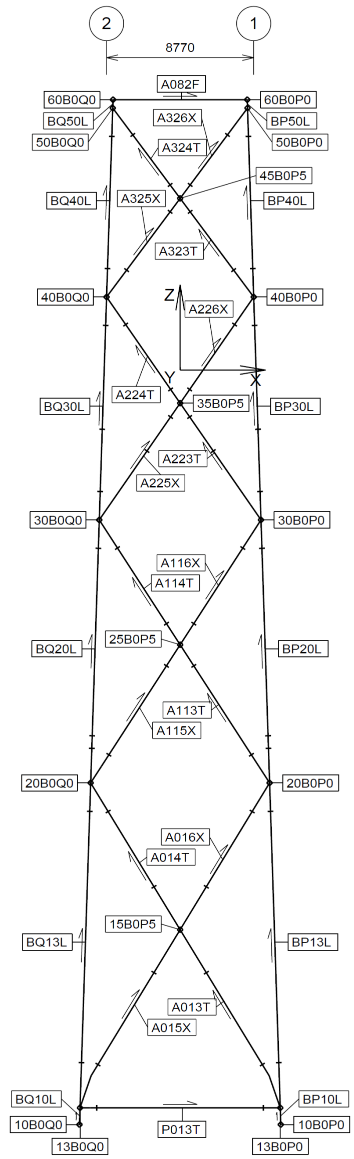
NFA_FATBEM_BD12_OD82_P48M_pl03_A___04.PS



Node and Element Names

Jacket Row B

Ramboll Oil & Gas

Direction:	X: 0.000	Y: 0.999	Z:-0.033
Limits:	X(-6.000, 6.000)	Y(-6.000, -4.000)	Z(-45.000, 16.150)
Plot 3:	Date: 2009-11-27	Time: 06:51:15	Job QQGVRA (STPLOT 4.4)

COMPANY: E.U Research Project

PROJECT: UpWind Jacket design-Deep water loacation

SUBJECT : GEOMETRY PLOTS

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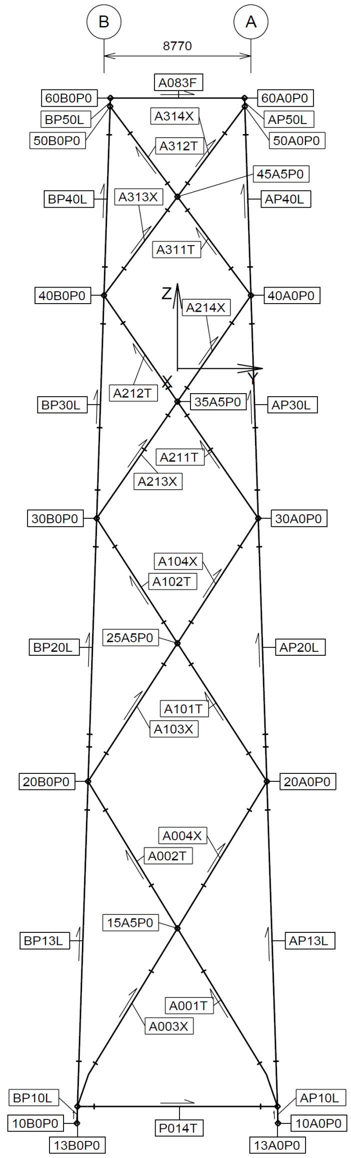
NFA_FATBEM_BD12_OD82_P48M_pl03_A___04.PS



Node and Element Names

Jacket Row 1

Ramboll Oil & Gas

Direction:	X:-0.999	Y: 0.000	Z:-0.033
Limits:	X(4.000, 6.000)	Y(-6.000, 6.000)	Z(-45.000, 16.150)
Plot 4:	Date: 2009-11-27	Time: 06:51:15	Job QQGVRA (STPLOT 4.4)

COMPANY: E.U Research Project

PROJECT: UpWind Jacket design-Deep water loacation

SUBJECT : GEOMETRY PLOTS

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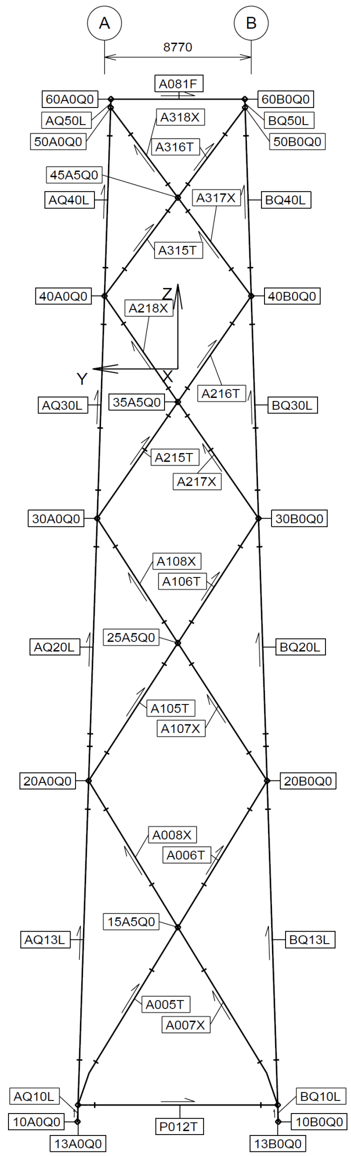
NFA_FATBEM_BD12_OD82_P48M_pl03_A___04.PS



Node and Element Names

Jacket Row 2

Ramboll Oil & Gas

Direction:	X: 0.999	Y: 0.000	Z:-0.033
Limits:	X(-6.000, -4.000)	Y(-6.000, 6.000)	Z(-45.000, 16.150)
Plot 5:	Date: 2009-11-27	Time: 06:51:16	Job QQGVRA (STPLOT 4.4)

COMPANY: E.U Research Project

PROJECT: UpWind Jacket design-Deep water loacation

SUBJECT : GEOMETRY PLOTS

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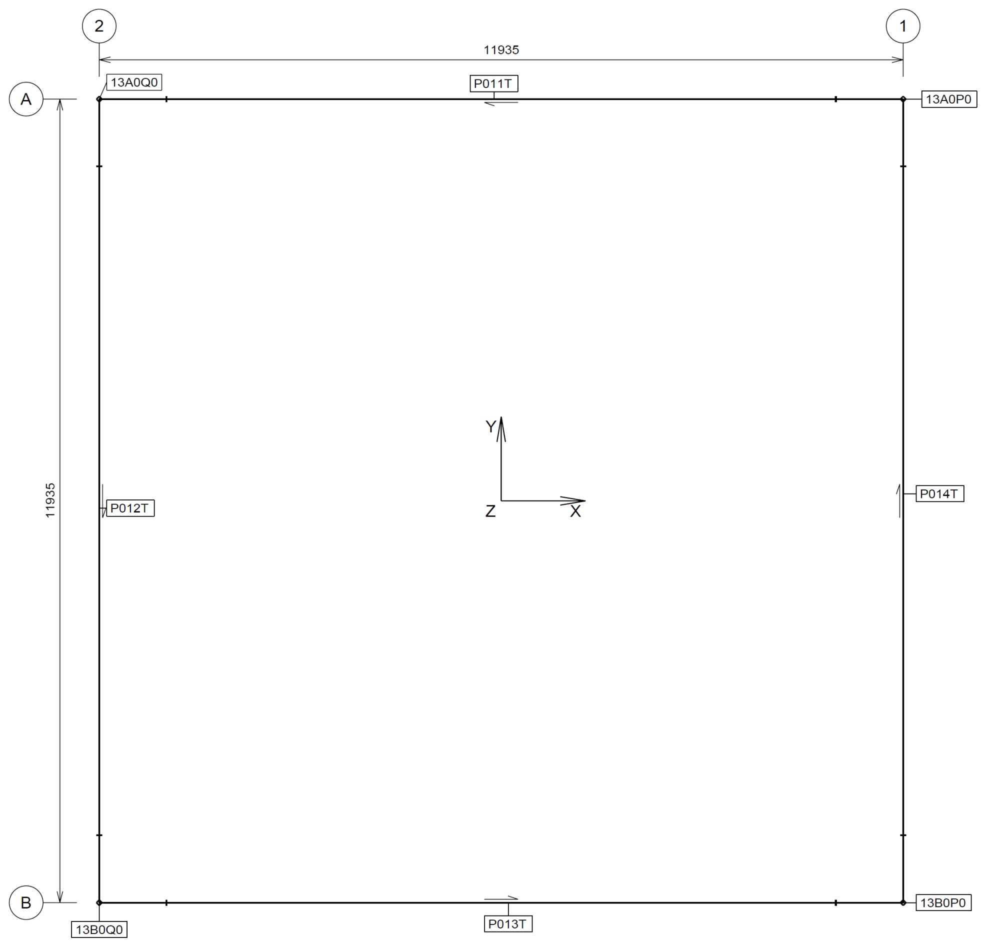
NFA_FATBEM_BD12_OD82_P48M_pl03_A___04.PS



Node and Element Names

Mud Bracing

Ramboll Oil & Gas

Direction:	X: 0.000	Y: 0.000	Z:-1.000
Limits:	X(-5.967, 5.967)	Y(-5.967, 5.967)	Z(-44.001, -44.001)
Plot 6:	Date: 2009-11-27	Time: 06:51:16	Job QQGVRA (STPLOT 4.4)

COMPANY: E.U Research Project

PROJECT: UpWind Jacket design-Deep water loacation

SUBJECT : GEOMETRY PLOTS

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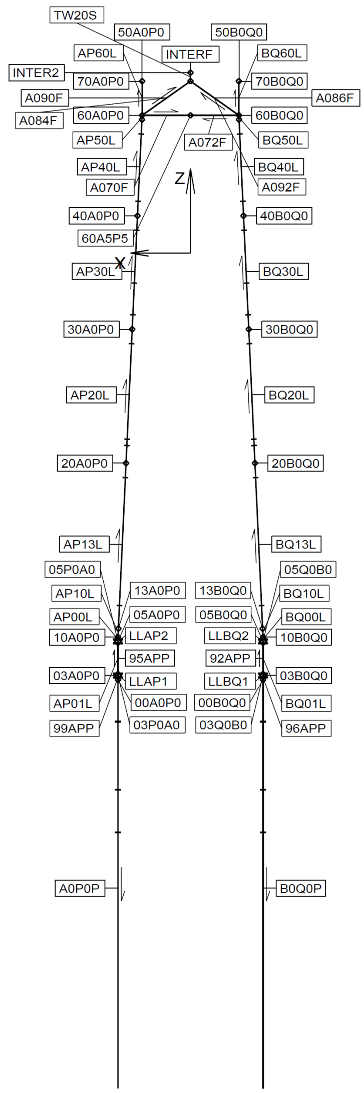
NFA_FATBEM_BD12_OD82_P48M_pl03_A___04.PS



Node and Element Names

Piles on Diagonal Crossing of Module B-1

Ramboll Oil & Gas

Direction:	X: 0.707	Y:-0.707	Z: 0.000
Limits:	X(-6.000, 6.000)	Y(-6.000, 6.000)	Z(-97.900, 21.150)
Plot 7:	Date: 2009-11-27	Time: 06:51:16	Job QQGVRA (STPLOT 4.4)

COMPANY: E.U Research Project
PROJECT: UpWind Jacket design-Deep water loacation
SUBJECT : GEOMETRY PLOTS

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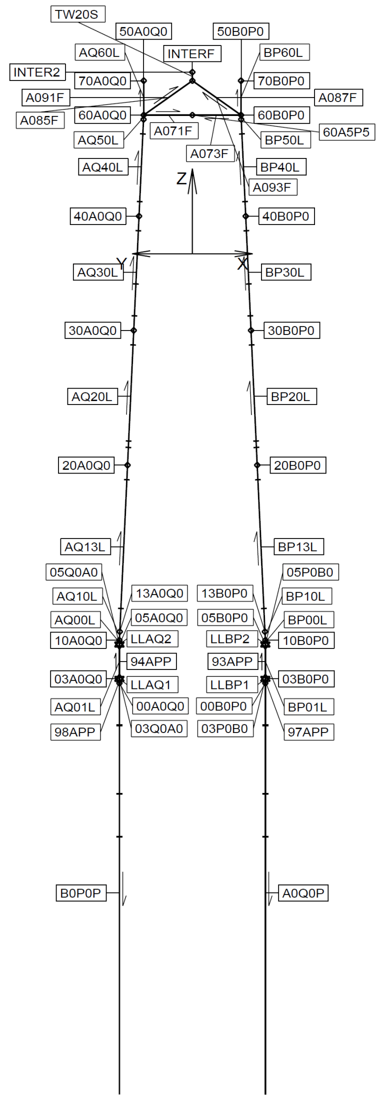
NFA_FATBEM_BD12_OD82_P48M_pl03_A___04.PS



Node and Element Names

Piles on Diagonal Crossing of Module A-2

Ramboll Oil & Gas

Direction:	X: 0.707	Y: 0.707	Z: 0.000
Limits:	X(-6.000, 6.000)	Y(-6.000, 6.000)	Z(-97.900, 21.150)
Plot 8:	Date: 2009-11-27	Time: 06:51:16	Job QQGVRA (STPLOT 4.4)

COMPANY: E.U Research Project

PROJECT: UpWind Jacket design-Deep water loacation

SUBJECT : GEOMETRY PLOTS

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NFA_FATBEM_BD12_OD82_P48M_pl03_A___04.PS

Appendix IV Structural drawings for the jacket

Appendix V Utilization ratios for members and joints

Appendix VI Fatigue lives for members and joints

Appendix VII Intersection curves at tubular joints

Appendix VIII Elements utilization ratios for DLC load cases

Appendix IX DLC6.1 Load combinations

Appendix X Load case description for preliminary design phase

DLC 1.2 – FATIGUE	
Wind conditions	Normal turbulence model (NTM), $V_{in} < V_{hub} < V_{out}$ (V_{hub} in 2m/s bins)
Sea conditions	No waves, no currents, MSL
Partial safety factor	1.0

Description of simulations:

Filename	Mean wind speed [m/s]	Longit. turbulence intensity [%]	Sig. wave height [m]	Peak spectral period [s]	Time [hrs/year]	Wind-wave-misalignment [deg]
1.2a_1-3	4.0	0.137	0.0	0.0	865.8	0°
1.2b_1-3	6.0	0.118	0.0	0.0	1151.5	0°
1.2c_1-3	8.0	0.109	0.0	0.0	1062.6	0°
1.2d_1-3	10.0	0.105	0.0	0.0	1062.3	0°
1.2e_1-3	12.0	0.103	0.0	0.0	1181.1	0°
1.2f_1-3	14.0	0.101	0.0	0.0	946.3	0°
1.2g_1-3	16.0	0.101	0.0	0.0	762.0	0°
1.2h_1-3	18.0	0.101	0.0	0.0	439.0	0°
1.2i_1-3	20.0	0.101	0.0	0.0	352.5	0°
1.2j_1-3	22.0	0.101	0.0	0.0	213.2	0°
1.2k_1-3	24.0	0.101	0.0	0.0	156.8	0°

Comments

- 3D, 3-component Kaimal turbulent wind field (10 min sample)
- 3 bin-combinations for each wind speed bin
- wind gradient exponent (exponential model), α = 0.14
- NTM according to section 6.3.1.3 of [3] or site specific

DLC 6.4 – FATIGUE	
Operating conditions	Parked (Idling)
Wind conditions	Normal turbulence model (NTM), $V_{hub} < 0.7\ V_{ref}$ (V_{hub} in 2m/s bins)
Sea conditions	No waves, no currents, MSL
Partial safety factor	1.0

Description of simulations:

Filename	Mean wind speed [m/s]	Longit. turbulence intensity [%]	Sig. wave height [m]	Peak spectral period [s]	Time [hrs/year]	Wind-wave-misalignment [deg]
6.4a_1-3	2.0	20.0	0.0	0.0	441.9	0°
6.4b_1-3	26.0	10.2	0.0	0.0	125.4	0°

Comments

- 3D, 3-component Kaimal turbulent wind field (10 min sample)
- 3 bin-combinations for each wind speed bin
- wind gradient exponent (exponential model), α = 0.14
- NTM according to section 6.3.1.3 of [3] or site specific

DLC 1.3 – ULTIMATE	
Operating conditions	Power production
Wind conditions	Extreme turbulence model (ETM) , $V_{in} < V_{hub} < V_{out}$
Sea conditions	No waves, no currents, MSL
Partial safety factor	Normal (1.35)

Description of simulations:

Filename	Mean wind speed [m/s]	Longit. turbulence intensity [%]	Sig. wave height [m]	Peak spectral period [s]	Yaw error [deg]
1.3aa_1-3	V_{rated} - 2 (10.0)	29.6	0.0	0.0	- 8°
1.3ab_1-3					0°
1.3ac_1-3					+ 8°
1.3ba_1-3	V_{rated} (12.0)	26.0	0.0	0.0	- 8°
1.3bb_1-3					0°
1.3bc_1-3					+ 8°
1.3ca_1-3	V_{rated} + 2 (14.0)	23.5	0.0	0.0	- 8°
1.3cb_1-3					0°
1.3cc_1-3					+ 8°
1.3da_1-3	V_{out} - 4 (20.0)	19.4	0.0	0.0	- 8°
1.3db_1-3					0°
1.3dc_1-3					+ 8°
1.3ea_1-3	V_{out} (24.0)	17.9	0.0	0.0	- 8°
1.3eb_1-3					0°
1.3ec_1-3					+ 8°
Comments	• 3D, 3-component Kaimal turbulent wind field (10 min sample) • 3 bin-combinations for each wind speed bin • wind gradient exponent (exponential model), α = 0.14 • ETM according to section 6.3.2.3 of [3] or site specific • extreme loads for each load case group (e.g. 1.3aa) are calculated as the mean of the maxima from each of the three seeds				

DLC 1.4 – ULTIMATE	
Operating conditions	Power production
Wind conditions	Extreme coherent gust with change of direction (ECD)
Sea conditions	No waves, no currents, MSL
Partial safety factor	Normal (1.35)

Description of simulations:

Filename	Mean wind speed [m/s]	Gust speed [m/s]	Direction change [deg]	Wave height [m]	Wave period [s]	Yaw error [deg]
1.4aa	V_{rated} - 2 (10.0)	15.0	72°	0.0	0.0	- 8°
1.4ab						0°
1.4ac						+ 8°
1.4ba	V_{rated} (12.0)	15.0	60°	0.0	0.0	- 8°
1.4bb						0°
1.4bc						+ 8°
1.4ca	V_{rated} + 2 (14.0)	15.0	51°	0.0	0.0	- 8°
1.4cb						0°
1.4cc						+ 8°
Comments	• steady wind speed and direction transient (rise time = 10s) • one minute simulation • transient occurs 15s into simulation • wind gradient exponent (exponential model), α = 0.14 • ECD according to section 6.3.2.5 of [1] or site specific					

DLC 6.2a – ULTIMATE	
Operating conditions	Idling with loss of electrical network (up to 6 hrs before storm occurs)
Wind conditions	Extreme wind model (EWM) ,(turbulent), ($V_{hub} = V_{50}$)
Sea conditions	No waves, no currents, MSL
Partial safety factor	Abnormal (1.1)

Description of simulations:

Filename	**Mean wind speed [m/s]**	**Longit. turbulence intensity [%]**	**Sig. wave height [m]**	**Peak spectral period [s]**	**Yaw error [deg]**
6.2a _1-3	V_{ref} (48.62)	11.0	0.0	0.0	0°
6.2b _1-3					30°
6.2c _1-3					60°
6.2d _1-3					90°
6.2e _1-3					120°
6.2f _1-3					150°
6.2g _1-3					180°
Comments	• 3D, 3-component Kaimal turbulent wind field (10 min sample) • 3 bin-combinations for each wind speed bin • wind gradient exponent (exponential model), α = 0.14 • turbulence intensity for EWM set to 11% as specified in section 6.3.2.1 of [1] • extreme loads for each load case group (e.g. 6.2a_x_y) are calculated as the mean of the maxima from each of the three seeds				

Appendix XI Load case description for final design phase

Fatigue load cases:

Design load case (DLC):	1.2
Operating condition:	Power production
Wind conditions:	Normal turbulence model, $V_{in} < V_{hub} < V_{out}$
Sea conditions:	Normal sea state, no currents, MSL + 10% of tidal range
Type of analysis:	Fatigue
Partial safety factors:	Partial safety factor for fatigue

Description of simulations:

	Wind conditions		Wave conditions			
	Mean wind speed (m/s)	Longitudinal turbulence intensity (%)	Significant wave height	Peak spectral period	Yaw error	Hours/year
1.2axy1-6	4	20.4	1.10	5.88	8 deg	874.7
1.2bxy1-6	6	17.5	1.18	5.76	8 deg	992.8
1.2cxy1-6	8	16.0	1.31	5.67	8 deg	1181.8
1.2dxy1-6	10	15.2	1.48	5.74	8 deg	1076.3
1.2exy1-6	12	14.6	1.70	5.88	8 deg	1137.2
1.2fxy1-6	14	14.2	1.91	6.07	8 deg	875.6
1.2gxy1-6	16	13.9	2.19	6.37	8 deg	764.7
1.2hxy1-6	18	13.6	2.47	6.71	8 deg	501.3
1.2ixy1-6	20	13.4	2.76	6.99	8 deg	336.0
1.2jxy1-6	22	13.3	3.09	7.40	8 deg	289.4
1.2kxy1-6	24	13.1	3.42	7.80	8 deg	130.4

Comments:

Three dimensional three component Kaimal turbulent wind field (10 min sample).

First 20s of output discarded to allow initial transients to decay

Six turbulent wind seeds per wind speed bin (indexed 1-6)

Simulations run with 12 wind directions in 30deg sectors around the structure from 0 - 330deg (indexed x=a-l)

Simulations run with wind/wave misalignment from -30 to 120deg relative to wind direction in 30deg intervals (indexed y=a-f)

Wind gradient exponent (exponential model), $\alpha = 0.14$

Normal sea state with irregular waves defined using Jonswap spectrum with $\gamma = 1$

Tidal range is 2.22m giving water level of 50 + 0.222 = 50.222m

Supervisory control is disabled for these simulations

Design load case (DLC):	6.4					
Operating condition:	Parked (stand still or idling)					
Wind conditions:	Normal turbulence model, $V_{hub} < 0.7\ V_{ref}$					
Sea conditions:	Normal sea state, no currents, MSL + 10% of tidal range					
Type of analysis:	Fatigue					
Partial safety factors:	Partial safety factor for fatigue					
Description of simulations:						
	Wind conditions		Wave conditions			
	Mean wind speed (m/s)	Longitudinal turbulence intensity (%)	Significant wave height	Wave period	Yaw error	Hours/year
6.4axy1-6	2	29.2	1.07	6.03	8 deg	434.3
6.4bxy1-6	30	11.8	4.46	8.86	8 deg	149.0
Comments:	Three dimensional three component Kaimal turbulent wind field (10 min sample) First 20s of output discarded to allow initial transients to decay Six turbulent wind seeds per wind speed bin (indexed 1-6) Simulations run with 12 wind directions in 30deg sectors around the structure from 0 - 330deg (indexed x=a-l) Simulations run with wind/wave misalignment from -30 to 120deg relative to wind direction in 30deg intervals (indexed y=a-f) Wind gradient exponent (exponential model), α = 0.14 Normal sea state with irregular waves defined using Jonswap spectrum with γ = 1 Tidal range is 2.22m giving water level of 50 + 0.222 = 50.222m All blades at idling pitch angle of 90 deg Supervisory control is disabled for these simulations					

Extreme load cases:

Design load case (DLC):	1.6a
Operating condition:	Power production
Wind conditions:	Normal turbulence model, 0.8*V_r, V_r, 1.2*V_r, V_{out},
Sea conditions:	Severe sea state, normal current model, NWLR
Type of analysis:	Ultimate
Partial safety factors:	Normal

Description of simulations:

	Wind conditions		Wave conditions		
	Mean wind speed (m/s)	Longitudinal turbulence intensity (%)	Significant wave height (m)	Peak spectral period (s)	Yaw error
1.6a1-6	10	15.20	9.40	13.70	8 deg
1.6b1-6	12	14.60	9.40	13.70	8 deg
1.6c1-6	14	14.20	9.40	13.70	8 deg
1.6d1-6	24	13.10	9.40	13.70	8 deg

Comments:

Three dimensional three component Kaimal turbulent wind field (10 min sample)

First 20s of output discarded to allow initial transients to decay

Six turbulent wind seeds per wind speed bin (indexed 1-6)

Simulations run with support structure at 0deg and 45deg orientation from North

Wind gradient exponent (exponential model), $\alpha = 0.14$

Normal sea state with irregular waves defined using Jonswap spectrum with $\gamma = 3.3$

50 year significant wave height H_{s50} used as a conservative value for $H_{s,SSS}(V)$.

Normal current of 0.6 m/s applied

1-year extreme water level (HAT) of 51.16m

Constrained extreme non-linear wave included in irregular wave history:

- Constrained wave height = Hmax50 = 17.48m
- Constrained wave period = T50 = 10.87s
- Time of constrained wave crest: 100s

The characteristic loads for each load case group are calculated as the mean of the maxima from each of the six seeds.

Design load case (DLC):	2.2
Operating condition:	Power production plus occurrence of fault
Wind conditions:	Normal turbulence model, $V_{in} < V_{hub} < V_{out}$
Sea conditions:	Normal sea state, normal current model, MSL
Type of analysis:	Ultimate
Partial safety factors:	Abnormal

Description of simulations:

	Mean wind speed (m/s)	Longitudinal turbulence intensity (%)	Significant wave height (m)	Peak spectral period (s)	Yaw error	Fault
2.2a1-6	10	15.20	1.48	5.74	8 deg	Collective pitch runaway: all blades pitch to fine at 5deg/s
2.2b1-6	12	14.60	1.70	5.88	8 deg	
2.2c1-6	14	14.20	1.91	6.07	8 deg	
2.2d1-6	20	13.40	2.76	6.99	8 deg	
2.2e1-6	24	13.10	3.42	7.80	8 deg	

Comments:

Three dimensional three component Kaimal turbulent wind field (1 min sample).

First 20s of output discarded to allow initial transients to decay

Twelve turbulent wind seeds per wind speed bin (indexed 1-12)

Simulations run with support structure at 0deg and 45deg orientation from North

Fault occurs 10s into simulation

Wind gradient exponent (exponential model), $\alpha = 0.14$

Normal sea state with irregular waves defined using Jonswap spectrum with $\gamma = 3.3$

Normal current of 0.6 m/s applied

The characteristic loads for each load case group are calculated as the mean of the upper half of the maxima from each of the twelve seeds.

Design load case (DLC):	2.3					
Operating condition:	Power production plus loss of electrical grid connection					
Wind conditions:	Extreme operating gust (EOG)					
Sea conditions:	Normal wave height, normal current model, MSL					
Type of analysis:	Ultimate					
Partial safety factors:	Abnormal					
Description of Simulations:						
	Wind conditions		Wave conditions			
	V_{hub} (m/s)	EOG gust (m/s)	Wave height (m)	Wave period (s)	Yaw error	Grid loss phasing
2.3axy	10	3.86	1.10	4.68	8 deg	$t_{start\ gust}$ + 0s $t_{start\ gust}$ + 2.25s $t_{start\ gust}$ + 4s $t_{start\ gust}$ + 5.25s
2.3bxy	12	4.45	1.58	5.62	8 deg	
2.3cxy	14	5.05	2.15	6.55	8 deg	
2.3dxy	16	5.65	2.81	7.49	8 deg	
2.3exy	18	6.21	3.55	8.42	8 deg	
2.3fxy	20	6.80	4.39	9.36	8 deg	
2.3gxy	22	7.43	5.31	10.30	8 deg	
2.3hxy	24	7.98	6.32	11.23	8 deg	
Comments:	Steady wind with transient gust (gust period = 10.5s) One minute simulations First 20s of output discarded to allow initial transients to decay Simulations run with support structure at 0deg and 45deg orientation from North Gust occurs 10s into simulation Wind gradient exponent (exponential model), α = 0.14 Normal wave height modelled with regular waves using stream function model. Normal current of 0.6 m/s applied Grid loss occurs at minimum wind speed, maximum gust acceleration and maximum wind speed (indexed x=1-3) Starting azimuth angle varied from 0-90deg in 30deg intervals (indexed y=1-4)					

Design load case (DLC):	6.1a
Operating condition:	Idling
Wind conditions:	Extreme wind model (turbulent) ($V_{hub} = V_{50}$)
Sea conditions:	Extreme sea state ($H_s = H_{s50}$), extreme current model (50yr return), EWLR
Type of analysis:	Ultimate
Partial safety factors:	Normal

Description of simulations:

<table>
<tr><th></th><th colspan="2">Wind conditions</th><th colspan="2">Wave conditions</th><th></th><th></th></tr>
<tr><th></th><th>Mean wind speed (m/s)</th><th>Longitudinal turbulence intensity (%)</th><th>Significant wave height (m)</th><th>Peak spectral period (s)</th><th>Yaw error</th><th>Wind/wave misalignment</th></tr>
<tr><td>6.1a1-6</td><td rowspan="6">42.73</td><td rowspan="6">11.00</td><td rowspan="6">9.40</td><td rowspan="6">13.70</td><td rowspan="6">8 deg</td><td>0 deg</td></tr>
<tr><td>6.1b1-6</td><td>30 deg</td></tr>
<tr><td>6.1c1-6</td><td>60 deg</td></tr>
<tr><td>6.1d1-6</td><td>90 deg</td></tr>
<tr><td>6.1e1-6</td><td>120 deg</td></tr>
<tr><td>6.1f1-6</td><td>150 deg</td></tr>
</table>

Comments:

Three dimensional three component Kaimal turbulent wind field (10 min sample).

First 20s of output discarded to allow initial transients to decay

Six turbulent wind seeds per wind speed bin (indexed 1-6)

Simulations run with support structure at 0deg and 45deg orientation from North

Wind gradient exponent (exponential model), $\alpha = 0.11$

Extreme sea state with irregular waves defined using Jonswap spectrum with $\gamma = 3.3$

Extreme current with 50-year return period of 1.2 m/s applied

50-year extreme water level (HSWL) of 53.29m

Constrained extreme non-linear wave included in irregular wave history:

- Constrained wave height = H50 = 17.48m
- Constrained wave period = T50 = 10.87s
- Time of constrained wave crest: 100s

The characteristic loads for each load case group are calculated as the mean of the maxima from each of the six seeds.

Design load case (DLC):	6.2a
Operating condition:	Idling with grid loss
Wind conditions:	Extreme wind model (turbulent) ($V_{hub} = V_{50}$)
Sea conditions:	Extreme sea state ($H_s = H_{s50}$), extreme current model (50yr return), EWLR
Type of analysis:	Ultimate
Partial safety factors:	Abnormal

Description of simulations:

	Wind conditions		Wave conditions			
	Mean wind speed (m/s)	Longitudinal turbulence intensity (%)	Significant wave height (m)	Peak spectral period (s)	Yaw error	Wind/wave misalignment
6.2a1-6	42.73	11.00	9.40	13.70	0 deg	deg
6.2b1-6					30 deg	deg
6.2c1-6					60 deg	deg
6.2d1-6					90 deg	deg
6.2e1-6					120 deg	deg
6.2f1-6					150 deg	deg
6.2g1-6					180 deg	deg

Comments:

Three dimensional three component Kaimal turbulent wind field (10 min sample).

First 20s of output discarded to allow initial transients to decay

Six turbulent wind seeds per wind speed bin (indexed 1-6)

Simulations run with support structure at ..deg orientation to wind (worst case from dlc6.1)

Wind/wave misalignment taken as worst case resulting from dlc6.1

Wind gradient exponent (exponential model), $\alpha = 0.11$

Extreme sea state with irregular waves defined using Jonswap spectrum with $\gamma = 3.3$

Extreme current with 50-year return period of 1.2 m/s applied

50-year extreme water level (HSWL) of 53.29m

Constrained extreme non-linear wave included in irregular wave history:

- Constrained wave height = H50 = 17.48m
- Constrained wave period = T50 = 10.87s
- Time of constrained wave crest: 100s

The characteristic loads for each load case group are calculated as the mean of the maxima from each of the six seeds.

Appendix XII Fatigue loads (as DEL) for preliminary design

Lifetime weighted equivalent loads: Support structure at Fx at 14.75m MSL

	f [Hz]						
	0,0158	0,1	0,27	0,29	0,31	1	10
m ╲ N	1.0 e7	6.312 e7	1.704 e8	1.83 e8	1.957 e8	6.312 e8	6.312 e9
3	315,701	170,83	122,681	119,793	117,159	79,2923	36,8042
4	274,624	173,262	135,165	132,771	130,576	97,4325	54,7903
5	264,513	182,987	150,019	147,89	145,931	115,457	72,8483
6	265,153	195,047	165,29	163,333	161,528	132,884	90,533
7	270,276	207,732	180,252	178,421	176,729	149,502	107,594
8	277,42	220,354	194,626	192,895	191,294	165,242	123,914
9	285,463	232,619	208,313	206,665	205,14	180,109	139,452
10	293,833	244,392	221,284	219,709	218,248	194,127	154,201
11	302,219	255,612	233,543	232,031	230,628	207,335	168,176
12	310,443	266,259	245,108	243,652	242,302	219,771	181,4

Lifetime weighted equivalent loads: Support structure at Fy at 14.75m MSL

	f [Hz]						
	0,0158	0,1	0,27	0,29	0,31	1	10
m ╲ N	1.0 e7	6.312 e7	1.704 e8	1.83 e8	1.957 e8	6.312 e8	6.312 e9
3	162,993	88,1979	63,3389	61,848	60,4882	40,9378	19,0017
4	147,801	93,2491	72,7451	71,457	70,2755	52,4378	29,4879
5	146,278	101,194	82,9621	81,7849	80,7012	63,8488	40,2859
6	149,418	109,913	93,1438	92,041	91,0236	74,8826	51,0169
7	154,352	118,634	102,94	101,895	100,928	85,3789	61,446
8	159,916	127,021	112,19	111,193	110,269	95,2522	71,4291
9	165,577	134,926	120,828	119,872	118,987	104,469	80,8862
10	171,081	142,295	128,841	127,923	127,073	113,029	89,782
11	176,311	149,121	136,246	135,364	134,546	120,957	98,1118
12	181,219	155,426	143,08	142,23	141,442	128,29	105,891

Lifetime weighted equivalent loads: Support structure at Fz at 14.75m MSL

	f [Hz]						
	0,0158	0,1	0,27	0,29	0,31	1	10
m ╲ N	1.0 e7	6.312 e7	1.704 e8	1.83 e8	1.957 e8	6.312 e8	6.312 e9
3	263,162	142,401	102,264	99,8571	97,6617	66,0965	30,6793
4	219,056	138,204	107,815	105,906	104,155	77,7178	43,7039
5	204,692	141,603	116,091	114,444	112,928	89,3455	56,3732
6	200,688	147,627	125,104	123,623	122,257	100,577	68,5223
7	201,224	154,659	134,2	132,837	131,577	111,306	80,1051
8	204,015	162,048	143,128	141,855	140,677	121,519	91,1264
9	208,013	169,507	151,795	150,595	149,483	131,243	101,617
10	212,687	176,9	160,174	159,033	157,976	140,517	111,616
11	217,743	184,164	168,263	167,174	166,163	149,381	121,168
12	223,001	191,262	176,069	175,024	174,054	157,869	130,306

Lifetime weighted equivalent loads: Support structure at Mx at 14.75m MSL

	f [Hz]						
	0,0158	0,1	0,27	0,29	0,31	1	10
m \ N	1.0 e7	6.312 e7	1.704 e8	1.83 e8	1.957 e8	6.312 e8	6.312 e9
3	16080,1	8701,18	6248,71	6101,62	5967,48	4038,73	1874,61
4	14129,3	8914,28	6954,17	6831,04	6718,09	5012,87	2818,94
5	13604,1	9411,16	7715,62	7606,13	7505,35	5938,04	3746,65
6	13587,6	9995,08	8470,17	8369,9	8277,38	6809,57	4639,31
7	13793,3	10601,4	9198,97	9105,54	9019,2	7629,66	5490,96
8	14105,1	11203,6	9895,52	9807,52	9726,1	8401,54	6300,27
9	14467,3	11789,2	10557,3	10473,9	10396,5	9127,94	7067,43
10	14850,1	12351,4	11183,6	11103,9	11030,1	9811,06	7793,21
11	15236,5	12886,9	11774,2	11698	11627,3	10452,9	8478,69
12	15616,7	13394	12330	12256,8	12188,9	11055,5	9125,25

Lifetime weighted equivalent loads: Support structure at My at 14.75m MSL

	f [Hz]						
	0,0158	0,1	0,27	0,29	0,31	1	10
m \ N	1.0 e7	6.312 e7	1.704 e8	1.83 e8	1.957 e8	6.312 e8	6.312 e9
3	31687,2	17146,4	12313,6	12023,8	11759,4	7958,65	3694,08
4	26263,4	16569,8	12926,4	12697,5	12487,5	9317,88	5239,83
5	24291,2	16804,4	13776,8	13581,3	13401,4	10602,8	6689,94
6	23592,4	17354,6	14706,9	14532,8	14372,2	11823,6	8055,31
7	23468	18037,3	15651,2	15492,3	15345,4	12981,2	9342,37
8	23627,4	18767,2	16576	16428,6	16292,2	14073,4	10553,6
9	23930,5	19500,6	17463	17324,9	17197	15098,6	11690,3
10	24304	20214,6	18303,2	18172,9	18052,1	16057	12754,5
11	24707,8	20897,5	19093,2	18969,6	18854,9	16950,6	13749,1
12	25119,3	21544,1	19832,7	19715	19605,7	17782,6	14677,9

Lifetime weighted equivalent loads: Support structure at Mz at 14.75m MSL

	f [Hz]						
	0,0158	0,1	0,27	0,29	0,31	1	10
m \ N	1.0 e7	6.312 e7	1.704 e8	1.83 e8	1.957 e8	6.312 e8	6.312 e9
3	6930,16	3750	2693,04	2629,66	2571,84	1740,6	807,913
4	6108,93	3854,18	3006,7	2953,47	2904,63	2167,36	1218,8
5	5997,14	4148,75	3401,3	3353,03	3308,6	2617,68	1651,65
6	6141,02	4517,36	3828,17	3782,84	3741,03	3077,64	2096,77
7	6384,79	4907,29	4258,12	4214,88	4174,91	3531,71	2541,72
8	6662,11	5291,69	4673,84	4632,28	4593,82	3968,21	2975,74
9	6943,07	5657,8	5066,62	5026,55	4989,44	4380,63	3391,76
10	7214,38	6000,47	5433,12	5394,43	5358,58	4766,35	3786,04
11	7470,51	6318,45	5772,92	5735,54	5700,87	5125,09	4157,12
12	7709,6	6612,32	6087,05	6050,91	6017,37	5457,83	4504,92