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Versioning and contribution history

Version	Description	Contributions
1.0	New offshore turbine: GE next generation 15+MW	JBO, PPI, DTU, DNV, GE

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Table of Contents

List of Figures

List of Tables

List of Abbreviations

ALS	Accidental limit state
CoG	Centre of gravity
DLC	Design load case
EC	European Commission
EU	European Union
FLS	Fatigue limit state
FP	Foot print
GBF	Gravity Base Foundation
H_s	Significant wave height of a sea state
HW	High water level (*Hochwasser*)
JBO	Jörss – Blunck – Ordemann GmbH
kn	Knot [sm/h]
LAT	Lowest astronomical tide
LC	Load case
LCoE	Levelized Cost of Energy
MP	Monopile
MSL	Mean sea level
MW	Megawatt
OWF	Offshore windfarm
PPI	Principal Power Portugal Unipessoallda
RNA	Rotor nacelle assembly
SCF	Stress concentration factor
SLS	Serviceability limit state
SWL	Still water level
Tb	Tower bottom (Interface Level)
TI	Turbulence intensity
T_m	Mean zero-crossing period of a wave
T_p	Peak period of a sea state
TP	Transition piece
ULS	Ultimate limit state
WTG	Wind turbine generator

Executive Summary

This document provides an overview on the **conceptual design of bottom fixed and floating support structures** such as loads and design studies regarding optimisation of the designs. This belongs to the development phase of a next generation 15+MW offshore wind turbine within the ReaLCoE project and is carried out for different sites (s. Table 1) as specified in the Design Basis [R1]. The analysed support structure types are:

- Monopile (Site 1 and Site 3, s. Chapter 2)
- Jacket (Site 1 and Site 3, s. Chapter 3)
- Gravity Base Foundation (Site 1, s. Chapter 4)
- Floating foundation (Site 5, s. Chapter 5)

It need to be mentioned that all designs presented here are on the level of a conceptual design. They are intended for comparison between the support structure types as specified above and need to be analysed in more detail for a specific project site.

Chapters and sections not specified otherwise are established by JBO. Main contribution of JBO are in Chapters 2, 3, 4 and 6. Principal Power is participating in the work package with the design of a floating substructure with main contributions to Chapters 5 and 6. Main topic of DTU is the load iteration and optimisation for the design documented in Chapters 2, 5 and 6. The contribution of DNV is related to Chapters 2 and 5. The contribution of GE is related to all chapters.

A **preliminary version 0.1** has been established to present intermediate results of the JBO part of WP1.3 due to project suspension dated 31st May 2019. At that stage initial concept designs for three bottom fixed foundations (Monopile, Jacket, Monobucket) have been established by JBO. These designs are based on the previous turbine configuration (Senvion 12M210) and a previous version of Design Basis.

This **final version 1.0** is established to present the overall results of the WP1.3 as defined in the project proposal. It focuses on a next generation 15+MW turbine and the current Design Basis (D1.1) version 3.0 [R1]. Foundation types as mentioned above have been analysed and optimised. According to the proposal, site loads from a site-specific integrated load analysis are not part of the conceptual design.
Some of the references to the project deliverables are confidential and only available to the project participants. This includes the following references: [R1], [R2]

1. Introduction

1.1. Objectives of Deliverable 1.3

ReaLCoE – “Next Generation 12+MW Rated, Robust, Reliable and Large Offshore Wind Energy Converters for Clean, Low Cost and Competitive Electricity” is a Research & Development (R&D) project funded by the EU within the Horizon 2020 program. Main goal of ReaLCoE is to accelerate the development of a high-performance 12+MW offshore wind energy converter (OWEC) leading to a competitive, subsidy free and clean energy.

The Conceptual Design of bottom fixed and floating structures stated in this document relates to Task 1.3 of WP1 “Development of 12+MW WEC including substructures and manufacturing concepts".

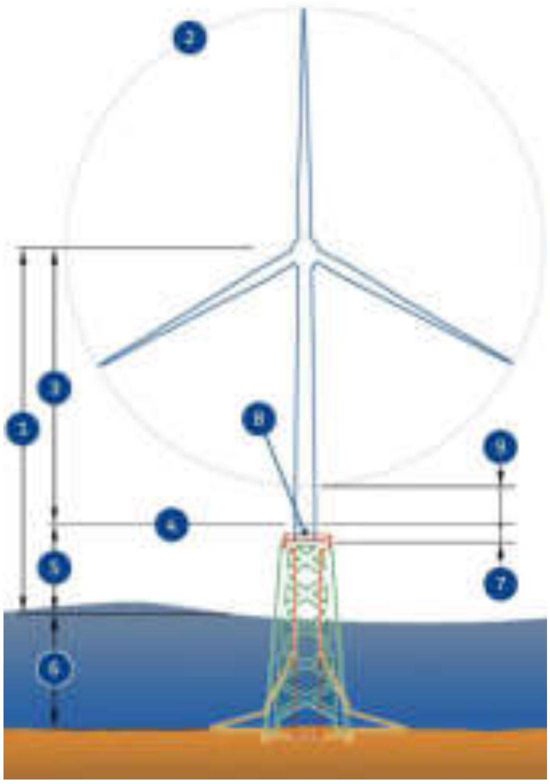
Aim of Task 1.3 is to optimise the design of substructures for large turbines considering effects of soil conditions and damping such as cost reduction though new interfaces of substructure and tower considering limitations from fabrication and installation. This requires analyses of the dynamic behaviour, loads and structural verification in a holistic manner. With the purpose of comparing different types of substructures three types of bottom fixed (monopile, jacket and gravity base foundation) and one floating foundation have been investigated for different water depths.

Wind-only loads are used for this conceptual design stage and combined with site specific wave loads and further met-ocean conditions without integrated load analysis.

All concept designs shown here need to be verified for use within a specific site.

1.2. General project data

GE provides a turbine layout in an early stage of development, which is to be further developed and optimized during the project. The turbine is named GE next generation 15+MW offshore wind turbine. A sketch of the WTG is shown in Figure 1 including optional types of substructures detailed for the part above mudline.



nr.	Description	nr.	Description
1	Hub height	7	Interface height from service platform
2	Rotor diameter	8	Service Platform
3	Tower height	9	Blade tip clearance to interface level
4	Interface level	(green)	Jacket structure
5	Tower height above sea level	(red)	Monopile
6	Water depth	(orange)	GBS

Figure 1: Overview of the WTG

1.3. Environmental conditions

Environmental conditions are defined for 5 different sites, where 3 sites have been selected for the investigations within this report. An overview is provided in Table 1. Details on the environmental conditions can be taken from the Design Basis [R1], *Appendix A*. Due to some leak of data in the Design Basis [R1] additional data are given in [R2].

Table 1: Overview of water depths of the different sites

Location	Site 1	Site 3	Site 5
Water depth	35-40 m	60m	75m

1.4. Methodology of design verification

The Design Basis [R1] acts as main document of valid input data and methodology description for the design phase in the current stage. Especially the hierarchy of norms and standards and further design procedures are provided in [R1], *Section 2.2*, and is used for the design verification within this document unless stated otherwise. Specific assumptions for the individual designs are given in the individual chapters. An overview is provided in Figure 2.

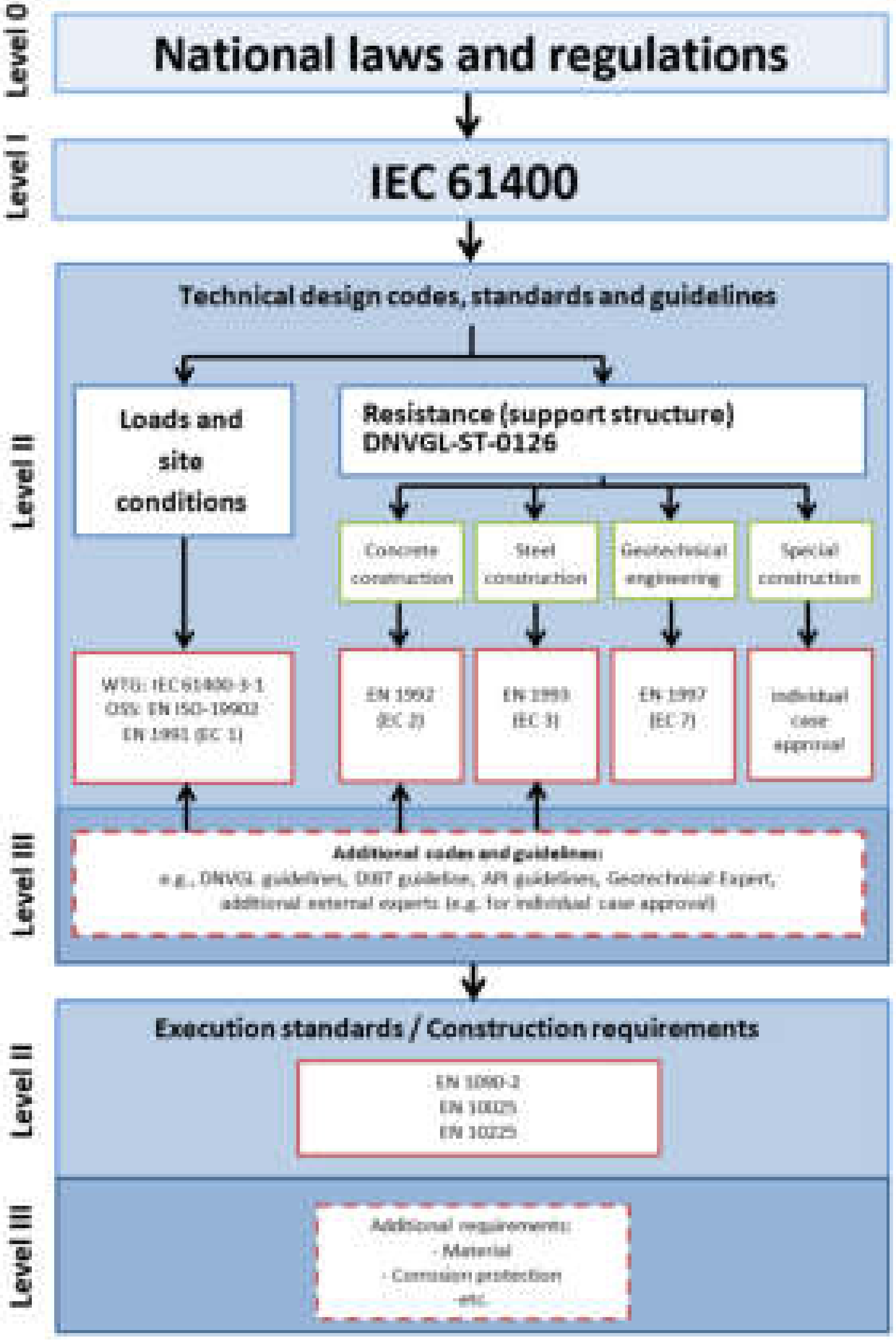


Figure 2: Hierarchy of norms and standards for WEC, substructure and foundation

2. Monopile conceptual design

2.1. General

Monopile designs have been developed for Site 1 and Site 3. Meanwhile Site 1 with 35 m to 40 m water depth is close to the industrial practice today, Site 3 is progressive with 60 m water depth. The investigation indicates the growth rule for the monopile in a highly relevant range of water depth.

The monopiles are object of several optimisation studies. At first the effect of the embedded monopile diameter in several parameters like eigenfrequency, extreme and fatigue design optimisation and the embedded length have been investigated. Regarding the damping effects studies are performed regarding structural and aerodynamic damping and their influence on the fatigue damage. In addition, the global sectioning, and interfaces between monopile sections is evaluated considering a water range from 40 m to 70 m water depth.

A summary of the monopile conceptual designs and studies is provided within this chapter.

2.1.1. Design procedure

The design process for the monopiles is an iterative one. The iteration loop is characterised by the following phases.

Geotechnical assessment
For the i^{th} pile configuration (diameter-thickness distribution), the pile-soil-interaction is simulated with the site-specific soil model derived from the design basis data. It results in a set of nonlinear springs which can be used supporting the entire structure in the following.

Load assessment
A beam element model is set up using the i^{th} pile configuration. It is supported by the soil springs generated before. Wave loads are generated with respect to the pile dimension for the sea states given in the design basis. Wave loads and wind turbine loads are combined to form the extreme load event for ultimate limit state (ULS) assessment and to form damage equivalent fatigue loads for the fatigue limit state (FLS) assessment. These sectional loads are available continuously along the pile and tower axis. A further important output of the model in this phase is the eigenfrequency of the system, which must meet specific turbine requirements.

Structural design
The sectional loads allow the structural verification of the pile can by can. Several design checks are performed. The resulting utilization may lead to adjustments in diameter or wall thickness. The updated pile configuration will be handed over to the geotechnical assessment starting the next loop. Once the utilization of the cans satisfies, the design process is finished.

2.1.1. Design methodology

The following sections describe the basic assumptions and outline the applied design methodologies in the conceptual design.

The layout of the monopile structure is decided based on several parameters:
- Statutory and regulatory requirements,
- Employer's or WTG requirements.
- Site specific conditions, such as wind, wave, current, and soil profiles over the site.
- Turbine characteristics, such as tower length and tower bottom diameter.
- Practical limitations due to fabrication, installation, and others.
- Design Basis [R1] is applied and additionally [R2] for Site 3

The main structural dimensions are defined in [R1] and below as:
- Turbine Power: 15+ MW
- TP-Tower bottom interface: +15.80…+21.20 mMSL (depending on the site)

The design process follows the general procedures defined by the IEC 61400 series [N27] and Eurocode [N8] - [N18].

2.1.2. Monopile structure

The monopile structure is a traditional monopile (MP) build-up by cans welded together. It extends from its interface to the transition piece (TP) and continues all the way until the target penetration depth below seabed.
It includes a conical section to slowly increase the diameter of the overall structure towards seabed. The MP continues as tubular can sections penetrating and embedded in the seabed. A transition piece is considered to connect the MP with the tower base (base case). A cylindrical structure (whose horizontal extension is the tower base diameter) is designed to carry the access system, platforms, and other equipment. A skirt structure below the MP-TP interface height of 5 mLAT may further support the access system and seal the interface connection.

2.1.3. Embedded pile

Geotechnical design methodology follows the Design Basis [R1].

2.1.4. Variants

Two levels of water depth have been investigated for Site 1:
- 35 m also named C3 with embedded monopile diameter of 9.5 m
- 40 m also named C4 with variating diameters
  - Embedded monopile diameter 9.1 m
  - Embedded monopile diameter 9.3 m
  - Embedded monopile diameter 9.5 m

Following monopile has been investigated for Site 3:
- 60 m with embedded monopile diameter of 11 m

2.2. Concept designs Site 1

This section focusses on the monopile design for the Site 1 according to [R1]. Site 1 represents a location with a water depth of 35 to 40 m. General requirements like tower bottom diameter, hub height, turbine masses are considered according to [R1]. Also, the site conditions like water depth, extreme and normal sea state and wind conditions are included in the definitions of seabed pile diameter, height of the main access platform and positioning of the monopile cone.
An embedded monopile diameter of 9.5 m is considered as base case.

2.2.1. Load assessment and structural design

The structural design of the Monopile foundation depends on different conditions, which need to be fulfilled:

- Severe state limits (SLS)
  For the conceptual phase, especially the structural dynamics are from high interest. The entire structure must fulfil the eigenfrequency limits defined by the turbine supplier.
- Ultimate state limits (ULS)
  Structural integrity for occurring single extreme events includes the material limits like material strength and structural limits of shell buckling.
- Fatigue state limits (FLS)
  Structural failure for accumulated damages of material in numerous load cycle ranges occurring over the structural lifetime.

The design loads are based on conceptual load definitions for a realistic load estimation based on stated environmental conditions as described in the Design Basis [R1].

Loads at mudline level are used for a load extrapolation for the embedded pile from mudline down to pile tip. The analysis of the loads below mudline base on soil conditions described in the Design Basis [R1].

The loads include foundation and tower inclination due to installation tolerances and loads contingencies for loads uncertainties due to conceptual design phase.

The fatigue design considers structural damages for the turbine in operational conditions for a lifetime of 25 years and additionally two years of commissioning and decommissioning.
Regarding ULS, the load calculation is limited to DLC6.1.

2.2.1. Design verification

All above mentioned structural verifications have successfully been done. The resulting constructive overview drawing is shown in Figure 3.

Figure 3: Monopile general arrangement for Site 1 C4 -40mWD, diameter up to 9.5 m

2.2.1. Modal analysis

The global eigenfrequency of the system consisting of the wind turbine on top of the monopile foundation must be found in defined limits to ensure the functionality of the wind turbine and the avoidance of resonances. In first order the structure shall be stiff enough to avoid its excitation by the rotor frequency. Following table and figure summarize the first eigenfrequencies and mode shapes (for a symmetrical structure). Most important is the first eigenfrequency as this has high influence on the dynamic behaviour of the structure. The turbine restrictions are fulfilled.

Table 2: Eigenfrequency calculation for monopiles Site 1 (diameter 9.5 m)

Mode	EIgenfrequency 35 water depth	Eigenfrequency 40 m water depth	Comparison
1st Eigen Frequency	0.155 Hz	0.153 Hz	-1%
2nd Eigen Frequency	0.774 Hz	0.736 Hz	-5%
3rd Eigen Frequency	1.437 Hz	1.354 Hz	-6%
4th Eigen Frequency	2.548 Hz	2.453 Hz	-4%
5th Eigen Frequency	4.598 Hz	4.438 Hz	-3%
6th Eigen Frequency	7.112 Hz	6.701 Hz	-6%

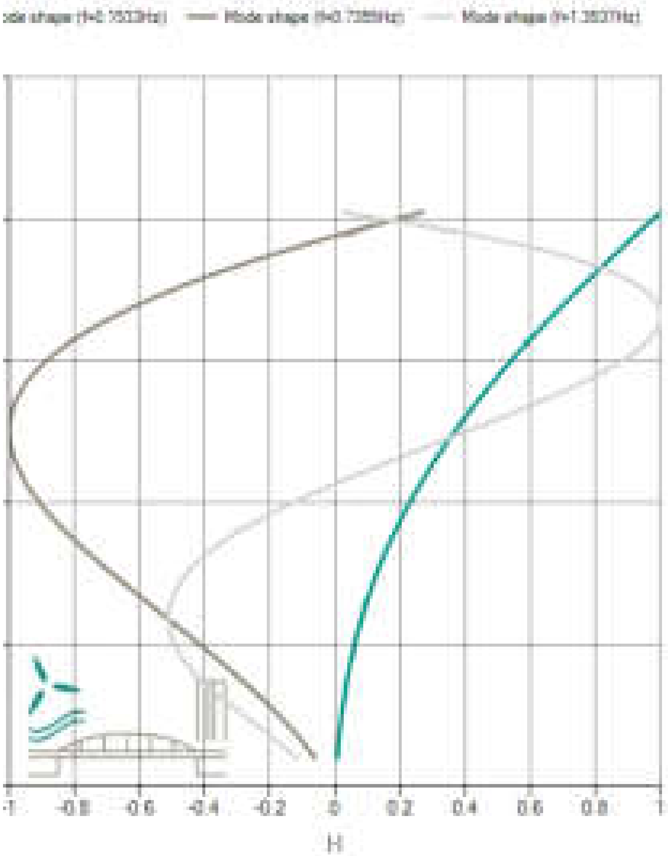


Figure 4: Mode shapes for foundation design Site 1 C4 -40mWD

2.3. Optimization study: Variation of diameter

In the early phase of the monopile design, the general dimensions of the structure are decided, in particular the distribution of the diameters. The diameter of the monopile has an influence on the loads from the waves and on the bedding of the pile in the ground as well as the embedment length. Therefore, the pile diameter at mudline shall be estimated good in an early state of the pile design. Later changes will produce an increment of the process time and work due to additional iteration loops.

The structural behavior for the steel foundation follows general rules. The increase of the pile diameter for a section results in:

- …an increase of wave loads for sections in wave induced zones,
- …a decrease of the structural wall thickness,
- …therefore, a decrease of the section mass for constant geometrical moment of inertia,
- …a decrease of pile embedded length,
- …an increase in the number of single plates and therefore the fabrication work

This section summarizes the results of the investigations of foundations of different diameters at mudline height. The investigation is based on the monopile Site 1 40 m water depth. Three designs were analysed with respect to the pile diameter variation:

Table 3: Design overview monopile structure study Site 1

Variation	pile diameter at mudline [m]
9p1	9.1
9p3	9.3
9p5	9.5

2.3.1. Eigenfrequency

The diameter variation has a direct impact to structural stiffness and therefore to structural eigenfrequencies, which is shown in Table 4 for the 1^{st} and 2^{nd} global bending modes.
The natural frequency mainly affects the fatigue loads. This becomes more important for structures with low natural frequencies of 0.15Hz or below. Thus, even if the changes in eigenfrequency appear small, it might have an impact depending on the site-specific sea states (in case the peak in the wave spectrum is close to the eigenfrequency).

Table 4: Eigenfrequency results monopile structure study Site 1

Variation	First (for-aft) EF [Hz]	Second (for-aft) EF [Hz]
9p1	0.151 (-1.3%)	0.721 (-2.0%)
9p3	0.152 (-0.65%)	0.727 (-1.2%)
9p5	0.153	0.736

2.3.2. Embedded Length

The geotechnical response to the pile depends on both the diameter and the fatigue and extreme loads at the ground surface. Since an increase in pile diameter results in an increase in the contact area with the soil per meter of embedded pile and an increase in the volume displaced by the pile, the increase in diameter generally results in a decrease in the required pile depth or embedded length. The impact of the variated range of diameter (4.3%) on the embedded length is within 2%.

2.3.3. Design optimization

Varying the diameter affects the design in various ways. The design variants have been verified regarding extreme and fatigue limit state. Resulting masses for the primary steel is listed in Table 5.

The general tendency of the steel masses with a variation of the pile diameter can be read directly from the results of this study. Aspects that also affect the decision on the diameter can be determined by the foundation manufacturer. Manufacturing becomes more complicated when the pile diameter is increased. These costs depend on the market and the manufacturer's limitations on plate dimensions and therefore cannot be considered in this study.

Table 5: Pile primary steel masses for Site 1 monopile structure study on varying diameters

Variation	Pile weight [t]	Comparison
9p1	1,606.8	+4.3%
9p3	1,582.0	+2.7%
9p5	1,540.4	Reference

The decision for the monopile Site 1 is based on the mass optimization resulting from this study. Therefore, the pile design 9p5 is used for further design evaluations. Concepts 9p1 and 9p3 will not be pursued further.

2.3.4. Detailed load analysis and sensitivity studies (DTU)

The aeroelastic model of the 15+MW conceptual wind turbine design with the JBO monopile is simulated in DTU’s in-house code HAWC2, using the DTU-WEC controller. The simulations are performed in an offshore Design Load Basis (DLB) for conditions of Site 1. ULS and FLS loads are compared with simulation results from JBO.

The sensitivity of the lifetime fatigue loads to the turbine availability is investigated (Figure 5, Figure 6), where the turbine availability is accounted for through the percentage of the occurrence of DLC 7.2 in the lifetime. It is seen that turbine availability has a considerable impact on the lifetime fatigue loads along the tower and monopile length, due to the contribution of the loads in the idling cases.

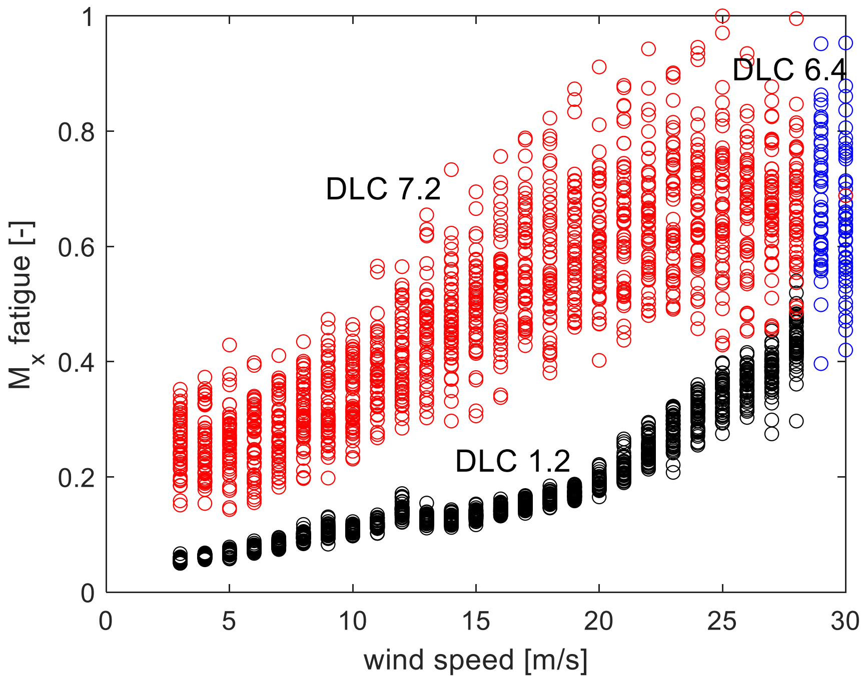


Figure 5: Short-term equivalent fatigue load statistics at the tower interface for different DLCs as a function of wind speed. Non-dimensionalized by maximum value.

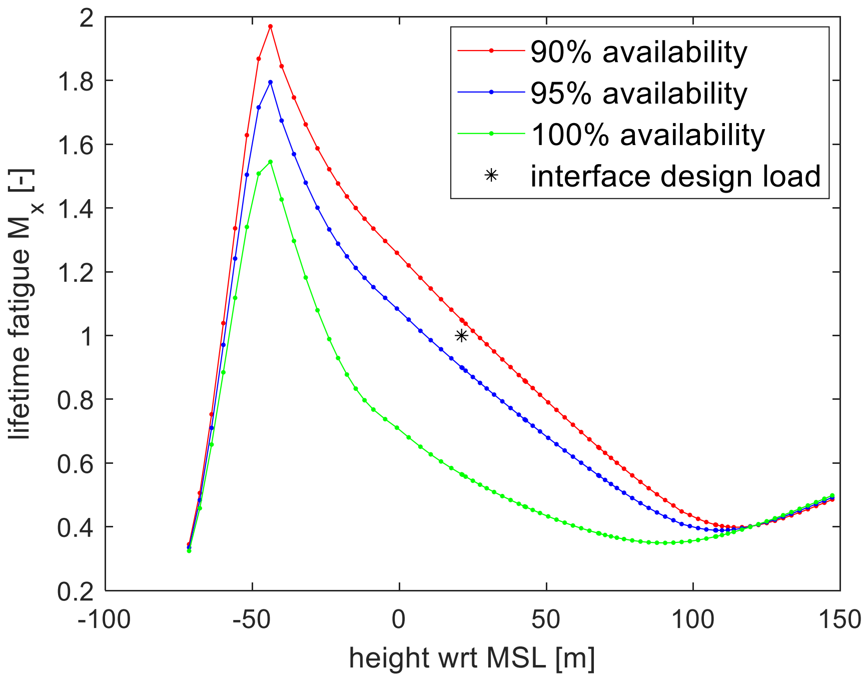


Figure 6: Lifetime fatigue load distribution along the tower and monopile length as a function of turbine availability. Non-dimensionalized by interface design load value.

2.4. Concept designs Site 3

This section focusses on the monopile design for the Site 3 according to [R1] with additional environmental input taken from an updated appendix [R2] to the Design Basis [R1]. Site 3 represents a typical location with 60m water depth. General requirements like tower bottom diameter, hub height, turbine masses are considered according to [R1]. Also, the site conditions like water depth, extreme and normal sea state and wind conditions are included in the definitions of seabed pile diameter, height of the main access platform and positioning of the monopile cone. The embedded pile diameter is iterated to be **11.0 m**.

2.4.1. Load assessment and structural design

Section 2.2.1 applies as well.

2.4.2. Design verification

All above mentioned structural verifications have successfully been done. The resulting constructive overview drawing is shown in Figure 7.

Figure 7: Monopile general arrangement for Site 3, -60mWD, diameter up to 11.0 m

2.4.3. Modal analysis

The global eigenfrequency of the system consisting of the wind turbine on top of the monopile foundation must be found in defined limits to ensure the functionality of the wind turbine and the avoidance of resonances. In first order the structure shall be stiff enough to avoid its excitation by the rotor frequency. Following table and figure summarize the first eigenfrequencies and mode shapes. Most important is the first eigenfrequency as this has high influence on the dynamic behaviour of the structure. The turbine restrictions are fulfilled.

Table 6: Eigenfrequency calculation for monopiles Site 3 (compared to Site 1)

Mode	Eigenfrequency Site 3 60 water depth	Eigenfrequency Site 1 40 m water depth	Comparison
1st Eigen Frequency	0.149 Hz	0.153 Hz	+3%
2nd Eigen Frequency	0.571 Hz	0.736 Hz	+29%
3rd Eigen Frequency	1.167 Hz	1.354 Hz	+16%
4th Eigen Frequency	2.238 Hz	2.453 Hz	+10%
5th Eigen Frequency	3.642 Hz	4.438 Hz	+22%
6th Eigen Frequency	5.579 Hz	6.701 Hz	+20%

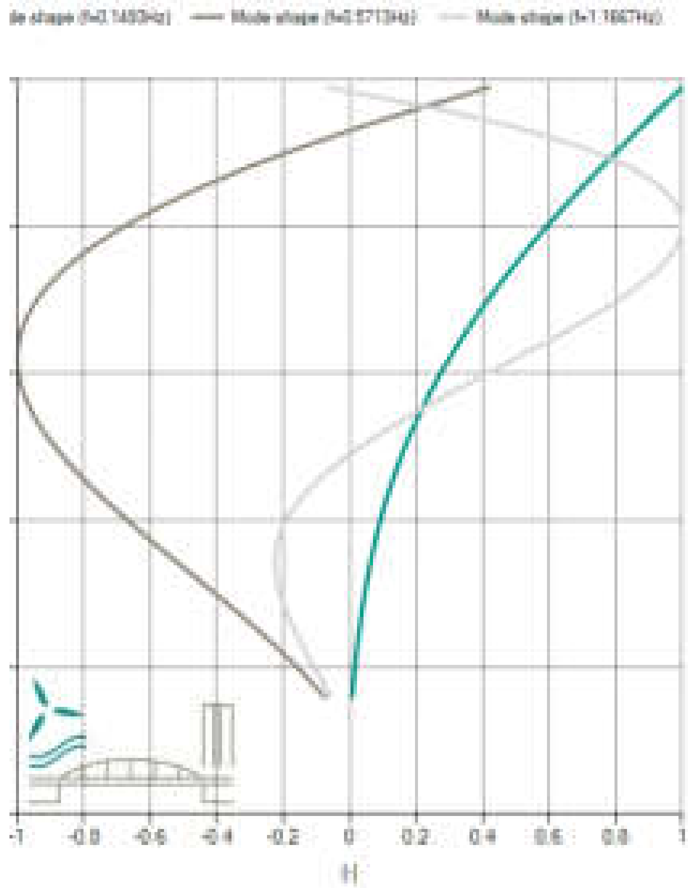


Figure 8: Mode shapes for foundation design Site 3, -60mWD

2.5. Optimization studies

Three additional optimization studies were performed, shown in this section:
- Influence of tower damping on fatigue loads
- Influence of aeroelastic damping on fatigue loads
- Influence of water depth on global sectioning

2.5.1. Parameter study tower damping

Structural damage due to fatigue loads is influenced by various environmental and technical limitations of the turbine foundation. The natural frequency of the new generation turbines with a rated power of 15+ MW and more is increasingly approaching the wave periods of the sea states that occur. Damping gets therefore a relevant factor. Total damping includes
- structural damping,
- soil damping and
- aerodynamic damping.

While the soil damping is a factor, which cannot be influenced, structural damping can be increased by active and passive tower damping, and aerodynamic damping depends directly on the turbine blades and the pitch control. Both aspects are investigated here.

Structural damping includes the foundation material, the soil and the built-in tower damping systems. For steel towers, a common damping ratio is assumed for the entire structure.

To get an idea on the influence of the tower damping to the structural damping, the tower damping was varied and analysed in terms of the fatigue damage at mudline for the different scenarios. Reference case is the tower with additional damping system. The analysis bases on the turbine design for Site 1 configuration C3 with 40m water depth.
Figure 9 shows that a structure with a typical tower damping system with a common damping ratio has about 23% less damage equivalent loads.

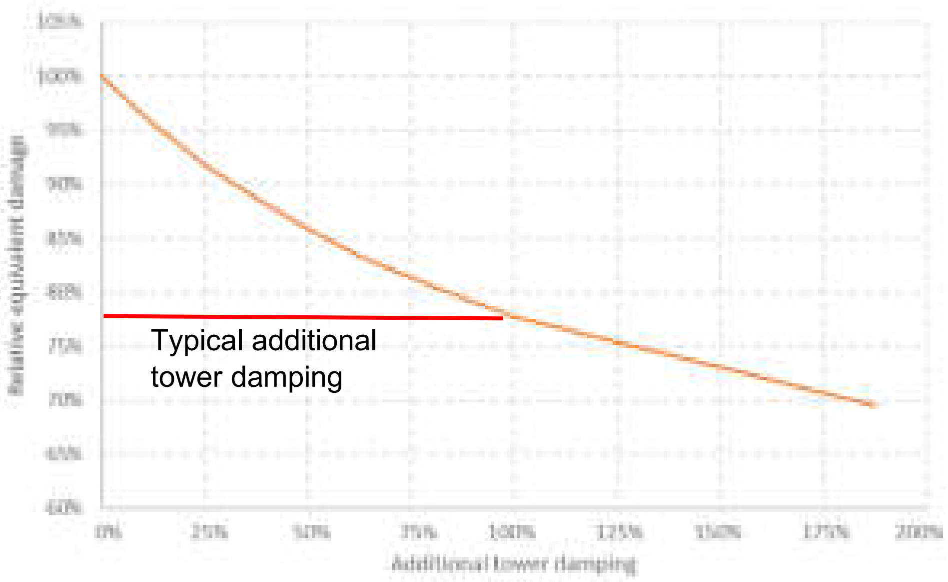


Figure 9: Relative fatigue damage depending on tower damping systems

2.5.2. Parameter study on turbine availability

Aerodynamic damping occurs from air flow around the structure. In particular, the blades of the rotating rotor, which are aligned with the wind direction, dampen the structure with a ratio of around 10%. This damping is significantly lower for an idling turbine. The total fatigue damage is therefore directly influenced by the availability of the turbine.

JBO analysed the relative equivalent damage loads for different turbine availabilities, ref. to Figure 10.

It can be concluded that a reduction of the stand still time for the turbines significantly reduces the fatigue damage loads due to the aerodynamic damping. Already a variation of 5% availability results in variations in fatigue loads of about 7-9%.

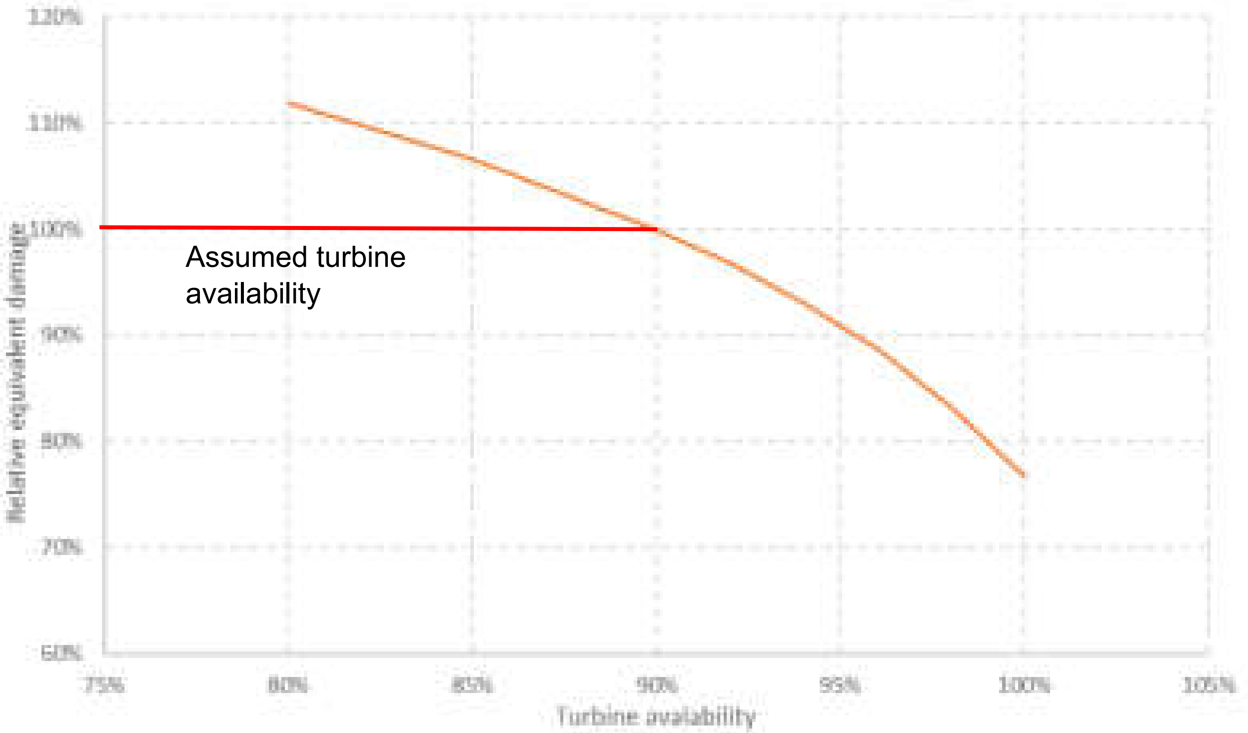


Figure 10: Relative fatigue damage for different turbine availabilities

2.5.3. Global Sectioning

The overall sectioning of the entire substructure including the Monopile foundation structure and the tower should be reflected in the light of varying water depth. The range of water depth is set from 40 to 70 m for this investigation. The weight per pile length is derived according to a generic scaling rule. The weight per pile length was increased by 10% per 10 m water depth. Monopiles have been designed for 40 m and 60 m water depth, see Sections 2.2 and 2.4. Figure 11 shows the correlation between the design results, the generic rule, and the linear extrapolation of historic designs by JBO. The historic JBO piles contain about 10 designs for the 15 MW turbines class in water depth from 20 to 45 m. The data show a linear relationship between all results despite the variety of turbines, geotechnical and marine site conditions in first order. It needs to be mentioned, that the linearity is limited to a certain range of water depth.

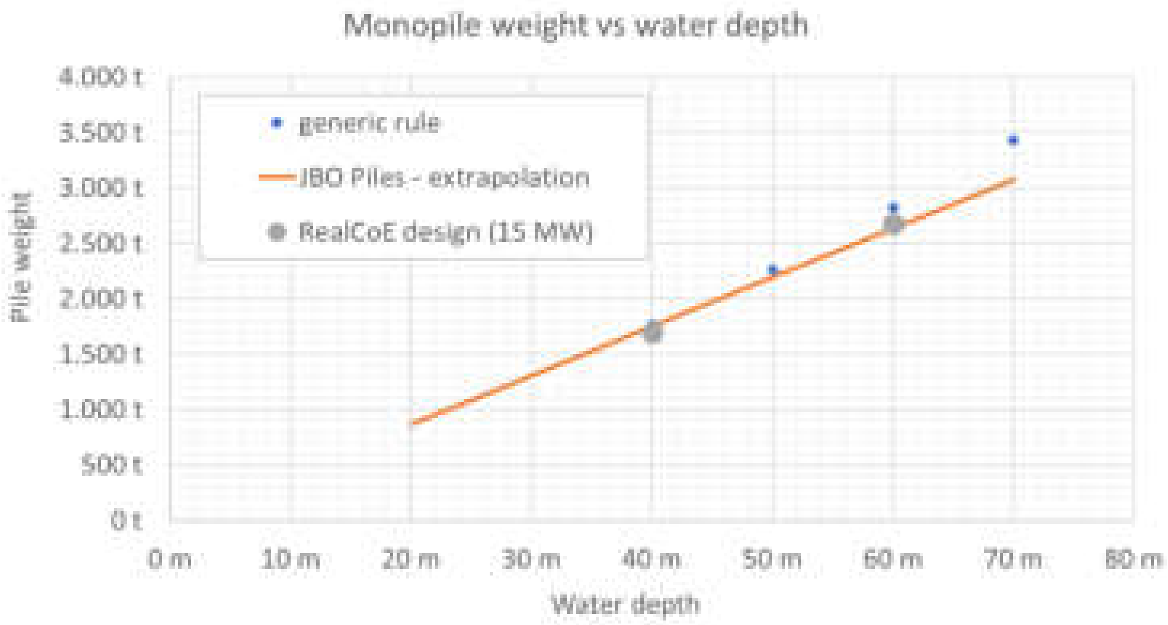


Figure 11: Monopile weight vs water depth

Weight limitations of sections to be assembled on site were set for maximum section dimensions in accordance with the state of the art, as shown in Table 7. Differences between tower and monopile limitations are caused by different fabrication requirements. It may happen, that length limitations become relevant in the further process. A section within this study is defined as single piece of monopile, transition piece or tower.

Table 7: Section limitations

Section	Mass limitation used for sectioning	Length limitation used for sectioning
Tower	350 t	no
Monopile	2,000 t	no

The results are given in Table 8. The level of the external platform is assumed at 20 m LAT, but the tower bottom level is varying depending on the sectioning. Thus, the external platform might either be at the TP or at the lowest tower section. The embedded pile length is increased accordingly.

Table 8: Results for global sectioning of the structure

	40 m	50 m	60 m	70 m
Number of sections (pile tip to tower top)	3	5	6	6
Structural mass incl. tower	2.700 t	3,200 t	3,700 t	4,400 t

Each case can be characterized as follows:

40 m

A single piece of monopile of about 2,000 t with a total length of 114 m (75 m + 39 m) is considered. There would be sufficient design space to provide a TP-less Monopile. Once this height is achieved, only two tower sections would be sufficient to achieve the full height to the upper end of the tower.

50 m

The monopile with about 1,900 t will achieve the LAT level. Its length is with 90 m more sound compared to the 40 m version. A transition piece will be mandatory in this condition. The interface is nominal at/close to sea level, what is also today the case with often about +5 m LAT. But environmental impact needs to be addressed when the joint is designed. The height of the TP was chosen to reduce the number of tower sections to a number of three.

60 m

The monopile for the 60 m water depth situation stays under water, here about 17 m below LAT. It weighs about 2,000 t and is 88 m long. This means that the TP needs to be mounted in submerged condition. A TP of 37 m length would result in about 800 t. Such TP would reach the platform level as usual today. The tower would consist of four sections.
This view correlates to the result for Site 3, where the monopile reaching out of the water ends up with a weight of about 2,400 t, exceeding the present limit of 2,000 t.

70 m

The monopile loses length furthermore. It is now 82 m long. The pile head is at -38 m LAT nearly at the half of the water depth. The TP must compensate the height up to tower button. It would be about 58 m with a weight of about 1,450 t, what is more a Monopile dimension than a traditional transition piece.

2.5.4. Connection types of MP / TP / tower sections

Several connection types are available to connect section of the monopile, transition piece and tower, where two are shown in Figure 12: The bolted flange connection and the grouted connection.

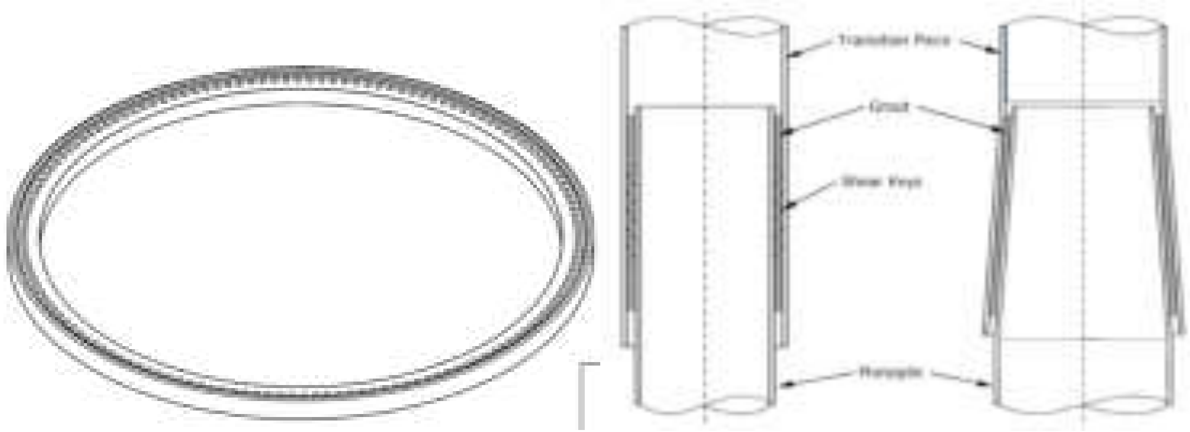


Figure 12: Connection types: bolted flange connection (left) and grouted connections (right) [L18]

The bolted connection is commonly used for all interfaces above the external working platform. Regarding the connection of the monopile to the TP or lowest tower section the choice becomes more specific. If the interface lies above sea level (e.g. 50 m water depth as shown in Section 2.5.3) bolted connection and grouted connection are feasible. For water depths of 60 m or more the monopile ends in the submerged area and thus, a grouted connection becomes a feasible solution. Due to the fact, that a conical section is typically considered in this area, the grouted connection could also become a conical shape. This would also allow further adjustments of this connection. E.g. a slip joint could become an economic option allowing a simplified installation, but further investigation and prototypic testing will be needed to use this connection type in future. A Slip joint could also be an alternative for bolted connections after reaching the limitation of L-flanges at certain dimensions.

2.6. Conclusion Monopile

The applicability of the design tools for monopiles for a 15 MW+ turbine was demonstrated in this chapter. The resulting weights were shown in the table below:

	Site 1 40 m water depth	Site 3 60 m water depth
Monopile weight	1,541 t	2,475 t
TP weight	233 t	310 t
Total substructure weight	1,774 t	2,785 t
Contingency factor	15%	15%
Embedded length	31.5 m	36 m
Embedded diameter	9.5 m	11 m
Lowest eigenfrequency	0.153 Hz	0.149 Hz

The scaling factor of the monopile weight and sizes could be derived based on the two selected sites with water depth of 40 and 60 m. The steel mass increases significantly and the eigenfrequency for the 60 m system decreases somewhat, which increases the risk of possible resonances. The diameter has been investigated for Site 1 resulting in slight advantages for the largest diameter of 9.5 m.

Three additional optimization studies were performed: At first a parameter study on the effect of tower damping was done. It was shown that by an additional tower damping in common range the fatigue loads can be reduced by about 23%. This is highly relevant because of the effect of the loads to the damage is powered by 3 or 5. Furthermore, the impact of aeroelastic damping has been analysed by varying the turbine availability. It was found that a reduction of 5% availability results in an increase of fatigue loads of about 7-9%.

Finally, options of sectioning of the support structure (pile tip to tower top) were investigated. Three parts would allow to reach the hub height at a water depth of 40 m for the given assumptions. A submerged offshore assembly joint comes into the consideration at about 60 m water depth.

Meanwhile the design of monopiles is well mastered for the large turbine, however, for increasing water depth the effort for the foundation is increasing significantly. Economic considerations regarding the profitability of wind farms under extreme environmental conditions may increase.

3. Jacket conceptual design

3.1. General

Following jacket designs are investigated by JBO with the GE next generation 15+MW turbine in the subchapters of this Chapter 3 and compared regarding the design results:

	Site 1 (40m WD)	Site 3 (60m WD)
Footprint 23.25m x 23.25m	Section 3.2	Section 3.3
Footprint 30m x 30m	Section 3.2	Section 3.3

The jacket designs are compared to each other w.r.t. weight and frequencies.

All designs presented here are on the stage of a conceptual design. Wind turbine loads at tower button and representative wave loads were considered for ULS design. The utilisation of the structure is limited to a certain extend because fatigue loads were not considered in this stage. This approach is usual for conceptual jacket design. Soil conditions are generic ones. They need to be analysed in more detail for a specific project site. All design results are presented without contingency.

The layout of the jacket structure is decided based on several parameters:

- Statutory and regulatory requirements,
- Employer's requirements.
- Site specific conditions, such as wind, wave, current, and soil profiles over the site.
- Turbine characteristics, such as tower length and tower bottom diameter.
- Practical limitations due to fabrication, installation, and others.
- Design Basis [R1] is applied and additionally [R2] for Site 3

The main structural dimensions are defined below as:

- Turbine Power at 15+ MW
- WTG interface level: +24.0…+26.5 mMSL incl. TP (depending on the site)
- 4-legged jacket with pre-piled piles
- Pile stick up: 8.00 m
- Head print of jacket: 15.00 m
- Foot print (FP) of jacket: 23.25 m / 30.00 m (varied)

The design process follows the general procedures defined by the DNVGL guidelines.

A summary of the jacket conceptual designs is shown in the following subchapters.

3.1.1. Load cases and combinations

Environmental conditions, load cases and load combination are based on [R1]. For the conceptual phase, DLC6.1 (max non-operational wind load combined with max wave height $H_{max,50}$) is assessed.
Load combinations will be carried out according to ISO 61400-3 [N27].

3.1.2. Material properties

The steel types and yield strengths for primary structures are in accordance with EN 10025 [N22].
Can sections which are estimated to be subjected to stresses in through-thickness direction are chosen with Z-quality steel grade.
The steel properties used in the analysis are chosen acc. to EC3.

3.1.3. Corrosion allowance

A lifetime of 27 years for the substructure is assumed for calculation of the corrosion allowance. For parts inside the splash zone a corrosion allowance of 0.3 mm/year for the outer surface according to DNV is assumed. Taking into account that the steel coating resists to the sea water at least 15 years for primary steel, a corrosion allowance over the remaining 12 years of lifetime results to:

12.0 yr * 0.3 mm/yr = 3.6 mm. (Outer surface)

Corrosion will be applied at the outer surface inside splash zone. Corrosion is applied only at the inner surface of the legs assumed flooded.

3.1.1. Soil model

The soil structure interaction is modelled by the inclusion of non-linear T-z, Q-z and P-y curves for the axial shear resistance, axial end bearing resistance and lateral soil resistance respectively.

3.1.1. Scour

A scour of 4.55 m is considered and is applied to the model.

3.2. Concept Jacket designs Site 1 (40m)

The substructure is a 4-legged jacket with pre-installed piles. The connection between jacket and piles will be established by a grouted joint. The jacket model has one boat landing as access system and two J-tubes.

There are two jackets designed for Site 1 with varying footprint:

- Base case jacket with a footprint of 30 m x 30 m.
- Optimized jacket with a footprint of 23.25 m x 23.25 m (value defined by fabrication limitations)

The wave loads and structural verifications have been performed with the software SACS, numerical calculation models are shown in Figure 13. The Geotechnical design has been performed using an in-house tool.

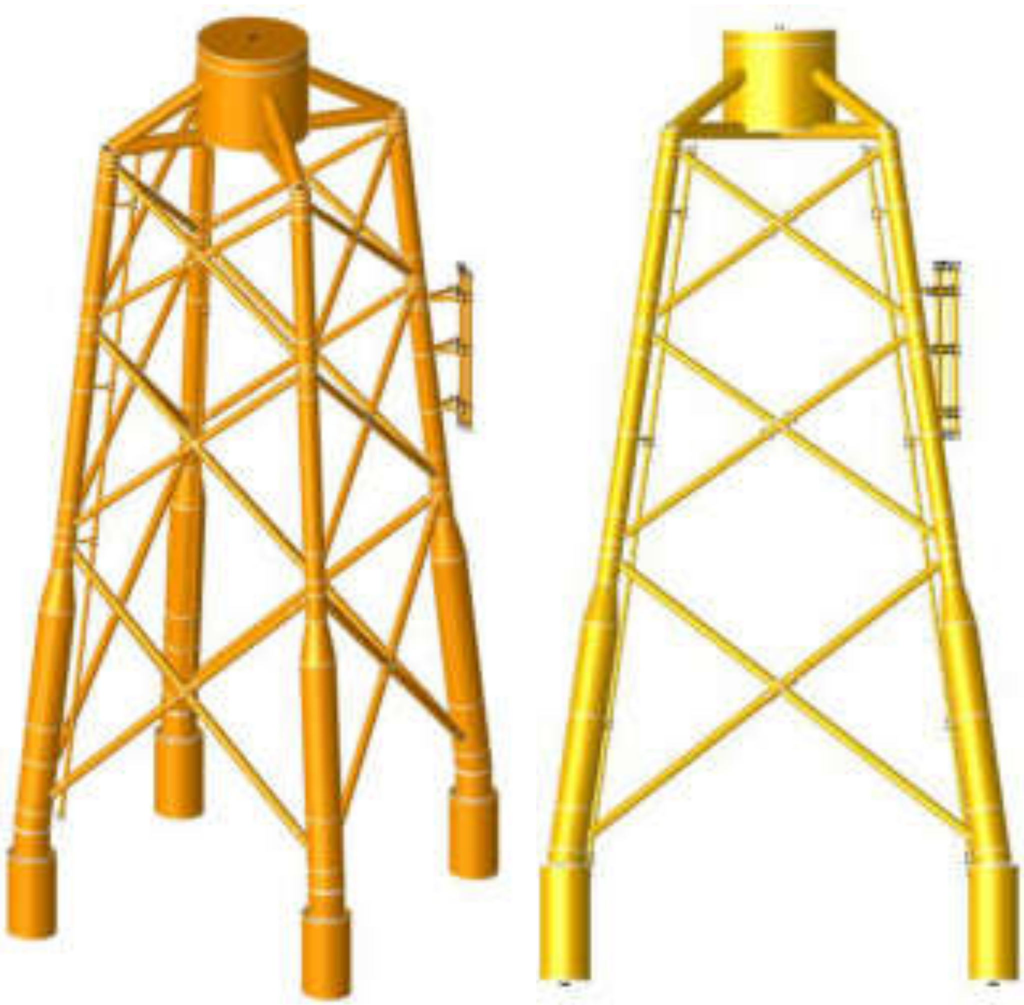


Figure 13: Jacket Site 1 (40 m water depth), left: 23.25m FP, right: 30m FP

Table 9: Pile length of jackets – 40 m water depth

	23.25m x 23.25m FP	30m x 30m FP
Pile stick up length	8 m	8 m
Embedded pile length	**37.71 m**	**30.28 m**
Overall length	45.71 m	38.28 m

A resulting summary of all masses is given in the table below without adaption of contingencies w.r.t. to further design stages.

Table 10: Mass summary jackets – 40 m water depth

Mass summary	Mass 23.25m FP	Mass 30m FP	Comparison 23.25 m to 30.0 m FP
Jacket – primary steel	760t	887 t	-14% Braces are shorter at smaller footprint
Jacket – secondary steel	50 t	50 t	-
Transition Piece	282 t	282 t	-
Piles	945 t	883 t	+7% Piles have the lowest specific price. The increase might be reasonable.
Total	**2,037 t**	**2,102 t**	-3% The overall weight of the jacket including TP and Piles is 65 t less

3.2.1. Eigen modes

Eigen frequency analysis has been performed using SACS software. The model consists of RNA, tower, jacket substructure including TP, one boat landing, 2 J-tubes and the pile foundation implemented as super element. The tower was modelled as described in [R1]. RNA was modelled as described in [R1].
The allowable first global bending eigenfrequency range is specified in the design basis [R1] to avoid resonance with 1P and 3P excitation.
The following eigenfrequencies have been calculated for the structure ready for operation.

Table 11: Eigen frequency calculation for jackets Site 1

Mode	Natural Frequency FP 23.25m	Natural Frequency FP 30m	Comparison
1st Eigen Frequency	0.189 Hz	0.193 Hz	+2%
2nd Eigen Frequency	0.189 Hz	0.194 Hz	+3%
3rd Eigen Frequency	1.213 Hz	1.547 Hz	+28%
4th Eigen Frequency	1.216 Hz	1.565 Hz	+29%
5th Eigen Frequency	1.808 Hz	3.093 Hz	+71%
6th Eigen Frequency	1.841 Hz	5.307 Hz	+188%

Selected eigenmodes are shown in the figures below exemplary for FP 23.25m:

1st side-to-side global shape

2nd Fore-Aft global shape

3.3. Concept Jacket designs Site 3 (60m)

The substructure is a 4-legged jacket with pre-installed piles. The connection between jacket and piles will be established by a grouted joint. The jacket model has one boat landing as access system and two J-tubes.

There are two jackets designed for Site 3 with varying footprint:

- Base case jacket with a footprint of 30 m x 30 m.
- Optimized jacket with a footprint of 23.25 m x 23.25 m

The wave loads and structural verifications have been performed with the software SACS, numerical calculation models are shown in Figure 13. The Geotechnical design has been performed using an in-house tool.

Figure 3-14: Jacket Site 3 (60 m water depth), left: 23.25m FP, right: 30m FP

Table 3-12: Pile length of jackets – 60 m water depth

	23.25m x 23.25m FP	30m x 30m FP
Pile stick up length	6 m	6 m
Embedded pile length	**53.32 m**	**39.20 m**
Overall length	59.32 m	45.20 m

A resulting summary of all masses is given in the table below without adaption of contingencies w.r.t. to further design stages.

Table 3-13: Mass summary jacket – 60 m water depth

Mass summary	Mass 23.25m FP	Mass 30m FP	Comparison 23.25 m to 30.0 m FP
Jacket – primary steel	911 t	968 t	-6% Braces are shorter at smaller footprint
Jacket – secondary steel	50 t	50 t	-
Transition Piece	282 t	282 t	-
Piles	1,094 t	891 t	+23% The pile mass is higher since the footprint is smaller
Total	**2,337 t**	**2,191 t**	+7% The overall weight of the jacket including TP and Piles is 146t more

3.3.1. Eigen modes

Eigen frequency analysis has been performed using SACS software. The model consists of RNA, tower, jacket substructure including TP, one boat landing, 2 J-tubes and the pile foundation implemented as super element. The tower was modelled as described in [R1]. RNA was modelled as described in [R1].
The allowable first global bending eigenfrequency range is specified in the design basis [R1] to avoid resonance with 1P and 3P excitation.
The following eigenfrequencies have been calculated for the structure ready for operation.

Table 14: Eigen frequency calculation for jackets Site 3

Mode	Natural Frequency FP 23.25m	Natural Frequency FP 30m	Comparison
1st Eigen Frequency	0.190 Hz	0.194 Hz	+2%
2nd Eigen Frequency	0.190 Hz	0.195 Hz	+3%
3rd Eigen Frequency	1.249 Hz	1.373 Hz	+10%
4th Eigen Frequency	1.256 Hz	1.385 Hz	+10%
5th Eigen Frequency	2.414 Hz	2.508 Hz	+4%
6th Eigen Frequency	3.839 Hz	4.065Hz	+6%

Selected eigenmodes are shown in the figures below exemplary for FP 30m:

1st side-to-side global shape

2nd Fore-Aft global shape

3.4. Design of Transition Piece

The Transition piece has a height of 7 m. A concept design has been established to carry the tower bottom loads to the jacket legs. An overview is shown in Figure 15.

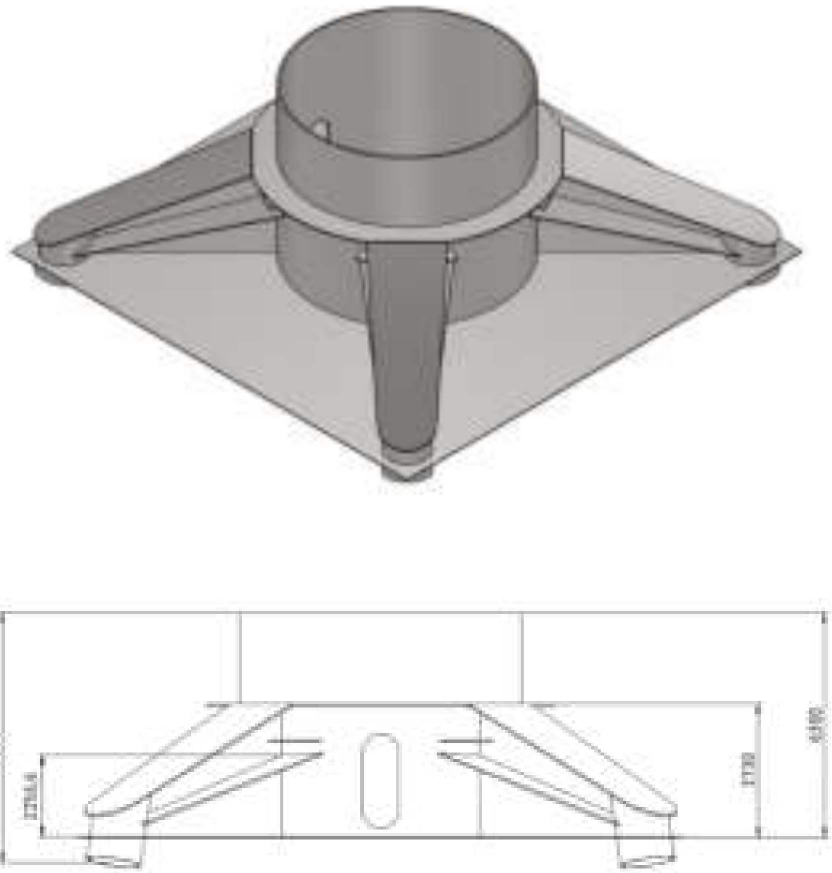


Figure 15: Transition piece used for the concept designs

3.5. Conclusion (Jacket)

Within the ReaLCoE project jacket foundations have been designed for the GE next generation 15+MW offshore turbine for Site 1 (40m water depth) and Site 3 (60m water depth). The investigation was made on a conceptual design stage leading to a certain contingency in the design. For both sites the Jacket footprint is varied from 23.25m to 30m resulting in following mass summary:

Footprint	Component	Site 1 (40m) mass	Site 3 (60m) mass
23.25m	Jacket	760 t	911 t
	Piles	945 t	1,094 t
	TP + sec. steel	332 t	332 t
	Total mass	**2,037 t**	**2,337 t**
30m	Jacket	887 t	968 t
	Piles	883 t	891 t
	TP + sec. steel	332 t	332 t
	Total mass	**2,102 t**	**2,191 t**

The mass summary indicates that for 40m water depth the 23.25m footprint is the preferred solution whereas for the 60m location the 30m footprint results in a lighter design. This is caused by need of a very high required pile length at deep location with reduced footprint. The reduced footprint was chosen due to constrains from fabrication and logistics after consultation with Work Package 2 of the project.

There can be seen a quite small increase in mass from Site 1 (40m) to Site 3 (60) of only 154 t (7.5%) indicating that the jacket foundation may become more competitive for higher water depths.

Finally, there should be an investigation on the cost site including fabrication, transport, installation and maintenance aspects. For project specific designs a higher detailed level in the design process and more precise basis of design is required.

4. Gravity Base Foundation (GBF) conceptual design

With the purpose of comparing different foundation concepts JBO has investigated a conceptual design of a Gravity Base Foundation (GBF) for the GE next generation 15+MW offshore turbine besides Monopile and Jacket foundations. A summary on the GBF conceptual design is shown in the following subchapters.

4.1. General

Currently, monopiles (MPs) are the favoured foundation type for offshore WTG. Due to increasing requests and higher steel prices, more and more alternatives to the proven monopile foundation are currently being designed and analysed. Various projects have already shown that **Gravity Base Foundation**s (GBFs) are often preferred for structures in shallow, stable waters or rocky surface ground (s. Figure 16 for an overview of foundation types).

GBFs must be designed to prevent uplift or overturning (i.e., no tension between the support structure and seabed). Stability to the structure against the action of overturning moments shall be provided by an adequate dead load. In case of insufficient dead weight of the structure itself including Tower and RNA, additional ballast (e.g., rock, sand, or concrete) will be needed.

Various geometries for the base plate of the foundations have been investigated in the past for various GBFs. A selection can be found in Figure 17. For example, foundations with a square, rectangular, hexagonal, or circular base were used. For the stability and integrity of the GBF structure, concrete caisson / bucket structures, partitioned cells for ballast or cylindrical sections have already been used.

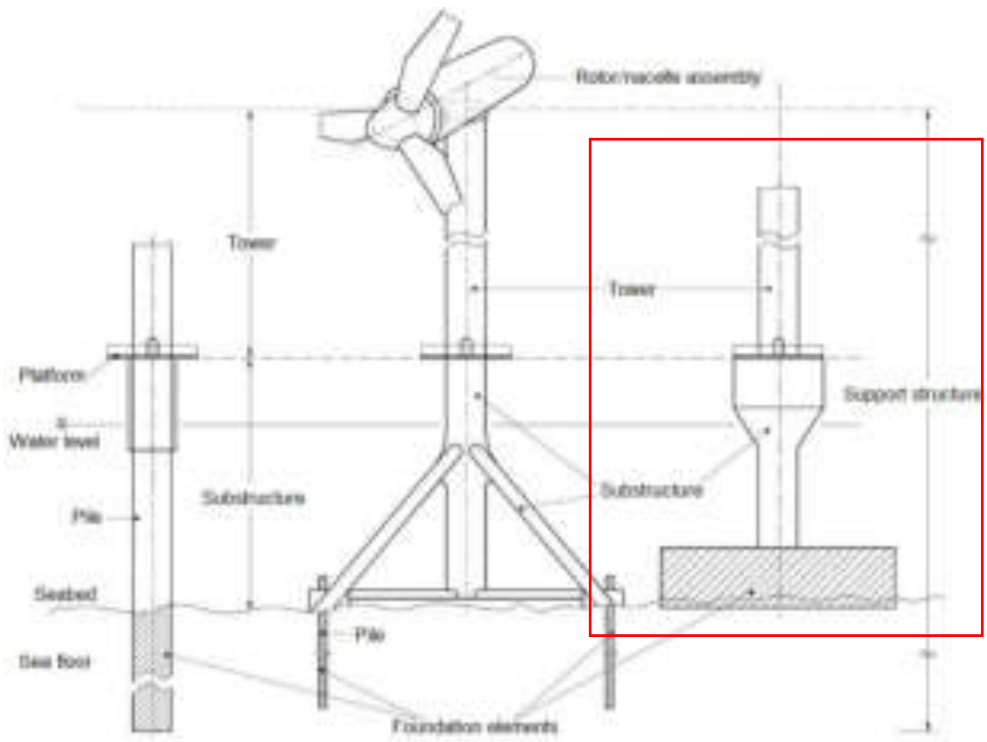


Figure 16 Definition of offshore wind turbine primary foundation components acc. to [N1]

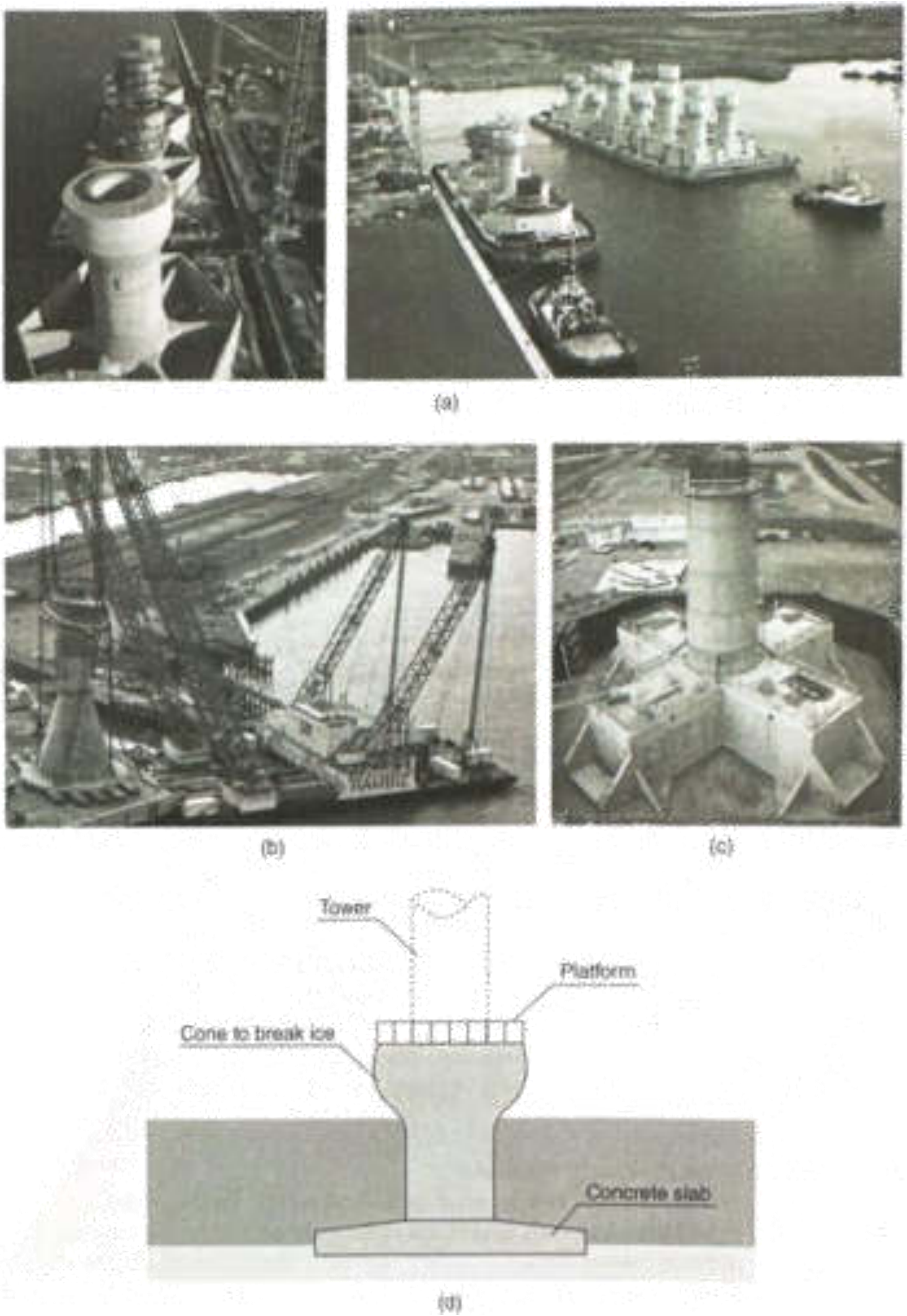


Figure 17: Various examples of shapes of GBFs according to [L16]: (a) Example of GBS (transportation for Karehamn wind farm – Sweden); Courtesy: Jan DE Nul Group. (b) GBS from Thornton Bank project; (c) GBS – Strabag concept; and (d) foundation for Middelgrunden wind farm (Denmark) – shallow gravity-based foundation.

The production of GBFs is less expensive than steel foundation types but demand a large fabrication yard and storage area. Concrete foundations are heavier than equivalent steel ones. Therefore, larger T&I tools such as cranes and vessels are needed. Increasing the depth for offshore wind farms the GBF foundation is getting heavier, so that transport and installation may get more challenging.

The goal of this chapter is to provide a design for a gravity-based foundation (GBF) on a conceptual design level for deeper waters. This conceptual design case, a concrete GBF shall be designed considering the following conditions according to [R1]:

- Site 1,
- 40 m water depth,
- Good conditions

The layout of the gravity-based foundation (GBF) is decided based on several parameters:

- Statutory and regulatory requirements, such as governing design codes and local requirements.
- Employer's requirements.
- Site specific conditions, such as wind, wave, current, and soil profiles over the site.
- Turbine characteristics, such as tower length and tower bottom diameter.
- Practical limitations due to fabrication, installation, and others.

The main structural dimensions are defined in accordance with [R1] as:

- Turbine Power at 15+ MW
- WTG interface level at +21.2 mMSL

The design process follows the general procedures defined by IEC 61400-1 and the DNV guidelines.

4.2. Conceptual Design GBF for 15MW+ turbine class

4.2.1. General structural concept

This GBF is designed as a concrete conus part with sand filling. These foundation types are typically made with in-situ concrete or precast concrete units.

To avoid inclination of the substructure, this foundation type may require a seabed preparation.

The geometry of the substructure analysed in this concept design consists of a cylindrical tube with an outer diameter of 10.0 m and height of 31.2 m and a conically shaped part with an outer diameter of 30.0 m and height of 28.0 m. Both the cylinder and the cone feature a wall thickness of 0.60 m. At the transition between the substructure and the tower as well as between the cylinder and cone the wall thickness is increased to 1.50 m.

The cone is supported by a ring foundation with an outer diameter of 40.0 m and an inner diameter of 20.0 m. The ring foundation has a thickness of 2.0 m.

The total height of the substructure adds up to 61.2 m from mudline to interface.

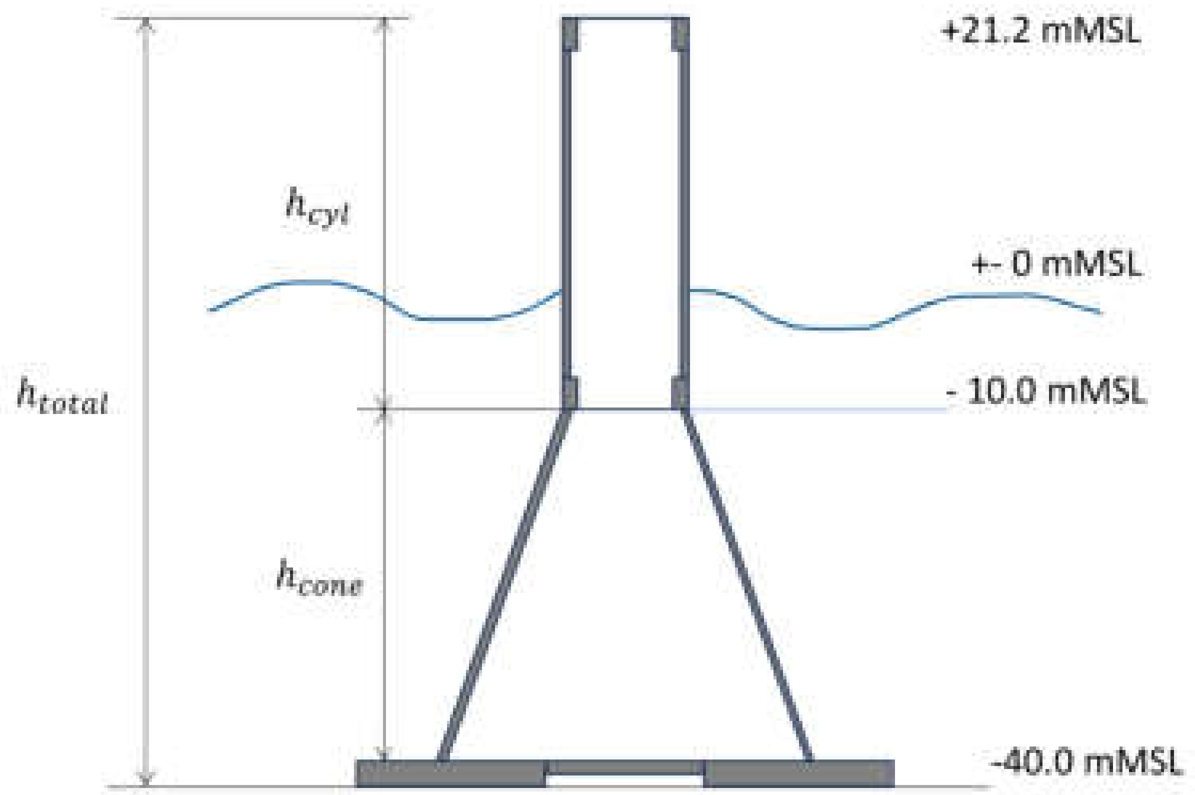


Figure 18: Schematic drawing of GBF cross section:

4.2.2. Site conditions

The site conditions for Site 1 with 40m water depth are defined in the Design Basis [R1]. Apart from this, for the soil conditions it is assumed that the first two layers are replaced with cohesionless sand during construction/preparation of the seabed. For the ULS design a characteristic angle for internal friction of $\varphi' = 35°$ was assumed.

4.2.3. Materials

The gravity-based foundation is made of reinforced concrete C45/55 (ring foundation, cone and middle part of the cylinder) and C55/67 (lower and upper shaft) with properties provided in [N11].

4.2.4. Load assumptions

Permanent and variable loads are considered acc. to IEC 61400-3-1 [N27]. Furthermore, wind loads are given in [R1].

The self-weight of the gravity-based foundation and the weight of the ballast is calculated iteratively during the design. Marine growth is not considered as it is increasing the stability of the foundation. For the concrete an effective submerged unit weight of $\gamma = 1400$ kg/m³ is used conservatively for the whole structure.

Wave-induced loads acting on the substructure are calculated by means of Morison's equation. The applicability of Morison's equation may be violated due to diffraction effects of the great diameter of the GBF near seabed. However, diffraction is considered appropriately within the calculation. Hydrodynamic load simulations are carried out to determine the loads caused by the operational and 50-years extreme wave. The loads were calculated using an internal JBO program that determines the loads according to [N27].

The load-sided safety factors are chosen in accordance with Eurocode 1 [N9] and Eurocode 7 [N18].

The combination factors are chosen based on DIN 18088-1 [N54]. The wave conditions are considered independent of the wind conditions, so no corelation between wind and wave has been considered and no integrated load analysis has been performed. This assumption is conservative as the maximum wave load does not necessarily correspond to the maximum wind turbine loads.

4.2.5. Ballasting

A ballasting is foreseen as sand filling (may be replaced by other material) limited to the conical part of the GBF. The ballasting is required to ensure stability of the turbine against overturning.

4.2.6. Design verification

Following design verifications have been performed during the conceptual design:

- Geotechnical verifications
  - Ultimate bearing capacity: fulfilled
    The bearing capacity of the foundation is calculated with an effective foundation area that is required when eccentric loading is considered. The calculation of the effective area is carried out acc. to DNV-RP-C212
    The weight of the ballast is limited by the inner volume of the cone to ensure feasibility. Conservatively, the maximum shear force and bending moment are superimposed.
  - Sliding resistance: fulfilled
    Foundations subjected to horizontal loading must be investigated for sliding resistance as the horizontal force can cause sliding of the foundation over the seabed. The sliding resistance is reduced by 30% to reflect the structure-soil interaction.
  - Stability check: fulfilled
    For the stability check the stabilizing moment shall be greater than the overturning moment at the edge of the foundation ring. The load calculation for the stability check is carried out using the load safety factors according to Eurocode 7 [N18] for EQU and the load combination factors acc. to DIN 18088-1 [N54].
  - SLS verification: fulfilled
    According to Eurocode 7 [N18] the eccentricity of the permanent and variable loads for the serviceability limit state (SLS) shall not be greater than 0.59 times the radius of the foundation.

- Structural verifications
  The load calculation for the structural verifications is carried out with the RFEM model. The concrete structure is defined by surfaces with varying thicknesses. The surface bedding is defined with the RF-SOILIN.

  - Prestressed concrete investigation: fulfilled
    The verifications are carried out at the critical sections where the internal forces are shown in the load document.
    Possible prestressing for tendons has been determined.
    Clamping force losses have been determined.
    Preload has been selected.
  - Serviceability limit state for the shaft: fulfilled
    According to the standard, decompression does not have to be verified for WTGs. However, when determining the cross-section dimensions as part of the preliminary design, experience has shown that compliance with decompression in DLC 1.0 (D3) leads to a good estimation of the geometry.
  - ULS verification in levels: fulfilled
    Verifications for tensile stress and decompression have been performed in levels +21.20 m, -10.0 m and -40.0 m.
    Verifications for the shaft have also been performed in levels +21.20 m, -10.0 m and -40.0 m.

- <u>Minimum reinforcement for crack width limitation: fulfilled</u>
  This has been Verified for following positions:
  - Shaft 150 cm for vertical and horizontal reinforcement routing
  - Shaft 60 cm for vertical and horizontal reinforcement routing
  - Shaft 45cm horizontal in the area of the connection to the foundation
  - foundation h = 100 cm (two concreting sections)

- Natural frequency check
  The dynamic characteristics of the foundation-tower-system are analysed in form of natural frequencies to describe the dynamic behaviour of the system. The eigenfrequencies depend mainly on the stiffness of the tower itself and the dynamic stiffness of the foundation.

  The 1st and 2nd Eigenform are inside the allowable first global bending eigenfrequency range, s. Table 15. Therefore, there are no concerns regarding the natural frequency of the gravity-based foundation. The first eigenform is shown in Figure 19.

Table 15: Results modal analysis

Eigenform	Frequency [Hz]
1st tower bending mode	0.196
2nd tower bending mode	0.198

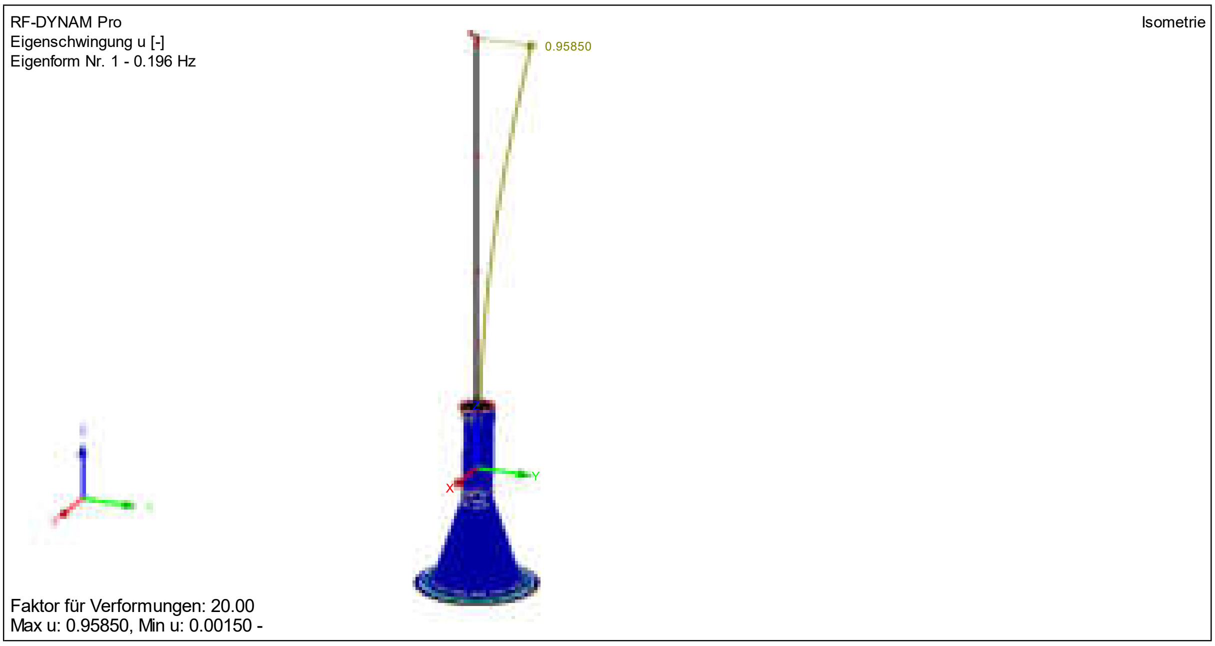


Figure 19: 1st Eigenform

4.2.7. Construction detail: Connection Tower – GBF

The connection from the tower to the concrete shaft of the gravity base foundation is exemplary done for a T-flange with two cycles of anchor bolts. Other solutions could become more beneficial after further design loops.
The shank (cylindrically shaped part of the GBF) is prestressed by internal guided tension members that run inside the shank in a vertical direction with subsequent bonding (alternatively without bonding).
The strand tensioning method VSL Tensioning Systems or equivalent is used. Type 6-27 tendons are used. In the following figure, a detail of the flange–concrete shaft is shown.

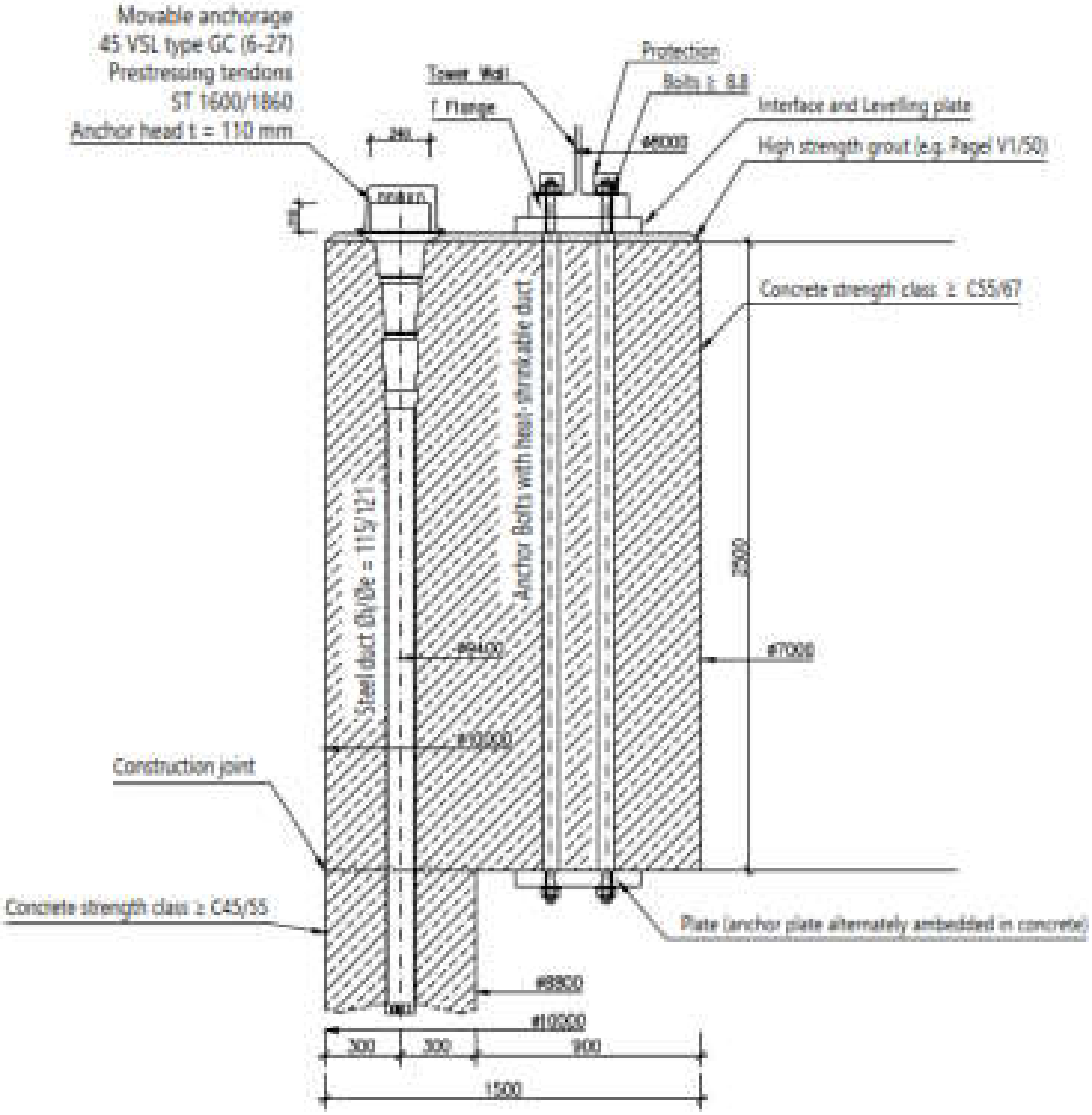


Figure 20: Detail flange - concrete shaft

4.3. Main results

As a result, we have obtained a gravity base foundation over a water depth of 40 m with a total height of 61.2 m and a base plate diameter of 40 m. The entire main supporting structure of the GBF is a rotationally symmetrical body in the form of a cylinder, which is connected to a cone and is expanded via the base plate. Table 16 shows the overall mass summary Secondary steel elements such as an access system, a main access platform or possible internal platforms are not yet part of this concept design.

Further investigations will be necessary in the course of the detailed design. Especially a scour protection may be needed depending on the specific site conditions.

Table 16: Total estimated masses of the GBF

Total estimated masses	
Parts	**Masses**
	[to]
Concrete C45/55	9371,28
Concrete C55/67	259,18
Ballast	16000,00
Reinforcing Steel	769,70
Prestressing steel	3,40

4.4. Conclusion (GBF)

The investigation in this section has shown that a Gravity Base Foundation is also technically feasible for OWEC of the 15MW+ class in larger water depths of up to 40 m.

The structure itself can be further optimized during a later stage of the project. For example, a combined foundation structure consisting of a gravity base and bucket foundation can save even more material. Possibly the base plate can be lowered into the seabed and included into the scour protection. In addition, with increasingly detailed verification, material can be saved in certain parts.

The prefabrication and floating of the structure make it possible to prefabricate secondary components and connect them to the GBF without the need for additional installation steps. A boat landing and the main access platform could be manufactured as reinforced concrete, in hybrid concrete-steel construction or with new composite building materials.

5. Floating conceptual design (PPI)

5.1. Main goals and steps

The main goals and steps of the floating conceptual design were to:

1. Issue a design basis for the floating concept study based on:
    - GE preliminary main tower properties designed for floating conditions,
    - GE next generation 15+MW offshore wind turbine for Site 5 environment in 75m water depth
    - Principle Power WindFloat® technology considering a relatively shallow 10.9m WTG integration draft (12m WD LAT)
2. Perform a preliminary design of a conventional field proven WindFloat Tubular platform concept and a typical 6 mooring lines catenary mooring system fitted with GE next generation 15+MW offshore wind turbine and tower.
3. Benchmark the two simulation tools HAWC2 from DTU and Orcaflex from PPI on the same floating concept:
    - PPI benchmarked the GE tower and WTG characteristics provided by DTU
    - DTU benchmarked the platform and mooring characteristics provided by PPI
    - Assess and compare preliminary loads (ULS, FLS) on GE tower/WTG using this floating concept (PPI versus DTU)

5.2. Floater design

5.2.1. Hull

The hull sized for this study is PPI's conventional WindFloat-T, already in operation in Portugal (WindFloat Atlantic offshore wind farm) and Scotland (Kincardine offshore wind farm) since a few years. It consists in a three-column semisubmersible platform designed to support a wind turbine (Figure 21: A generic conventional WindFloat-T with a wind turbine). The wind turbine nacelle yaws in response to changes in wind direction. To compensate for wind direction changes which would otherwise result in platform pitch, an active ballast system transfers water between column tanks thus maintaining optimum performance.

Figure 21: A generic conventional WindFloat-T with a wind turbine

5.2.2. Mooring system

The mooring system sized for this study is a catenary mooring arrangement, where six mooring lines are used in three clusters (Figure 22).

The mooring system uses studless chain along with polyester rope connected to a two degrees of rotation platform mooring connector attached to the hull.

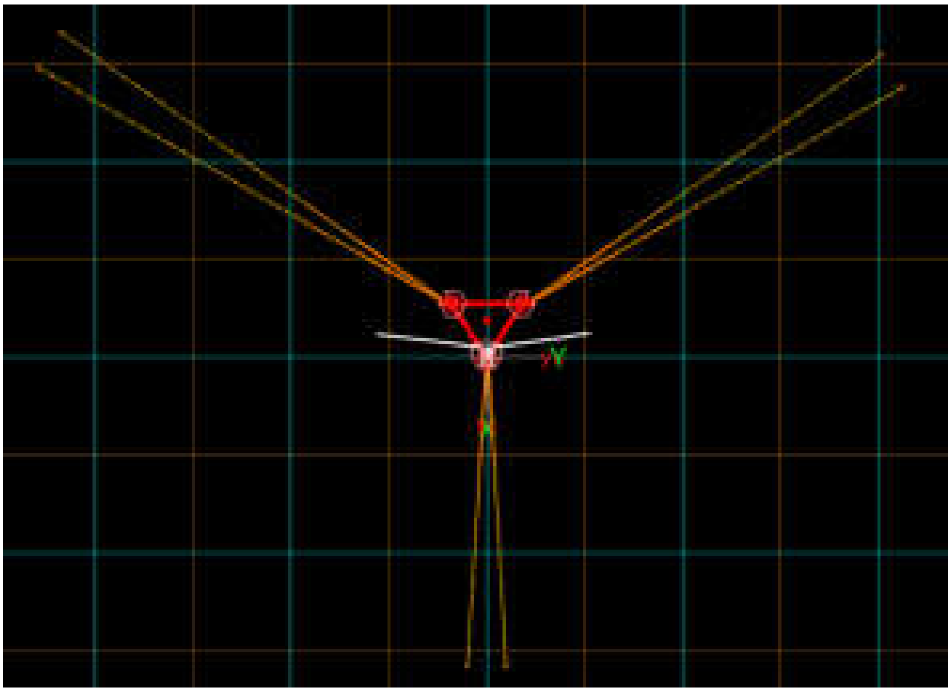


Figure 22: Catenary mooring system

5.3. Benchmark

5.3.1. Introduction

PPI, DTU and GE use different software packages for the WTG aero-elastic load simulations:

- PPI: applies OrcaFlex by using its hydrodynamic capabilities together with a built-in aerodynamic turbine model.
- DTU: applies HAWC2 - an aeroelastic code extensively validated, intended for calculating wind turbine response in time domain.
- GE: applies Bladed and HAWC2, but for this Benchmark only the input files were provided.

For the benchmark exercise with DTU, the aerodynamic verification is, therefore, to demonstrate that the OrcaFlex model sufficiently represents GE next generation 15+MW offshore wind turbine modelled in HAWC2, by obtaining reasonably equivalent aerodynamic behaviour under the same environmental. The same applies for the hydrodynamic verification.

The benchmark has been performed in the following extreme environment for Site 5:

- DLC 1.6a: Max Thrust (11 m/s) + SSS (Hs=7.35m, Tp=15.55sec, Cu=0.3 m/s)
- DLC 1.6b: Max Thrust (28 m/s) + SSS (Hs=12.7m, Tp=17.7sec, Cu=0.3 m/s)
- DLC 5.1: Max Thrust (11 m/s) + NSS (Hs=2.4m, Tp=11.45sec, Cu=0.3m/s)
- DLC 6.1: 50yr wind (35 m/s) + ESS (Hs=12.8m, Tp=16.3sec, Cu=0.4 m/s)

Table 17: Design Load Cases (DLCs) Definition

DLC	Turbine Status	System Conditions	Wind	Wave	Current
1.6a	Production at max thrust	Intact	Normal turbulence	Severe (50-y RP)	Normal
1.6b	Production at cut out	Intact	Normal turbulence	Severe (50-y RP)	Normal
5.1	Emergency shut down at max thrust	Intact	Normal	Normal	Normal
6.1	Parked	Intact	Extreme (50-y RP)	Extreme (50-y RP)	Extreme (50-y RP)

5.3.2. Main results

The impact of the floating-tuned controller has been investigated by DTU. In this model setup, the aeroelastic model has been simulated in HAWC2 on the floating configuration, where the load impact of the baseline controller (tuned for fixed bottom) and the floating-tuned controller (tuning for lower closed-loop system pole frequency and additional pitch controller feedback based on tower-top velocity) is compared in the DLC cases presented in the above mentioned DTU-PPI benchmark comparisons.
The impact of the floating-tuned controller on the ultimate loads of the tower bottom interface resultant moment is shown in Figure 23, where the ultimate load level for every simulated case is presented. The floating-tuned controller has a small impact on the operational cases (DLC 1.2 and DLC 1.6), but since the maximum load is affected by the extreme response at DLC1.6b (severe sea state at cut-out), the overall impact is not noticeable.
In Figure 24 the impact of the floating-tuned controller on the short-term equivalent fatigue loads of the tower bottom interface resultant moment is shown, where the impact of the controller tuning is visible on the cases with negative damping of the pitch motion at high wind speeds. Again, the overall effect on the total lifetime fatigue load is small, since the short-term equivalent fatigue loads are weighted by the probability of occurrence in the lifetime, which is dominated by lower wind speeds.
A comparison of PPI loads from OrcaFlex and DTU loads from HAWC2 is shown in Figure 25.

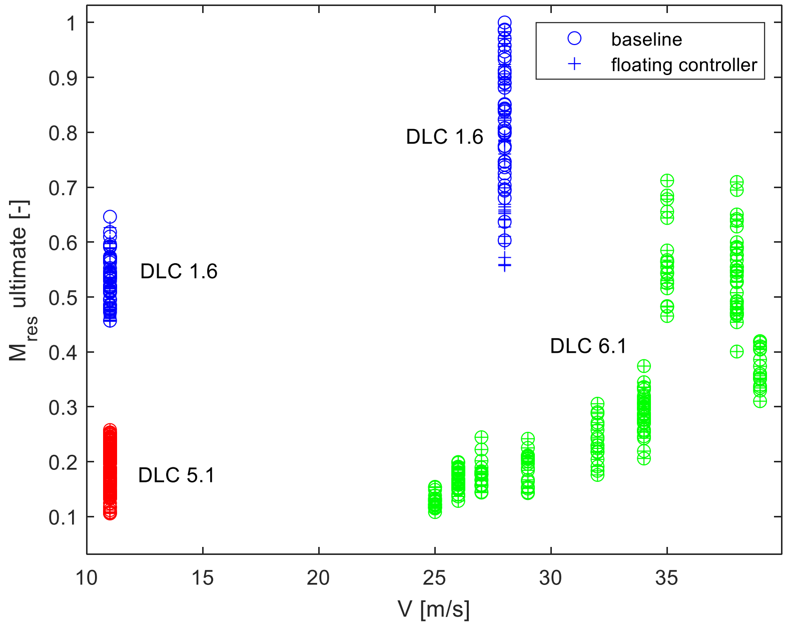


Figure 23 : Ultimate resultant reaction bending moment on the tower interface as a function of wind speed and DLC. Comparison of baseline with floating-tuned controller. Non-dimensionalized by maximum value.

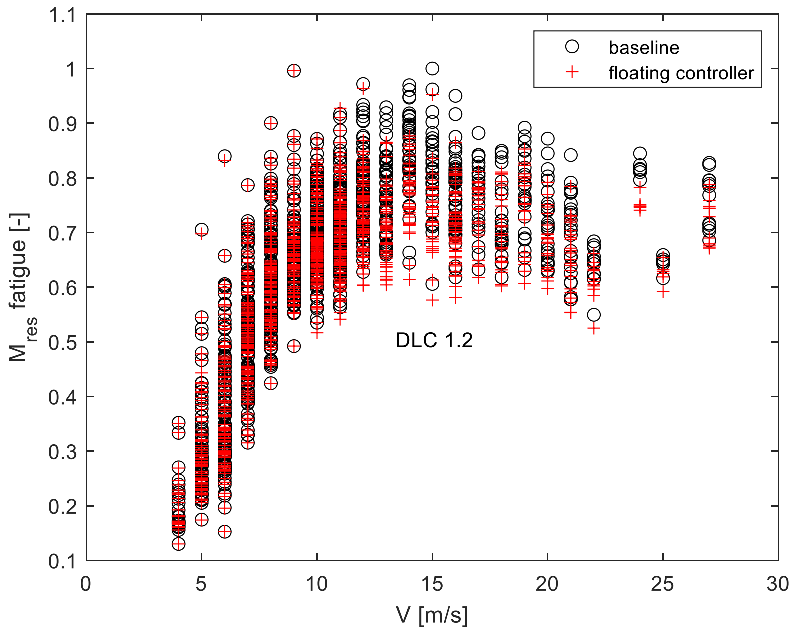


Figure 24 : Lifetime fatigue resultant reaction bending moment on the tower interface as a function of wind speed. Comparison of baseline with floating-tuned controller. Non-dimensionalized by maximum value.

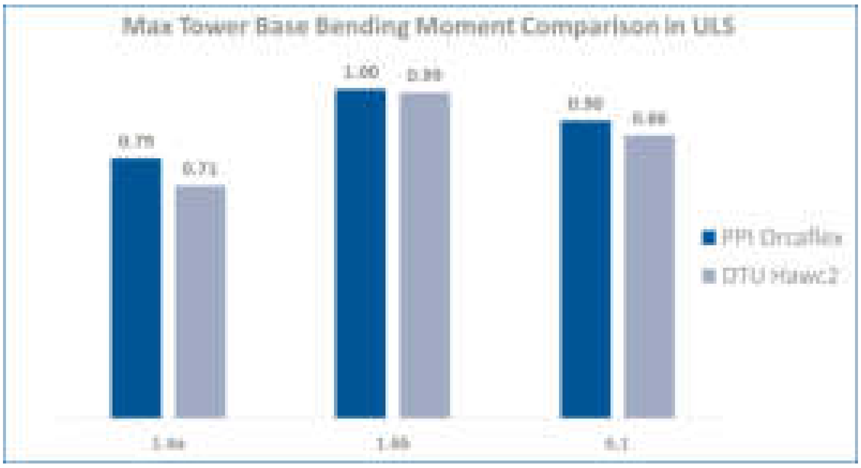


Figure 25: Max Tower Base Bending Moment Comparison in ULS

5.4. Conclusion on floating conceptual design

At this early design stage of the floating conceptual design the general geometry of the floater has been chosen to be a semi-submerged structure. Furthermore, the turbine controller has been adapted for a floating turbine based on the bottom fixed controller with limited impact to the overall ULS load level. Regarding FLS some load reduction could be shown.

Regarding a benchmark comparison between OrcaFlex and HAWC2 it has been shown that both codes lead to comparable results for ULS situation.

6. Comparison of Monopile, Jacket and GBF design

The idea of the investigations of foundation designs for large wind turbines within the ReaLCoE project is to get an impression of the suitability and comparability of several foundation types.

6.1. Comparison of bottom fixed foundations for Site 1

In a first step the three bottom fixed foundations investigated at Site 1 on a conceptual design stage (which are Monopile, Jacket and Gravity Base Foundation (GBF)) are compared to each other regarding weight / material and eigenfrequencies. All conceptual design presented here are based on 15+MW wind turbine. If more than one conceptual design is available (like for monopile and jacket) the best variant w.r.t. the structural weight has been used for the comparison.

Site 1	**Monopile**	**Jacket**	**GBF**
Water depth	40 m		
Structural specification	Diameter: 9.5 m	Foot print: 23.25 m	Concrete conus
Total substructure weight *) (without secondary components)	1,774 t	1,987 t	Concrete: 9,630 t Steel:773 t Ballast:16,000 t
Embedded length	31.5 m	37.7 m	0 m
Lowest eigenfrequency	0.153 Hz	0.189 Hz	0.196 Hz

*) All mentioned masses are stated without any contingency. This means that due to several uncertainties the weight of the final design might change.

It need to be mentioned, that further investigation on monopile, jacket and a monobucket had been performed with a Senvion turbine at an earlier project stage.

6.2. Comparison of monopile and jacket depending on water depth

The foundation types monopile and jacket have also been investigated for Site 3 with 60m water depth. The results are shown in the same manner

Site 3	Monopile	Jacket
Water depth	60 m	
Structural specification	Diameter: 11.0 m	Foot print: 30 m
Total substructure weight *) (without secondary components)	2,785 t	2.141 t
Embedded length	36.0 m	39.2 m
Lowest eigenfrequency	0.149 Hz	0.194 Hz

*) All mentioned masses are stated without any contingency. This means that due to several uncertainties the weight of the final design might change.

Comparing the situation from Site 1 (40 m) and Site 3 (0 m) it can be seen, that the increase of weight is significant for the monopile whereas the jacket weight is not so much affected by the water depth. It can be seen from Figure 26, that the break-even point where the jacket becomes more beneficial w.r.t. weights lies around 48 m water depth. This does not consider the fabrication cost and transport and installation cost and maintenance cost. These aspects need to be further investigated and will most probably shift the break-even point more to deeper locations. It needs to be mentioned, that the linearity is limited to a certain range of water depth.

Figure 26: Monopile and jacket designs at different water depths

7. Conclusion and outlook

This final report on conceptual substructure designs provides initial and optimised designs for bottom fixed foundation types Monopile (Chapter 2), Jacket (Chapter 3) and Gravity Base Foundation (Chapter 4). Investigations are made with a next generation 15+MW offshore wind turbine for Site 1 (35 to 40m water depth) and Site 3 (60m water depth). Investigations on a conceptual floating foundation are documented in Chapter 5. A comparison of the investigated foundation designs is provided in Chapter 6.

The monopile design becomes more and more challenging with increasing water depth. Due to lowered tower eigenfrequencies the overall structure comes close to the highly exited area of the wave spectrum causing higher fatigue loads. It is important to ensure a sufficient damping by additional tower damping and high availability (aeroelastic damping). By selecting interface levels depending on mass limitations per section the total number of sections of the support structure (MP, TP, tower), can be specified with three at 40m water depth up to six at 70m water depth.

Regarding the Jacket design it could be demonstrated that the footprint need to be chosen with care. A smaller footprint leads to a reduced jacket weight but to an increased pile weight, which has significant contribution to the overall substructure mass. Taking this into account there is only a minor impact on the weight going from 40m to 60m water depth.

As a main result the comparison of monopile and jacket design for Site 1 and 3 a break-even point with jacket as the preferable solution has been determined at about 48 m water depths considering the design weight only. This need to be further investigated considering fabrication and installation cost.

Furthermore, a gravity base foundation (GBF) has been designed. Although a concrete concept with ballasted conus has been chosen, a high amount of steel is needed. Further design optimisation might be needed to show advantages of this solution.

DTU and PPI have performed detailed site-specific load simulations of the turbine and floater at Site 5 and benchmarking of load predictions with two aero-hydro-servo-elastic tools, HAWC2 and OrcaFlex. The sensitivity of the design loads to the turbine controller tuning has also been investigated.

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- this is an update of Appendix A in [R1] considering additional data after finalizing [R1] -

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