



# JBO

## Engineering Group

JBO ref. number      **25A514-JBO-1001-DRCD-04**  
Empl. ref. number      -

Revision:      **04**  
Status:      **Issued For Publishing**

**C1 Connections BV**

C1 CONNECTIONS

**Monopile Design Study**

**Design Report**

*Whitepaper: Design Case Study for 22 MW  
Turbine Type*

Rev. JBO	Rev. Employer	Date	Description	Author	QC	Approver
04	04	12.11.2025	Issued for Publishing	FOs, BSt, BCe, PPa	FOs	FOs

## TABLE OF CONTENT

<b>1</b>	<b>Introduction .....</b>	<b>4</b>
1.1	General.....	4
1.2	Document Purpose.....	4
1.3	Abbreviations & Definitions .....	5
<b>2</b>	<b>References .....</b>	<b>6</b>
2.1	Technical Guidelines and Standards .....	6
2.2	Literature .....	6
2.3	Software .....	6
<b>3</b>	<b>Motivation .....</b>	<b>7</b>
3.1	Monopile Challenges.....	7
3.2	Load Mitigation Measures .....	7
3.3	Structural Measures .....	8
3.4	Study Outline.....	8
<b>4</b>	<b>Case Study Boundary Conditions .....</b>	<b>9</b>
4.1	Site Conditions .....	10
4.2	Characteristic Values of the WTG .....	12
4.3	Structural Layout of Monopile and Transition Piece .....	12
4.4	Applied Design Principles.....	14
4.4.1	Structural steel verifications .....	14
4.4.2	Geotechnical design approach .....	14
4.5	Further Study Design Assumptions.....	15
<b>5</b>	<b>FLS Parametric Study .....</b>	<b>16</b>
5.1	Load Analysis Approach - FLS.....	16
5.2	Modal Properties and FLS Loads.....	16
5.3	Structural FLS Design Results .....	18
<b>6</b>	<b>ULS Parametric Study.....</b>	<b>19</b>
6.1	Load Analysis Approach - ULS .....	19
6.2	Design Extreme Load Level .....	19
6.3	Geotechnical Design Results .....	21
6.4	Structural ULS Design Results.....	22
<b>7</b>	<b>Design Comparison .....</b>	<b>23</b>

---

<b>8 Conclusion</b> .....	<b>24</b>
<b>List of Figures</b> .....	<b>25</b>
<b>List of Tables</b> .....	<b>26</b>
<b>Appendix A: Summary of Design Results</b> .....	<b>27</b>

## 1 INTRODUCTION

### 1.1 General

For demonstration purposes, C1 Connections BV (hereinafter referred to as C1) commissioned a comparative monopile design study for a 22 MW wind turbine generator in a water depth of 45 meters, under typical North Sea site conditions. The whitepaper aims to assess the impact of varying interface diameters between the monopile and the tower on the weight of the support structure. Specifically, the analysis will examine the effects of diameter variations in the wave-loaded zone on fatigue limit state (FLS) and ultimate limit state (ULS) load levels, demonstrating their influence on potential interface connections (MP-tower and MP/TP).

### 1.2 Document Purpose

This document aims to offer a comprehensive understanding of how Ultimate Limit State (ULS) and Fatigue Limit State (FLS) loads on an offshore wind WTG support structure are influenced by the interface diameter between the tower and monopile, as well as its impact on the structural steel weight.

### 1.3 Abbreviations & Definitions

<b>Abbreviations</b>	<b>Definition</b>
DEL	Damage Equivalent Load
DLC	Design Load Case
EBD	Embedment Depth
EF	Eigenfrequency
FLS	Fatigue Limit State
ICCP	Impressed Current Cathodic Protection
IAC	Inter Array Cable
LCOE	Levelized Cost of Energy
MP	Monopile
OD	Outer Diameter
PSF	Partial Safety Factor
RP	Return Period
TP	Transition Piece
ULS	Ultimate Limit State
WT	Wall Thickness
WTG	Wind Turbine Generator

## 2 REFERENCES

### 2.1 Technical Guidelines and Standards

- [N1] IEC61400-3-1: Wind energy generation systems – Part 3-1: Design requirements for fixed offshore wind turbines, Det Norske Veritas, 2019-04
- [N2] DIN 18088-1: „Structures for wind turbines and platforms - Part 1: Basic principles and actions”; 2019-01
- [N3] DNV-ST-0126: “Support structures for wind turbines”; Edition 2018-07 - Amended 2021-09
- [N4] DNV-RP-C203: “Fatigue design of offshore steel structures”; Edition 2024-10
- [N5] EN 1993-1-6:2010: „Eurocode 3 – Design of steel structures - Part 1-6: strength and stability of shell structures”; EN1993-1-6:2007 + AC:2009
- [N6] EN 1993-1-9:2010: Eurocode 3 – Design of steel structures - Part 1-9: Fatigue”; EN1993-1-9:2005 + AC:2009

### 2.2 Literature

- [L1] Passon: Offshore Wind Turbine Foundation Design, DTU Wind Energy PhD, No. 0044(EN), DTU Wind Energy, 2015
- [L2] IEA 22 MW turbine DNV Bladed model, part of DNV Bladed installation package. Based on IEA Task 55 report <https://orbit.dtu.dk/en/publications/definition-of-the-iea-wind-22-megawatt-offshore-reference-wind-tu>
- [L3] Thieken, K., Achmus, M., Lemke, K.: A new static p-y approach for pile with arbitrary dimensions in sand, Geotechnik 38,4, 2015.
- [L4] Terceros, M.A. (2021): “A new p-y Approach to Pile Foundations with Arbitrary Dimensions under Monotonic Load in Cohesive Soils”, Phd thesis, Leibniz University Hannover, Germany.
- [L5] EA-Pfähle: Empfehlungen des Arbeitskreises Pfähle der Deutschen Gesellschaft für Geotechnik, Verlag Ernst & Sohn, 2. Auflage, 2012.

### 2.3 Software

- [S1] DNV Bladed v4.17

### 3 MOTIVATION

The offshore wind industry is rapidly developing and pushes the required turbine sizes and support structure solutions to its feasibility limits. Furthermore, wind farms are planned with a larger distance to the shore in deeper and more complex site conditions. Detailed design solutions in deeper waters (>40 m) with turbines in the 15 MW class have already been conducted and installed in the North- and Baltic Sea. Currently design processes for even larger turbines are in detailed design stages (18 MW to 22 MW). Different types of substructure solutions were built and investigated in the past, here beside the monopile – jacket structures and gravity base foundation were the most promising variants.

#### 3.1 Monopile Challenges

Nonetheless, the monopile continues to be regarded as the most cost-effective foundation solution for future projects with water depths reaching up to approximately 60 meters. Efforts are being made to design soft and slender structures that minimize the weight of support structures and utilize existing installation equipment. Both of these factors will contribute to a reduction in the Levelized Cost of Energy (LCOE).

A primary disadvantage of these soft structures is their susceptibility to dynamic excitation and resonance in prevalent sea states. The first natural frequency of the system generally falls within the range of peak wave periods, and as a side effect low secondary global natural frequencies are more vulnerable to seismic or sea-ice excitation. Such resonance effects lead to additional fatigue damage in the support structures, thereby diminishing the technical feasibility of monopiles for future configurations. Beside the fatigue challenge also extreme loads resulting from unfavourable resonance sea states tend to exceed the typical design dominating effects from the 50-year storm conditions incl. the design wave.

#### 3.2 Load Mitigation Measures

There are several ways identified to mitigate these load effects. The two most effective approaches, apart from adjustments of the turbine and controller, are:

- a) **Application of a support structure damper device:** A support structure damper is typically an active or passive tuned mass damper or a tuned liquid column damper, placed in the top of the structure. The effect is an energy suppression and consequently the reduction of the design driving FLS and ULS resonance loading
- b) **Smart support structure design:** The objective of foundation design is to minimize the outer diameter in the wave-loaded zone and to reduce mode shape deflections in the wave impact zone. These measures help reduce Fatigue Limit State (FLS) as well as Ultimate Limit State (ULS) loading. This can be explained by Morison's equation, which is used to calculate viscous drag and inertia loading on tubular members in design practice. The drag term in this equation increases linearly with the loaded diameter, while the inertia term shows a quadratic increase. As all resonance situations are dominated by hydrodynamic inertia contribution, a larger diameter in the wave-loaded zone typically results into disproportionate ULS and FLS load increases.

### 3.3 Structural Measures

Since the load mitigation measures are limited, structural adaptations are required to transfer the increasing support structure loading from the rotor-nacelle assembly down to the soil.

- a) **Wall thickness adaptation:** Since the increase of the support structure diameter is not always beneficial, costly steel wall thickness need to be added to the tower and support structure. On the geotechnical design, pile diameter increase, and higher pile embedment length are suited for mitigation.
- b) **Interface connection:** A weak point remains in the structure: interface connections between tower and monopile, and potentially between a transition piece and the monopile. Typically, bolted L-flange connections are applied in monopile configurations to connect the tower to the substructure, and also to connect the TP to the monopile. For currently investigated 15 MW+ turbines the capacity of the bolted L-flange is near to its limits or exceeded. Geometric constraints and bolt sizes are the main limiting factors. Here alternative interface connection methods are required, such as the special high-capacity C1 Wedge Connection™.

### 3.4 Study Outline

The aforementioned effects were thoroughly examined in the subsequent study. Utilizing a typical North Sea location and a 22 MW turbine configuration on a monopile foundation, various diameters and structural solutions within the wave-loaded zone were assessed. Both Fatigue Limit State (FLS) and Ultimate Limit State (ULS) situations were analysed for this purpose. Subsequently, relative load increases and relative weight increases compared to the base case scenario are presented.

## 4 CASE STUDY BOUNDARY CONDITIONS

This section summarizes the characteristic boundary conditions, e.g. metocean data, but also turbine data and structural considerations.

Given the necessity for the study to be representative, a typical location within the North Sea was chosen, where several gigawatts of offshore wind capacity are anticipated in the forthcoming years. Additionally, a potential turbine configuration was identified to simulate the expected rated power and dimensions of future installations.

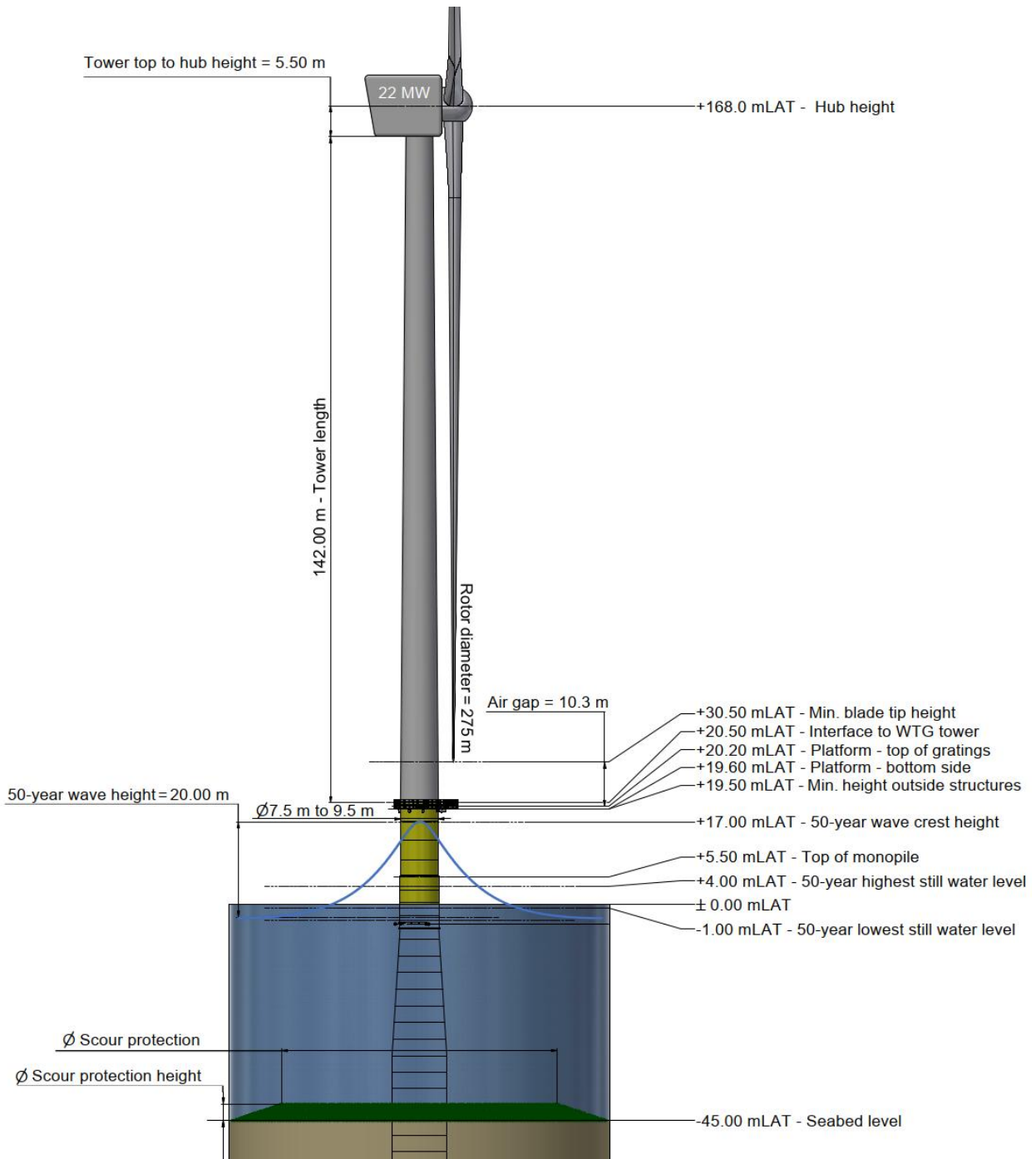


Figure 4-1: Case study 22 MW turbine configuration on monopile

### 4.1 Site Conditions

Site conditions for a typical German North Sea site in 45 m water depth are chosen and listed in the following table.

Table 4-1: Metocean conditions considered for this study

Parameter	Abbreviation	Unit	Value
Water depth	WD	[mLAT]	-45.0
Mean Sea Level	MSL	[mLAT]	1.2
Highest Still Water Level (50-year RP)	HSWL <sub>50</sub>	[mLAT]	4.0
Water density	$\rho_{water}$	[kg/m <sup>3</sup> ]	1025
Significant wave height (50-year RP)	H <sub>s</sub>	[m]	10.5
Peak period (50-year RP)	T <sub>P</sub>	[s]	14.4
Extreme wave height (50-year RP)	H <sub>max50</sub>	[m]	20.0
Wave period (ass. to H <sub>max50</sub> )	T <sub>ass, 50y</sub>	[s]	11.7 to 16.0
Current speed depth-averaged (50-year)	CS <sub>DA50</sub>	[m/s]	1.15
Marine growth	MG	[-]	acc. to DIN 18088 [N2]
Wind speed (50-year RP) at 168 mLAT	V <sub>50</sub>	[m/s]	46
Wind speed average at 168 mLAT	V <sub>avg</sub>	[m/s]	10.5
Wind shear exponent	$\alpha$	[-]	0.1
Air density extreme wind speed	$\rho_{wind,ext}$	[kg/m <sup>3</sup> ]	1.21
Air density normal wind speed	$\rho_{wind,norm}$	[kg/m <sup>3</sup> ]	1.23

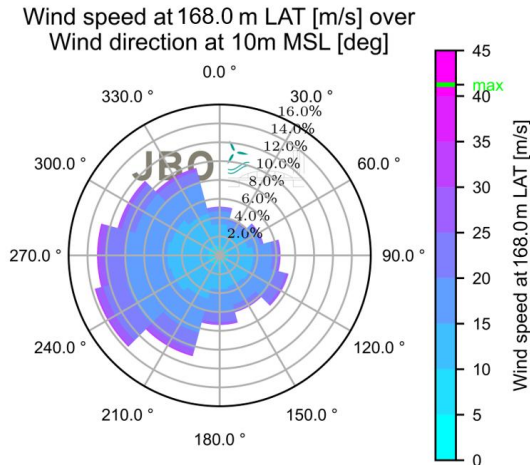


Figure 4-2: Wind rose at hub height

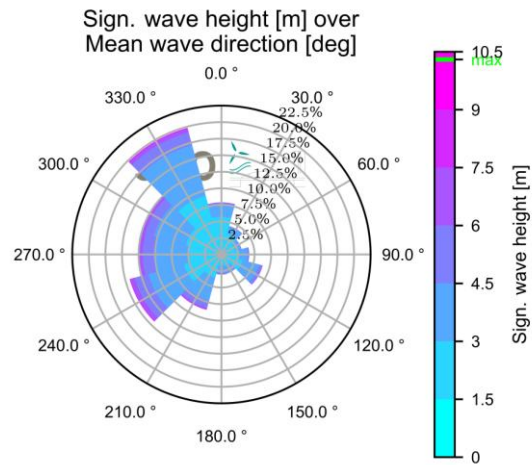


Figure 4-3: Wave rose (total sea)

Based on the assumed metocean data set parameter for the normal and extreme conditions sea states were derived. The normal sea state parameter were condensed from the full data set to wind speed dependent sea states. The method applied considers the structural dynamics and ensures the correct fatigue load level of the respective design variant. In Figure 4-4 and Figure 4-5 the scatter plots for the wind speed at hub height against significant wave height, and the peak wave period over significant wave height are shown.

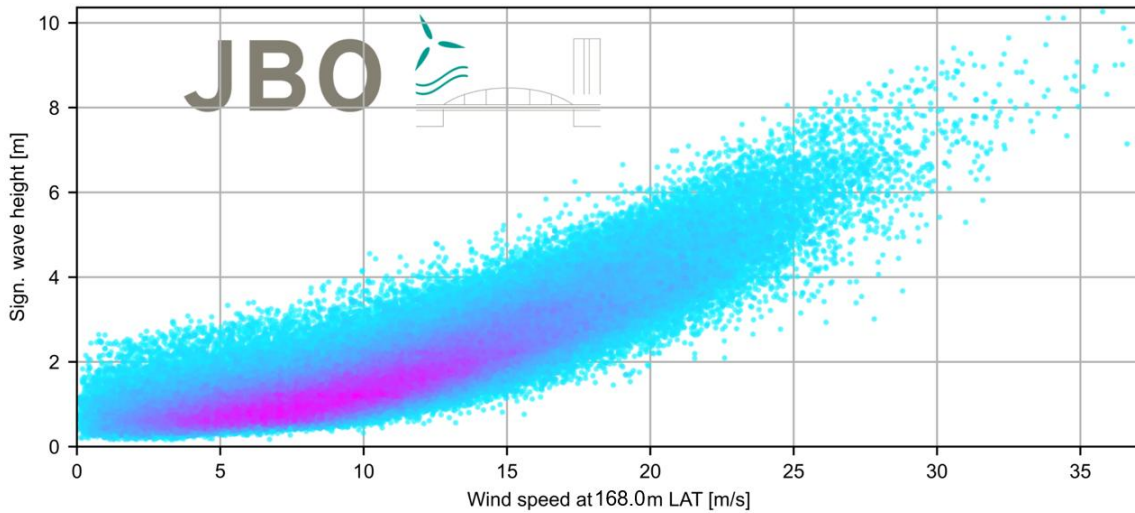


Figure 4-4: Significant wave height over wind speed at hub height (density plot)

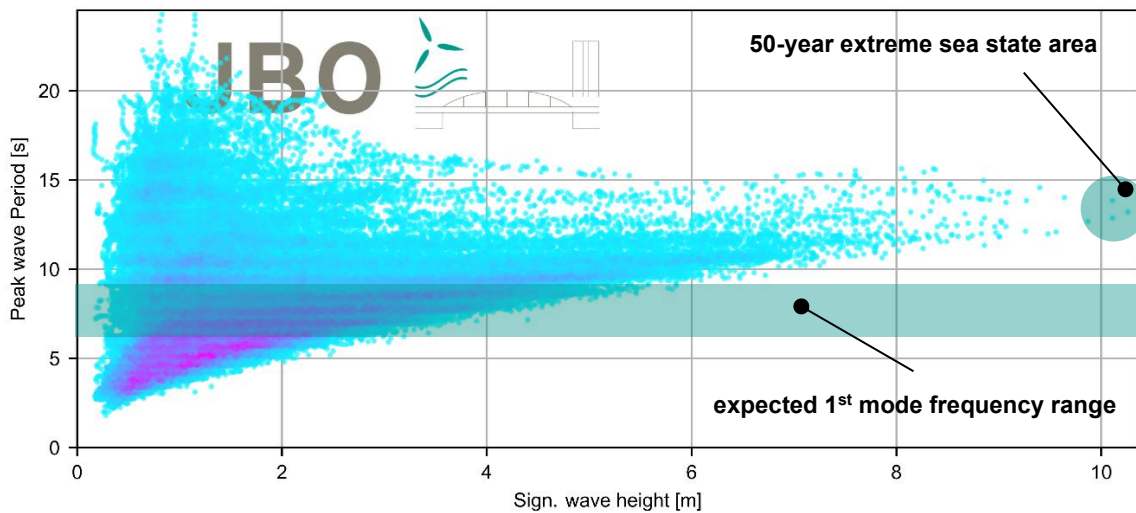


Figure 4-5: Significant wave height over wave peak period (total sea)

Figure 4-5 shows the frequency range of the first structural bending mode and the 50-year return period sea state area. These sea states will cause the highest resonance in the structure as they have either a peak period closest to the decisive natural frequency of the structure or show the highest energy due to highest waves.

Typical North Sea soil conditions are considered for the soil-structure interaction, consisting of mainly sandy soil layers, with best estimate friction angles ranging from 30 to 40 degrees. A scour protection system is assumed to be in place.

## 4.2 Characteristic Values of the WTG

The considered turbine model is following the current market trends with respect to rated power and dimensions. The aero-elastic simulation model is based on the public available 22 MW IEA reference model [L2]. Dimensions and other parameters of the WTG are listed below, please see also Figure 4-1 for further information.

Table 4-2: WTG characteristic values considered for this study

Parameter	Unit	Value
Rated Power	[MW]	22
Rotor Diameter	[m]	275
Hub height	[mLAT]	168.0
Interface height	[mLAT]	20.5
RNA mass	[t]	1300
Equivalent modal damping ratio 1 <sup>st</sup> mode (structural + tower damper)	[%]	1.45
Minimum allowable eigenfrequency	[Hz]	0.123
Tower mass (incl. internals and flanges)	[t]	1600 - 1680 (dependent on base diameter)

## 4.3 Structural Layout of Monopile and Transition Piece

The substructure is represented by a classical monopile - transition piece configuration for this study. Dependent on the investigated variant, the TP is modelled either as cylindrical or conical section. The monopile increases the TP diameter using a conical section to a constant embedded pile diameter of 11.5 m at seabed level. Dependent on the variant, a cone angle of 4 degree to 2 degree was modelled. To keep the hydrodynamic loading minimal, the conical section of the MP starts at -4 mLAT. A skirt structure supporting the access system, ICCP elements and protecting the interface connection MP to TP is considered.

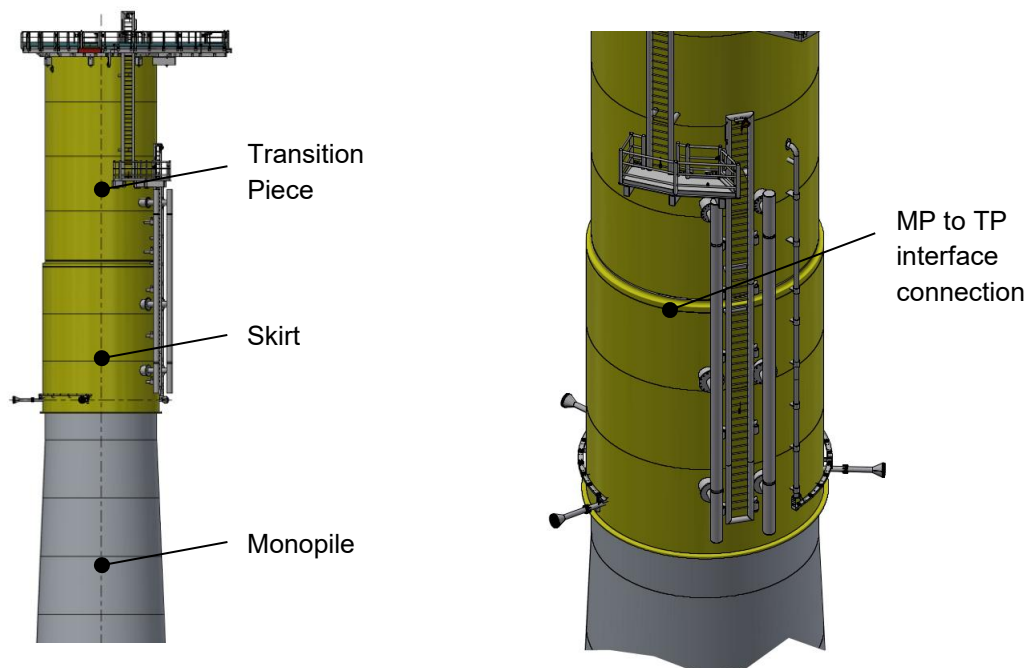


Figure 4-6: Illustration of MP and TP layout incl. skirt and access system

The embedded pile diameter of 11.5 m is not varied to account for the comparability of the investigated configurations. Internals, flanges, platforms etc. have been considered as lumped masses in the model. At the interfaces the assumed flange set weights are constant and based on bolted L-flange models, with

- 80 t at the MP to TP connection, and
- 60 t at the TP to tower interface.

Obviously, these flanges set weights depend on the actual connection type, geometry and loading. However, their mass impact on the global FLS and ULS loads is assessed to be negligible.

In the here presented study, different TP diameters in the wave loaded zone, in combination with structural variants, are investigated. The diameter range has been assumed from the current typical 15 MW size of 7.5 m to a maximum value of 9.5 m. The structural variants represent:

- a) a cylindrical TP, without a skirt (equivalent to TP-less monopile).
- b) a cylindrical TP including a skirt extension, ranging from the MP-TP interface level at 5.5 mLAT to -4.0 mLAT. The skirt has an additional mass of ca. 100 t and increases the outer diameter, relevant for hydrodynamic loads, at the upper MP part by 0.32 m. Detailed skirt weights for each variant can be found in Table 0-2.
- c) a conical section in the TP, starting from 3.2 m below the tower-TP interface. This cone shall increase the flange diameter to increase its typical capacity.

The following table summarizes the investigated variants which are grouped into 3 TP types with 5 sub-groups of different tower bottom diameters each, resulting in a total of 15 variants:

*Table 4-3: Investigated variants: TP with or without cone and with or without skirt combined with matching tower diameter. Stated are the structural outer diameter as well as the outer diameter incl. potential increase due to cone for the wave loads calculation.*

TP/skirt type	Tower base diameter [m]	7.500	8.000	8.500	9.000	9.500
TP: cylindrical Skirt: no	MP/TP structural OD [m]	7.500	8.000	8.500	9.000	9.500
	Hydrodyn. effective OD [m]	7.500	8.000	8.500	9.000	9.500
TP: cylindrical Skirt: yes	MP/TP structural OD [m]	7.500	8.000	8.500	9.000	9.500
	Hydrodyn. effective OD [m]	7.820	8.320	8.820	9.320	9.820
TP: conical Skirt: yes	MP/TP structural OD [m]	8.180	8.680	9.180	9.680	10.180
	Hydrodyn. effective OD [m]	8.500	9.000	9.500	10.000	10.500

For the primary steel design verifications, the presence of the following attachments/holes is assumed:

- **Transition Piece:**
  - Main access platform support
  - Ventilation opening
  - Bolting platform support
  - Junction box platform support
  - ICCP Cable opening
  - Access system support
- **Monopile:**
  - Water replenishment hole
  - IAC hole
  - Access system guide

## 4.4 Applied Design Principles

Design checks are conducted to confirm the structural integrity throughout the lifetime of the substructure for this study. The structural and geotechnical methods are referenced below. The load study approach is referred to in the respective main chapter for ULS and FLS in chapter 5.

### 4.4.1 Structural steel verifications

An overview of the applied standards is provided below.

Table 4-4: Summary of structural steel checks performed in the design study

Steel integrity check	Description	Governing standard
Global yielding primary steel check	Verification of structural capacity against yielding	DNV-ST-0126 [N3] DIN EN 1993-1-6 [N5]
Local shell buckling checks	Verification of structural capacity against local shell buckling	
Fatigue check for circumferential welds	The welds' verification has sufficient capacity to last longer than the specified lifetime.	DNV-RP-C203 [N4] DIN EN 1993-1-9 [N6]
Fatigue check for longitudinal welds		
Fatigue checks of attachments and holes	The verification of holes and welded attachments have sufficient capacity to last longer than the specified design lifetime.	

### 4.4.2 Geotechnical design approach

The soil resistance approaches, and geotechnical design principles are chosen to respect all design requirements for WTG foundations and to provide sufficient safety for this comparative study. Only key assumptions and aspects of the design verifications are outlined in the following.

The **axial pile-soil interaction** will be analysed by means of the standard API main text method, which represents a conservative design approach. Therewith, effects induced by an optional pile shoe are included.

For the **lateral pile-soil interaction** a realistic North Sea design soil profile is considered to feed the analytical p-y models that provides pile-, soil- and depth dependent soil resistance curves. For the conceptual design analyses, the p-y approach according to Thieken et al. [L3] for drained and Terceros [L4] for undrained units is applied.

In addition, the following geotechnical design verifications are considered with respect to the lateral bearing behaviour:

- Ultimate limit state verification for storm event (ULS)
- Pile length criterion for storm event to account for cyclic stability, expected also from numerical analysis, see [L5]
- Serviceability limit state verification (SLS)

#### 4.5 Further Study Design Assumptions

Additional design assumptions not addressed in the previous chapters are documented briefly below.

*Table 4-5: Distribution of lifetime considered for this study*

Parameter	Unit	Value
Design service lifetime	[y]	26
WTG operational time	[y]	25
(De-)commissioning time considering installed tower and RNA	[y]	1
Technical availability	[%]	92
Coating lifetime	[y]	20

It should be noted that the analysis results are based on the following assumptions:

- All foundation configurations are designed with the same number of sections and section heights. Further optimizations could be achieved especially for the lighter configurations by reducing the number of sections.
- A set of commonly required fatigue details, reflecting the secondary steel setup described in 4.3 is applied for all configurations (at equal elevations).
- Grinding of welded attachments (e.g. boat landing bumper & ladder attachments) have been applied for all configurations, when beneficial.
- A fixed monopile bottom diameter and pile penetration for all configurations in the FLS load study is accounted for. Weights documented account for the different pile embedment depth resulting from the ULS load study.
- Fixed elevations for conical transitions in MP (and TP, if any)
- The configuration with a tower base diameter of 8.0 m, cylindrical TP and skirt-less is considered as base case scenario!

In this study, an operational lifetime assumption of 25 years for the WTGs is adopted, despite the fact that detailed design works now consider operational lifetimes of up to 35 years. Nevertheless, the 25-year assumption will provide a conservative estimate for minimum effects on loads and the resulting structural masses. It should be noted that longer operational lifetimes or reduced efficiency (or the absence) of tower dampers would substantially increase fatigue load levels and hence structural masses.

A support structure damper device is accounted for in this study with a simplified representative equivalent modal damping ratio of the 1<sup>st</sup> mode (see also Table 4-2). Since the complexity of damper systems will increase which will, beside the irregularity and non-stationarity in the applied design load cases, make it even more difficult to provide engineering estimates for simplified damper models. Thus, it will get inevitable to have a dedicated damper model in the load model.

## 5 FLS PARAMETRIC STUDY

This section summarizes the process and key results of the FLS load study.

### 5.1 Load Analysis Approach - FLS

FLS loads for the foundation/substructure design are calculated in a superimposed approach cf. Passon [L1] with discrete wind (aerodynamic) loads from the WTG and wave (hydrodynamic) loads being derived in the frequency domain. The FLS load calculation is based on IEC 61400-3-1 [N1] design load cases (DLCs) 1.2, 6.4 and 7.2, covering all wind speeds in the operational range (and beyond) as well as all wind-wave-misalignments in 30° intervals. For DLC7.2 this resulted in a total of 2,700 wave load simulations for the condensed load setup which has been derived from & calibrated against results from a complete hindcast set of 350,000 simulations. Damage equivalent loads (DEL) of the bending moments are evaluated as fatigue reference values.

The probability distribution for the individual simulations per DLC is based on the wind-wave climate, service lifetime, installation & decommissioning period as well as the technical availability provided in chapter 4. Furthermore, the already introduced 22 MW WTG along with a set of experience-based WTG features (controller, equipment, etc.) are considered for the investigations, if considered relevant and realistic for the loads.

### 5.2 Modal Properties and FLS Loads

In the below tables the dynamic key results with respect to first global bending eigenfrequency, normalized FLS design loads and wind load contribution for all 15 configurations.

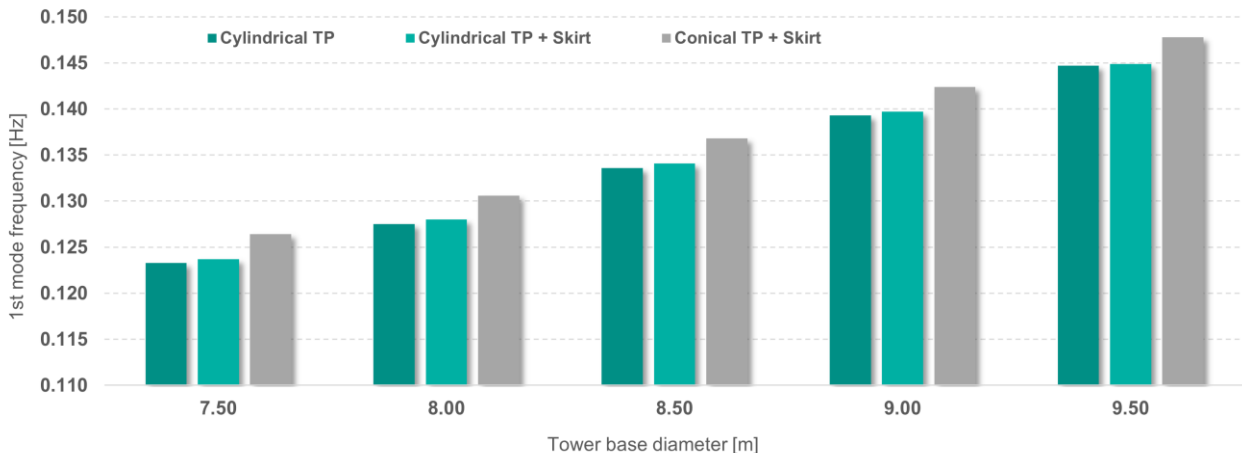


Figure 5-1: Eigenfrequencies without gravity effects for the FLS configuration

The eigenfrequencies in Figure 5-1 are presented without accounting for the influence of gravity. It should be noted that, under the consideration of gravitational effects, the eigenfrequencies for all configurations with a tower bottom diameter of 7500 mm fall below the required limit of 0.123 Hz. Consequently, these configurations are deemed non-feasible unless further adjustments are made to the turbine/controller or the foundation is strengthened, such as by adding a significant amount of additional steel to the foundation.

Although the cylindrical TP and cylindrical TP + skirt variant have similar load-carrying structures, the cylindrical skirt variant has greater stiffness. This is due to the additional steel required to handle higher FLS loads for this variant.

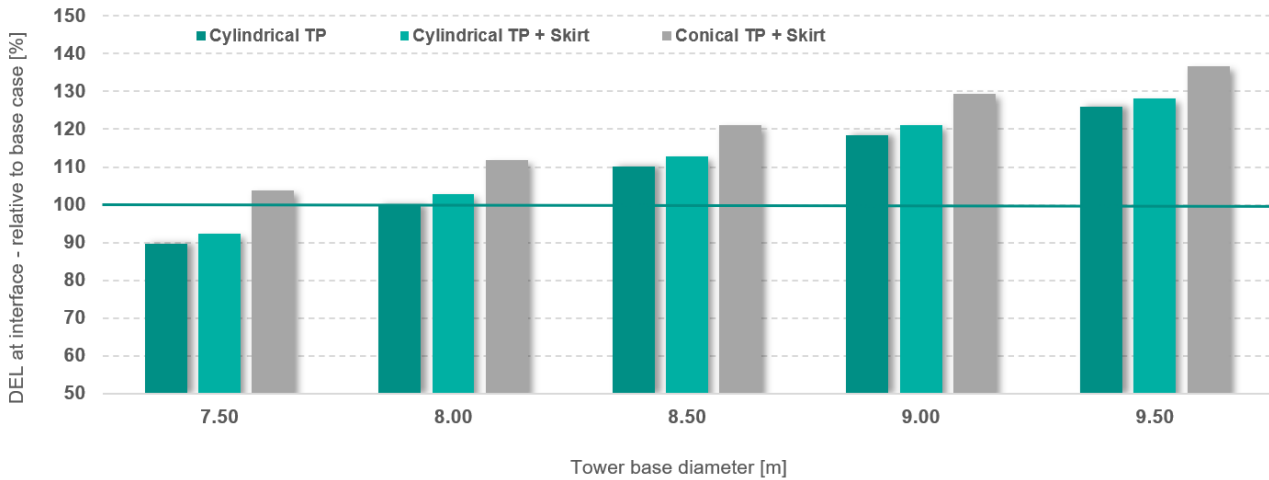


Figure 5-2: Relative DEL bending moment ( $m=5$ ) at MP-TP interface elevation, normalized against base case

The damage equivalent loads illustrated in Figure 5-2 are assessed relative to the baseline scenario of an 8.0 m tower diameter with a cylindrical skirt variant. It is evident that there is a distinct trend related to the diameter as reflected in the representative DELs. A more slender substructure in the wave-loaded zone exhibits a more favourable load profile. In contrast, the skirt variants experience higher fatigue loads due to their hydrodynamic effective presence. The combination of conical transition pieces and skirts results in the most unfavourable FLS loading situation. Based on these findings, it appears uncertain whether the increased resulting diameter and the potential bolted flange capacity will present an economically viable solution.

Furthermore, the wind load contribution has been evaluated. In Figure 5-3 below it is demonstrated that the wind portion of the FLS loading is significantly decreasing with increasing tower base diameter.

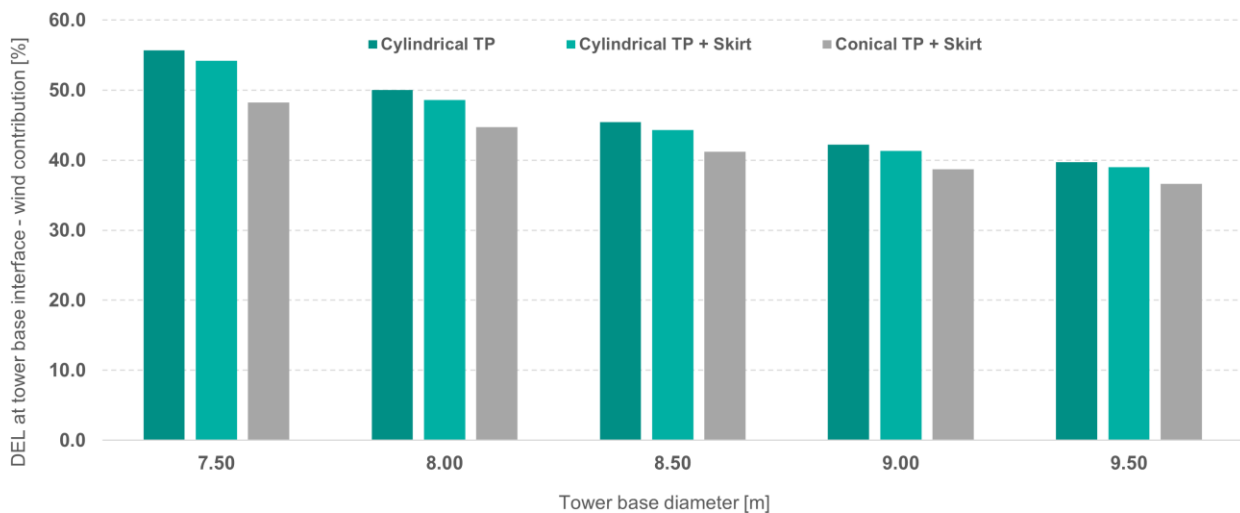


Figure 5-3: Wind contribution to equivalent bending moment ( $m=5$ ) DEL at tower TP interface elevation

### 5.3 Structural FLS Design Results

In the following, structural results in form of weights and maximum wall thicknesses are shared. The damage and D/t ratio are kept at the same level for all 15 variants.

Please note that the weight effect demonstrated in the below graphs is purely due to FLS effects, structural modifications which might result from ULS load increases such as a higher embedded depth of the pile are consequently not considered to isolate the FLS impact.

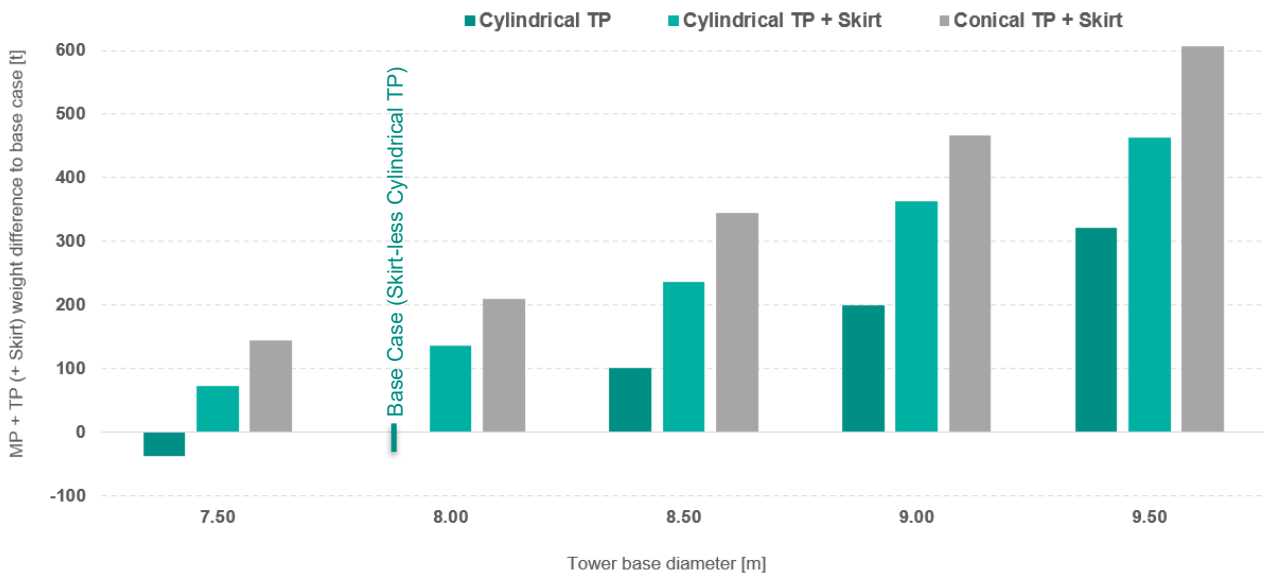


Figure 5-4: Substructure and foundation weight difference caused by FLS load deviations, normalized against base case

The resulting weight differences outlined in Figure 5-4 show the clear trend towards higher masses for increasing tower base diameter.

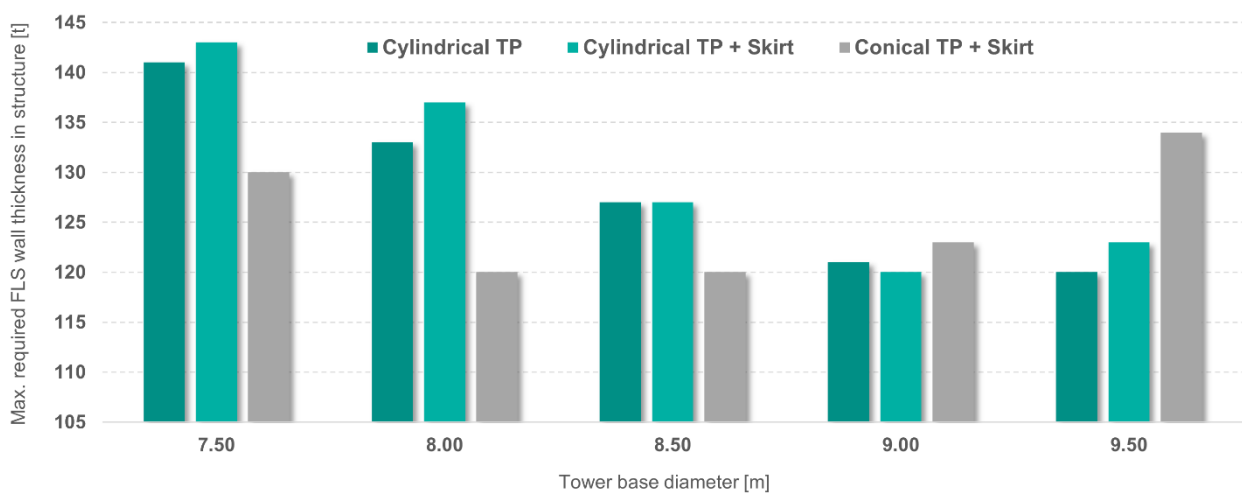


Figure 5-5: Substructure max. wall thickness requirements due to FLS loads

The required FLS wall thicknesses in Figure 5-5 show a significant increase for a 7.5 m OD tower variant, however it shall be noted that this configuration would theoretically even more wall thickness to achieve the stiffness requirements. This is not accounted for in this study, since frequency was not set as a strict design criterion due to the fictive nature of the turbine properties.

## 6 ULS PARAMETRIC STUDY

### 6.1 Load Analysis Approach - ULS

The ULS load calculation is based on IEC 61400-3-1 [N1]. A reduced design load case set is considered, which includes the design dominating DLCs, such as DLC 6.1 (incl. resonance sea state and slamming impact) covering the 50-year storm event, and highest wind- and controller driven operational load case (internally named as DLC 1.99). 54 dynamic time-series with a duration of up to 30 min each were simulated per configuration in time domain using DNV Bladed [S1]. The resulting dynamic response time series were evaluated acc. to the applicable standards, and thus reflect the methodology applied in detailed design projects.

### 6.2 Design Extreme Load Level

The maximum resultant bending moments are evaluated and shown in the following figures for each variant at three characteristic heights: At tower interface 20.5 mLAT, at MP-TP interface 5.5 mLAT and at seabed level -45.0 mLAT.

The load results indicated that the design-relevant bending moments throughout the substructure are primarily influenced by the resonance sea state design conditions. Within this design situation, simulations are performed where

- a) the 50-year significant wave height is combined with the lowest realistic associated peak period, or
- b) a sea state is parametrized which peak period is almost matching the first natural frequency and comes along with the associated max. realistic significant wave height.

Please refer also to the scatter plot illustrated in Figure 4-5 previously.

As illustrated in Figure 6-1 and Figure 6-2, the maximum ULS bending moments remain consistent across the five leftmost variants. Notable increases in load at the tower/TP and MP/TP interfaces are only observed for variants with tower outer diameters of 8.0 m (conical TP) and 8.5 m (cylindrical TP) or higher. This behavior is attributed to the dominance of the operational load case (DLC 1.99) in these lower diameter variants, where hydrodynamic loads are relatively minor in the upper part of the MP and TP. DLC 1.99 is closely followed in magnitude by the resonance load case (DLC6.1r), which becomes decisive at lower elevations. The other variants with higher diameter are completely governed by DLC 6.1r. Consequently, Figure 6-3 illustrates a gradual increase in load at seabed level across all variants, which correlates with the rise in hydrodynamic loads and excitation resulting from larger outer diameters.

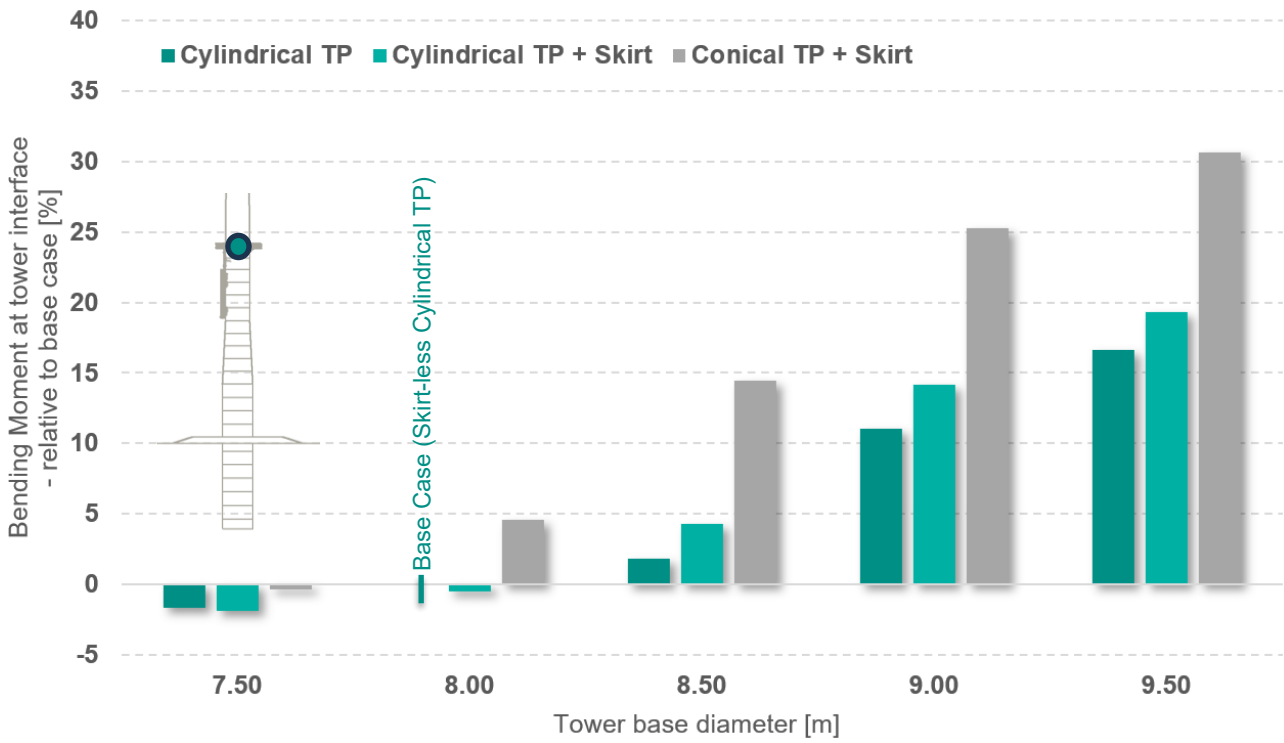


Figure 6-1: Extreme bending moment at tower interface level (incl. PSF and tower out of verticality loads) normalized against base case

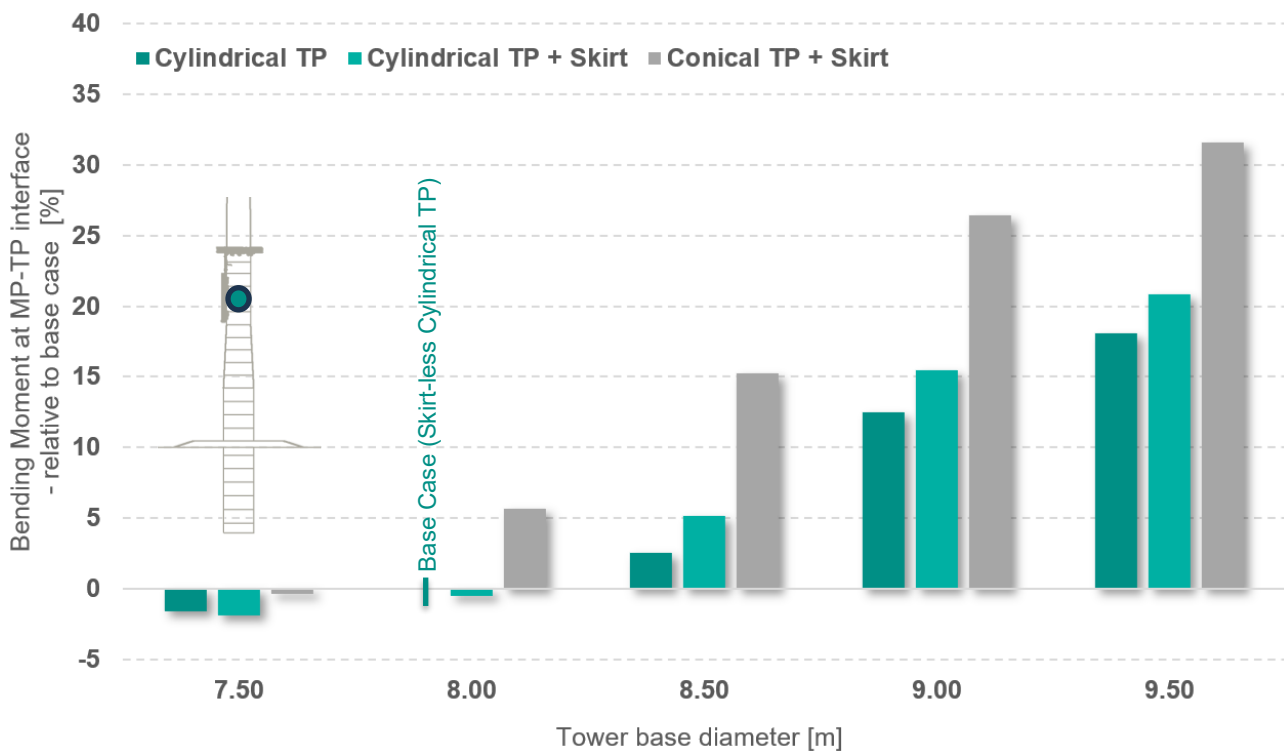


Figure 6-2: Extreme bending moment at MP-TP interface level (incl. PSF and tower out of verticality loads) normalized against base case

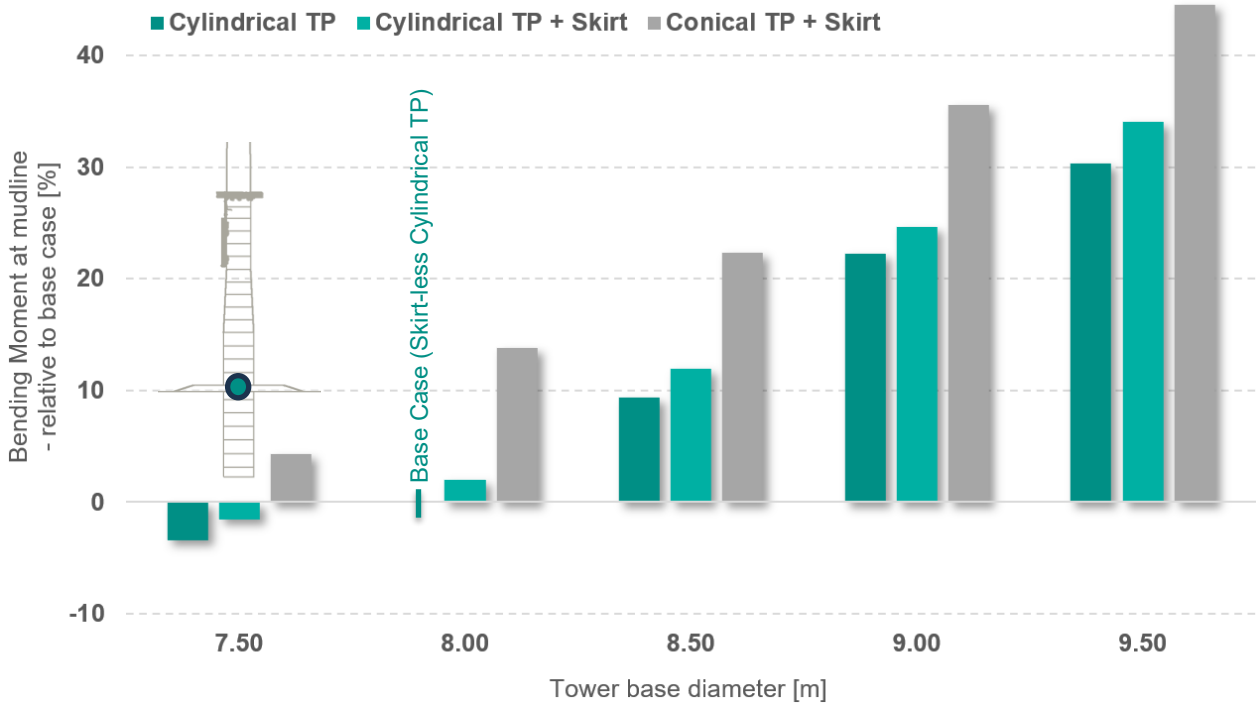


Figure 6-3: Extreme bending moment at seabed level (incl. PSF and tower out of verticality loads) normalized against base case

### 6.3 Geotechnical Design Results

Based on the seabed level extreme load results, the geotechnical design of the pile has been substantiated. The pile length criterion for storm events, introduced in 4.4.2, has shown to be the dominating criteria. Embedded pile length ranging from 32 m towards 37 m were resulting. Please note that this design result is highly dependent on the site-specific soil profile - in this case a representative North-Sea soil profile is assumed.

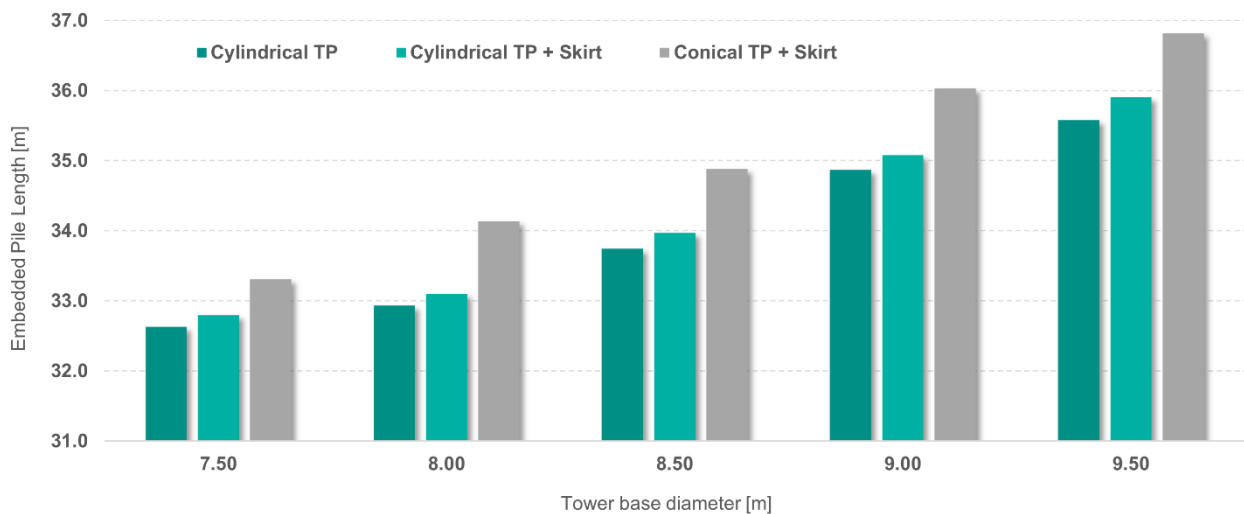


Figure 6-4: Expected pile embedded depth of the 15 variants, based on ULS load update

## 6.4 Structural ULS Design Results

The following section presents structural results in the form of total MP, TP, and skirt (if any) weights. It should be noted that the weight effects illustrated in the graphs below are based on ULS and FLS effects. The structures derived from the FLS parametric study serve as the basis for these results. The structural modifications incorporated, as compared to the FLS results in section 5.3, are justified by the following reasons:

- high ULS buckling utilization, which required increased wall thickness for only few cans and variants.
- the consideration of the analysed monopile load specific embedment depth as shown in section 6.3.

The weight variations illustrated in Figure 6-5 demonstrate a clear trend towards increased mass with larger tower base diameters, as previously identified in the FLS parametric study. In this ULS study, the additional masses are primarily attributed to the ULS load sensitivity of the diameter within the wave-loaded zone. The higher loads subsequently increase the pile embedment depth. Each additional meter of embedded pile contributes at least 25 tonnes to the monopile mass.

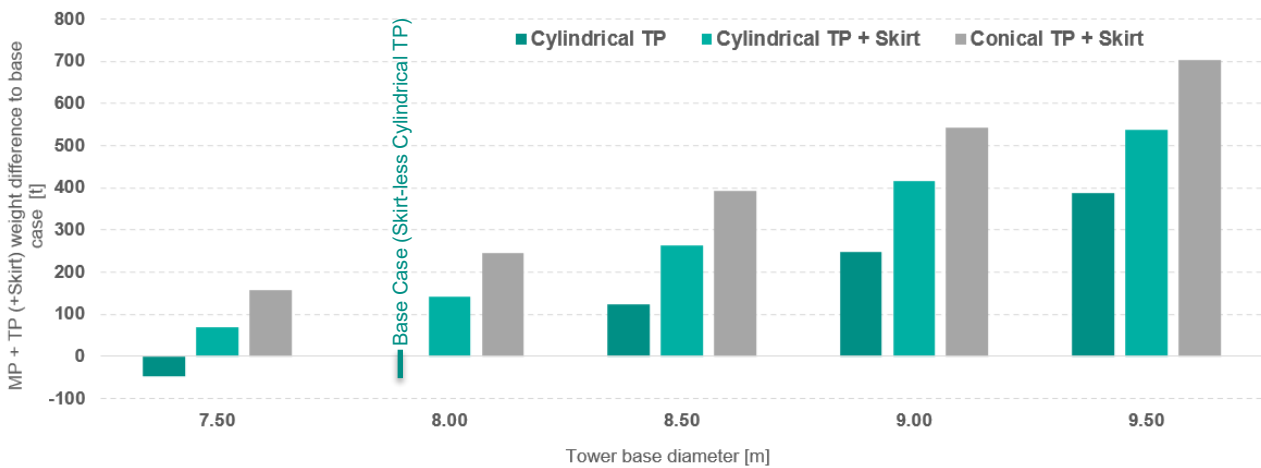


Figure 6-5: Substructure and foundation weight difference caused by ULS and FLS effects, normalized against base case

## 7 DESIGN COMPARISON

In recent years, the design of the MP-TP connection has become increasingly challenging. An often-chosen solution has been to increase tower base and TP diameters, allowing the L-flange to remain designable. However, as turbine sizes continue to grow, this approach is no longer a cost-effective solution. Table 7-1 summarizes key differences between two design scenarios, highlighting the impact of increased tower base and TP diameters on loads, substructure and foundation weights, and pile embedment length.

Table 7-1: Design comparison: impact of increased tower base and TP diameters

	8.0 m OD tower variant + Cylindrical TP	9.0 m OD tower variant + Cylindrical TP + Skirt
MP + TP (+ Skirt) weight <sup>1</sup>	Base Case	+416.0 t
ULS bending moment at MP-TP interface (incl. PSF and tower out of verticality loads)	Base Case	+15.6 %
FLS (DEL with m = 5) bending moment at MP-TP interface	Base Case	+21.1%
Pile embedment length	Base Case	+6.5 %

Hence, the industry trend of simply increasing tower base and TP diameters to extend the feasibility of traditional L-flange connections is no longer an optimal approach. The increased diameters result in substantially higher loads and do not provide sufficient additional capacity. Furthermore, it results in significantly heavier substructures and foundations, leading to considerable cost implications.

In terms of installation challenges, most of the currently available equipment can install tower interfaces up to 8.0 m OD / 8.5 m OD. Thus, larger interfaces would require the industry to adapt its tools and bear its associated costs. Moreover, an increased interface diameter - and the consequent need for increased pile penetration depth due to higher loads - not only increase installation risk and cost, but also accentuates underwater noise during installation, which can be a challenging environmental impact to mitigate.

Therefore, the development of alternative interface connection types, which have the capacity to transfer high loads over limited diameters, becomes a smart engineering solution to address industry demands.

<sup>1</sup> Based on ULS Parametric Study. Results are further detailed in Table 0-2

## 8 CONCLUSION

In section 3 the monopile feasibility challenges with increasing turbine sizes and deeper sites were outlined. One of the mitigation measures was smart engineering by selection of optimized diameters in substructure. The study results clearly confirm the theory that a larger diameter in the wave-loaded zone typically results into disproportionate ULS and FLS load increases for the here found dominating resonance situations. Thus, if the industry intends to solve the monopile feasibility for the future configurations by simply increasing diameters and adding steel to the structures, a significant disproportional negative impact on the LCOE is the result.

**“As a general principle, designing a more slender structure within the wave-loaded zone results in a reduced total mass of the substructure and foundation.”**

Obviously, the slenderness is limited. In the design of support structures for offshore wind typically natural frequencies shall keep a certain distance to the operational rotor frequency – hence the here investigated 7.5 m tower variant is already exceeding the allowed bandwidth and does not represent a feasible design solution – although the total mass is the lowest of the investigated variants. The assumed base case of 8.0 m tower base diameter, designed with a skirt-less cylindrical TP, represents nearly the optimum scenario from the investigated variants.

The study also outlines that the weak point at the interface connections between the tower and TP, and between TP and MP cannot be resolved by increasing the diameters. Introducing conical sections in the TP or simply increasing the tower and TP diameter will not be effective since structural total masses significantly increase, while simultaneously the design decisive extreme bending moments at these elevations also increase disproportionately. This utilizes the potential bolted flange design even further, despite the diameter increases.

The global aim of reducing the levelized cost of energy for offshore wind can be achieved by applying the most cost-effective substructure and foundation solution, which is currently the monopile. The industry needs to address the technical challenges associated with multi-megawatt soft structures. This study examines how optimizing the support structure design can contribute to this goal:

- Smart support structure design - development of slender structures to reduce total steel material usage.
- Development of alternative interface connection types - design connections which have the capability to transfer high loads over limited diameters.
- Application of a support structure damper device – high performance damper devices are required to reduce load resonance effects.

## LIST OF FIGURES

Figure 4-1: Case study 22 MW turbine configuration on monopile .....	9
Figure 4-2: Wind rose at hub height .....	10
Figure 4-3: Wave rose (total sea) .....	10
Figure 4-4: Significant wave height over wind speed at hub height (density plot) .....	11
Figure 4-5: Significant wave height over wave peak period (total sea) .....	11
Figure 4-6: Illustration of MP and TP layout incl. skirt and access system .....	12
Figure 5-1: Eigenfrequencies without gravity effects for the FLS configuration .....	16
Figure 5-2: Relative DEL bending moment (m=5) at MP-TP interface elevation, normalized against base case .....	17
Figure 5-3: Wind contribution to equivalent bending moment (m=5) DEL at tower TP interface elevation .....	17
Figure 5-4: Substructure and foundation weight difference caused by FLS load deviations, normalized against base case .....	18
Figure 5-5: Substructure max. wall thickness requirements due to FLS loads .....	18
Figure 6-1: Extreme bending moment at tower interface level (incl. PSF and tower out of verticality loads) normalized against base case .....	20
Figure 6-2: Extreme bending moment at MP-TP interface level (incl. PSF and tower out of verticality loads) normalized against base case .....	20
Figure 6-3: Extreme bending moment at seabed level (incl. PSF and tower out of verticality loads) normalized against base case .....	21
Figure 6-4: Expected pile embedded depth of the 15 variants, based on ULS load update .....	21
Figure 6-5: Substructure and foundation weight difference caused by ULS and FLS effects, normalized against base case .....	22

## LIST OF TABLES

Table 4-1: Metocean conditions considered for this study .....	10
Table 4-2: WTG characteristic values considered for this study .....	12
Table 4-3: Investigated variants: TP with or without cone and with or without skirt combined with matching tower diameter. Stated are the structural outer diameter as well as the outer diameter incl. potential increase due to cone for the wave loads calculation. ....	13
Table 4-4: Summary of structural steel checks performed in the design study.....	14
Table 4-5: Distribution of lifetime considered for this study.....	15
Table 7-1: Design comparison: impact of increased tower base and TP diameters .....	23
Table 0-1: FLS Parametric Study - Overview .....	27
Table 0-2: ULS Parametric Study - Overview.....	27

## APPENDIX A: SUMMARY OF DESIGN RESULTS

Table 0-1: FLS Parametric Study - Overview

Variant	Tower Bottom [mOD]	Eigen-frequency [Hz]	Weight [t]			Max. design parameter			DEL (normalized)			
			MP+TP	TP	MP	WT [mm]	Damage [%]	D/t ratio [-]	Interface [%]	MP-TP [%]	Mudline [%]	Interface wind load [%]
Cylindrical TP	7.500	0.123	2479	303	2176	141	95.3	129.4	87.3	87.5	87.7	55.7
	8.000	0.128	2517	311	2207	133	95.4	129.2	97.2	97.3	97.4	50.0
	8.500	0.134	2618	323	2294	127	95.9	129.2	107.1	107.1	108.1	45.4
	9.000	0.139	2717	322	2394	121	98.0	129.2	115.0	115.0	117.3	42.2
	9.500	0.145	2838	325	2514	120	97.4	129.2	122.5	122.6	126.3	39.7
Cylindrical TP + Skirt	7.500	0.124	2501	311	2190	143	95.8	129.4	89.7	89.9	89.9	54.2
	8.000	0.128	2558	323	2235	137	98.0	129.2	<b>100.0</b>	<b>100.0</b>	<b>100.0</b>	48.6
	8.500	0.134	2652	329	2322	127	97.3	129.2	109.6	109.6	110.6	44.3
	9.000	0.140	2773	335	2438	120	96.9	129.2	117.7	117.8	120.2	41.3
	9.500	0.145	2867	333	2535	123	97.6	129.2	124.7	124.8	128.8	39.0
Conical TP + Skirt	7.500	0.126	2564	336	2228	130	96.6	129.2	100.9	100.9	100.6	48.2
	8.000	0.131	2624	337	2287	120	98.4	129.2	108.8	108.7	109.1	44.7
	8.500	0.137	2752	340	2412	120	99.5	129.2	117.8	117.8	119.7	41.2
	9.000	0.142	2868	342	2526	123	97.8	129.2	125.7	125.6	129.4	38.7
	9.500	0.148	3001	342	2660	134	99.3	129.2	132.9	132.9	138.6	36.6

Table 0-2: ULS Parametric Study - Overview

Variant	Tower Bottom [mOD]	Weight [t]				Bending Moment (normalized)			Bending Moment (absolute values) including PSF and inclination moment			Geotechn. result
		MP+TP (+Skirt)	TP	MP	Skirt	Interface [-]	MP-TP [-]	Mudline [-]	Interface [MNm]	MP-TP [MNm]	Mudline [MNm]	EBD [m]
Cylindrical TP	7.500	2370	303	2067	-	0.984	0.984	0.965	738	823	1292	32.63
	8.000	2416	311	2106	-	<b>1.000</b>	<b>1.000</b>	<b>1.000</b>	751	836	1338	32.93
	8.500	2540	323	2217	-	1.018	1.025	1.094	765	858	1463	33.75
	9.000	2663	322	2341	-	1.111	1.125	1.223	834	941	1636	34.87
	9.500	2803	325	2478	-	1.166	1.181	1.304	875	988	1744	35.58
Cylindrical TP + Skirt	7.500	2485	311	2085	88.4	0.981	0.981	0.985	736	821	1317	32.79
	8.000	2557	323	2140	95.0	0.995	0.995	1.020	747	832	1364	33.10
	8.500	2680	329	2249	101.7	1.043	1.052	1.119	783	880	1498	33.97
	9.000	2832	335	2390	107.4	1.142	1.155	1.246	857	966	1668	35.08
	9.500	2953	333	2507	114.0	1.193	1.208	1.341	896	1011	1794	35.90
Conical TP + Skirt	7.500	2573	336	2140	96.9	0.996	0.996	1.043	748	833	1396	33.31
	8.000	2662	337	2221	103.6	1.045	1.057	1.138	785	884	1523	34.14
	8.500	2809	340	2359	110.2	1.145	1.153	1.224	859	964	1637	34.88
	9.000	2959	342	2502	115.9	1.253	1.264	1.356	941	1057	1814	36.03
	9.500	3119	342	2655	122.6	1.306	1.316	1.445	981	1101	1934	36.81