FOWPI

FIRST OFFSHORE WIND PROJECT OF INDIA

Advisory

Foundation Concept Design

This Project is funded by The European Union

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1 About FOWPI

The First Offshore Wind Project of India (FOWPI) is part of the “Clean Energy Cooperation with India (CECI) “, which aims at enhancing India's capacity to deploy low carbon energy production and improve energy efficiency, thereby contributing to the mitigation of global climate change. Project activities will support India's efforts to secure the energy supply security, within a well-established framework for strategic energy cooperation between the EU and India.

FOWPI is planned to achieve the first 200MW sized offshore wind farm near the coast of Gujarat, 25km off Jafarabad. Project will emphasis on bringing the vast experience of offshore wind rich European countries to India which aims to provide technical assistance for setting up the wind-farm and creation of a knowledge centre in the country.

FOWPI will be led by COWI A/S (Denmark) with key support from WindDForce Management Ltd. (India). The project is supported by European Union (EU), Ministry of New and Renewable Energy- India (MNRE) and National Institute of Wind Energy- India (NIWE).

Project is awarded under the Indo-European co-operation on Renewable Energy Program and funded through European Union.

FOWPI will focus on finalisation of design and technical specification of the windfarm including foundation, electrical network, turbines etc.. This will also include undertaking specific technical studies for the selected site (based on the outcome of FOWIND project), including coastal surveys, environmental assessments, cost-benefit analysis, transmission layouts, monitoring systems, safety measures, and other relevant technical studies as identified.

Contract: No 2015/368469 Start 01-2016 Duration: 42 months

2 Disclaimer

3 Copyright statements

4 Acknowledgements

FOWPI is grateful for the suypport provided by European Union (EU), Ministry of New and Renewable Energy-India (MNRE), National Institute of Wind Energy- India (NIWE), and the Wind Industry.

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Weather Windows for Installation

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FOWPI - ADVISORY

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APPENDICES

1 Introduction

This document has been prepared with the purpose of providing preliminary foundation design at the 200 MW FOWPI offshore wind farm site area in Gujarat, India. The document has been prepared by COWI on behalf of NIWE with the purpose of being used by NIWE to the call for tenders.

India has one of the fastest growing economies in the world and has a rapidly increasing energy demand. The Clean Energy Corporation Initiative has the purpose of assisting India to meet the future energy demand by utilising sustainable energy generation technologies and to introduce energy efficiency measures.

India has already introduced renewable energy in the energy supply system and has installed various renewable energy technologies during recent years. Wind energy plays an important role, with approximately 23 GW of installed onshore wind power capacity.

Offshore wind energy has become an important factor in European countries, with a total installed capacity of more than 14 GW. The offshore wind technology faces a number of technical challenges due to the harsh installation and operation conditions. Whereas the construction cost for the first offshore wind farms implemented were relatively high, the rates for new offshore wind projects in Europe are steeply declining. This has been achieved through lessons learned in design, manufacture, installation and O&M.

The present conceptual foundation study is based on the requirements set up in IEC 61400-3 code of practise and in the DNVGL family of codes as also applied in Europe.

Site-specific measurements with regards to metocean data (wind, waves, current and water level), detailed geotechnical and geophysical campaigns, as well as a detailed bathymetric survey are required in future project stages in order to develop the present conceptual design into a detailed design. The reported design is based on preliminary data only.

2 Summary

The concept design of foundation structures for an offshore wind farm in Gujarat, India, is presented in this report.

The layout of the foundation elements, as well as the applicable loads and relevant environmental conditions in the current site for the design are defined in accordance with the European standards and guidelines (Ref. /1/, Ref. /2/, Ref. /9/, Ref. /10/ and Ref. /11/).

An introductory description of structural appurtenances is provided, as well as loads typically applied to the foundation structure, according to European standards (e.g. Ref. /1/). Nevertheless, as further stated, the numerical values presented provide an initial insight into their order of magnitude, rather than reflect accurately final values for the current project.

Conventional methods and devices for corrosion protection in offshore wind farms are described under European recommendations (Ref. /5/).

The pertinent loads applied by the wind turbines to the foundation structure have been estimated on the basis of loads for turbines of size similar to the selected 3MW and 6MW reference turbines (Ref. /22/).

The environmental conditions which are relevant for the determination of the splash zone range, the interface level between the tower and the transition piece, and the upper and lower bounds of the boat landing structure are given in the preliminary metocean study report (Ref. /18/).

The soil conditions considered in the determination of the p-y and t-z curves, as well as in the calculation of adequate embedment length of the monopile are given in the geotechnical report (Ref. /20/). From the available data, a MP with 5.50m of bottom diameter, 528.8 MT of weight, and 57.60m length was designed for the 3MW model. Likewise, a MP with 7.00m of bottom diameter, 873.9 MT of weight and 63.00m length was designed for the 6MW model. Their respective design embedment lengths are 35.60m and 41.00m. Further features are presented in Section 6.4.1.

3 References, abbreviations and definitions

3.1 References

General project related references, as well as technical references relating to the current design report are given in the following subsections.

3.1.1 Standards

The following standards have been used as basis for the foundation concept design:

Ref. /1/ DNV GL AS, Standard DNVGL-ST-0126: Support structures for wind turbines. April 2016.

Ref. /2/ DNV GL AS, Standard DNVGL-ST-0437: Loads and site conditions for wind turbines. November 2016.

Ref. /3/ DNV GL AS, Recommended Practice DNV-RP-C202:2013: Buckling Strength of Shells, 2013.

Ref. /4/ DNV GL AS, Recommended Practice DNVGL-RP-C203: Fatigue design of offshore steel structures. April 2016.

Ref. /5/ DNV GL AS, Recommended Practice DNVGL-RP-0416: Corrosion protection for wind turbines. March 2016.

Ref. /6/ DNV GL AS, Service Specification DNVGL-SE-0074: Type and component certification of wind turbines according to IEC 61400-22, December 2014.

Ref. /7/ DNV GL AS, Standard DNV-OS-J101: Design of Offshore Wind Turbine Structures. May 2014.

Ref. /8/ DNV GL AS, Recommended Practice DNVGL-RP-B401: Cathodic protection design. June 2017.

Ref. /9/ Germanischer Lloyd (GL-COWT): Guideline for the Certification of Offshore Wind Turbines, Rules and Guidelines, IV – Industrial Services, Part 2, Edition 2012.

Ref. /10/ IEC, Standard 61400-1: 2014: Wind Turbines – Part 1: Design requirements, 2014.

Ref. /11/ IEC, Standard 61400-3: 2009: Wind Turbines – Part 3: Design requirements for offshore wind turbines, 2009.

Ref. /12/ ISO, Standard 19901-1:2005: Petroleum and natural gas industries -- Specific requirements for offshore structures -- Part 1: Metocean design and operating considerations, 2005.

Ref. /13/ EN 10025-2:2005: Hot rolled products of structural steels – Part 2: Technical delivery conditions for non-alloy structural steels, 2005.

Ref. /14/ EN 10025-3:2005: Technical delivery conditions for normalized rolled weldable fine grain structural steels, 2005.

Ref. /15/ EN 10025-4:2005: Technical delivery conditions for thermomechanical rolled weldable fin grain structural steels, 2005.

Ref. /16/ EN 10088-1: 2005: Stainless steels - Part 1: List of stainless steels, 2005.

3.1.2 Public

The following references relate to data which has been published and accounted for general information in this report:

Ref. /17/ Wind Europe, 2017, The European offshore wind industry – key trends and statistics 2016. Available at <https://windeurope.org/about-wind/statistics/offshore/european-offshore-wind-industry-key-trends-and-statistics-2016/>

3.1.3 Project documents

The following project documents have been used as basis for the foundation concept design:

Ref. /18/ COWI, FOWPI – Metocean Study, COWI Report No. A073635-014-001, Rev. 1.0

Ref. /19/ COWI, Gujarat. Cyclone Hindcasting Study, COWI Project No. A073635-014

Ref. /20/ FOWIND, Pre-feasibility study for offshore wind farm development in Gujarat, Chapter 6.1.5.2: Ground earthquake risk, pp.55-57, May 2015.

Ref. /21/ GENSTRU, Factual Report on Geotechnical Investigation for NIWE Project at Pipavav, March 2017.

Ref. /22/ FOWIND, Wind turbine layout and AEP, January 2017.

3.2 Abbreviations

The main abbreviations and symbols used in the present report are listed below.

ALS	Accidental limit state
BE	Best estimate
CA	Corrosion allowance
DEL	Design equivalent loads
DFF	Design fatigue factor
FLS	Fatigue limit state
HSE	Health, safety and environment
HSWL	Highest sea water level
ICCP	Impressed current cathodic protection
ILA	Integrated load analysis
LAT	Lowest astronomical tide
LB	Lower bound
LSWL	Lowest sea water level
MP	Monopile
O&M	Operation and maintenance
OSS	Offshore substation
PDA	Pile drive analysis
RNA	Rotor-nacelle assembly
SLS	Serviceability limit state
TP	Transition piece
UB	Upper bound
ULS	Ultimate limit state
WTG	Wind turbine generator

4 Design concepts

The following subsections provide a descriptive overview of the main offshore wind foundation design concepts.

4.1 Foundation types

Due to its site-specific character, offshore wind foundations represent a significant part of a project's capital expenditure, and therefore optimization of its structure might lead to substantial savings. In this regard, several factors are relevant to the foundation technology selection, i.e. water depth, wind turbine MW class, cost, ground conditions, installation vessels availability and local fabrication facilities between others. Typical foundation concepts are presented in Figure 4-1.

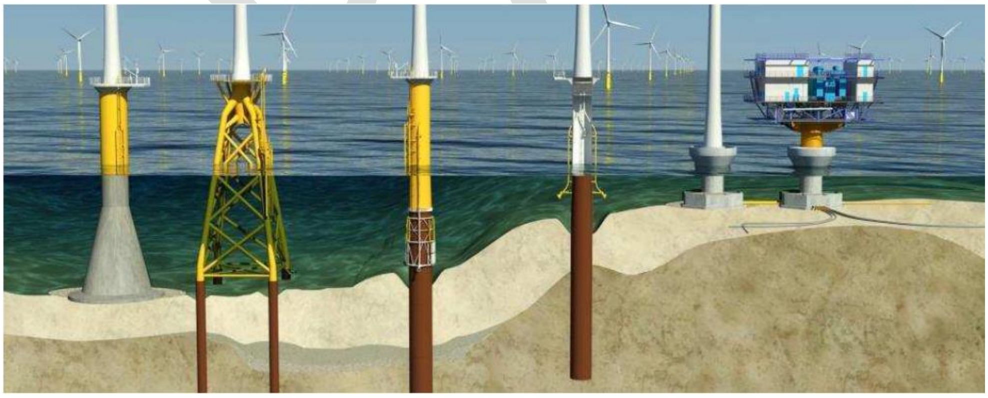


Figure 4-1 Illustration of OWF foundations designed by COWI. From left: Thornton Bank concrete gravity based, Wikinger jacket (preliminary stage), two times London Array monopile, Nysted WTG and Rødsand 2 offshore substation concrete gravity based

For the pilot site in question, three possible foundation types are considered: monopile (MP), jacket and concrete gravity based.

4.1.1 Concrete gravity based foundations

Concrete gravity based foundations are the most economic between the three alternatives regarding the foundation material costs. The system is well proven for application in water depths of up to around 40m, and features a reduced fatigue and corrosion sensitivity. The structures can be manufactured as simple concrete elements, which are further transported by barges or vessels to the site and then placed on the (prepared) seabed, which dismisses the pilling process. Nevertheless, the need of a robust soil at the seabed level for such system may require some preparations. Likewise, the slow and highly space demanding fabrication process, as well as the heavy lifting and transporting restrict its appliance in some locations.

For the present case, the expected thick layer of soft soils directly below sea floor level, which has to be removed and replaced by a gravel bed for a gravity based foundation in order to provide a level and stable support, is considered costly and critical. Moreover, the difficult and safe handling of such heavy foundation structures require some experience and the right equipment.

Under these circumstances, COWI discredits this system's application in FOWPI.

4.1.2 Steel jackets

Steel jackets are used in European offshore wind farms at deeper sea (30-50m) and for larger WTGs. From the fabrication perspective, the lower requirements concerning the welding and assembly of the structural components (smaller wall thicknesses and lighter components) and the inexpensive workforce in India in comparison to Europe would facilitate its production in the region. Nevertheless, such process demands a high number of welds or requires expensive castings for the joints. Therefore, jackets are generally not used in shallow waters.

The fabrication procedure is expected to be costlier than the one for monopiles even considering the lower workforce costs in India. As the installation expenses for jackets are also relatively high due to the larger installation time needed compared to monopiles and considering the relatively shallow water depth, COWI perceives this system to be less suitable in the context of FOWPI.

4.1.3 Monopiles

Monopiles are used in most offshore wind farms worldwide, and the technology is being constantly developed for larger turbines and deeper locations. The system is currently applied in sites up to 40 meters deep, and for 6-8 MW wind turbines in its majority. Moreover, it is suitable for a wide variety of soil conditions, and consists of a single steel pile embedded into the seabed, which is further connected to the WTG tower through a transition piece (TP), as displayed in Figure 4-2.

A monopile foundation consists of a 50-80m long steel pile of 5-8m diameter which is generally driven into the seabed soil. On top a 20-30m high "Transition Piece (TP)" is mounted. The TP contains a number of appurtenances, including working platform, boat landing, J-Tubes for cable protection and much more.

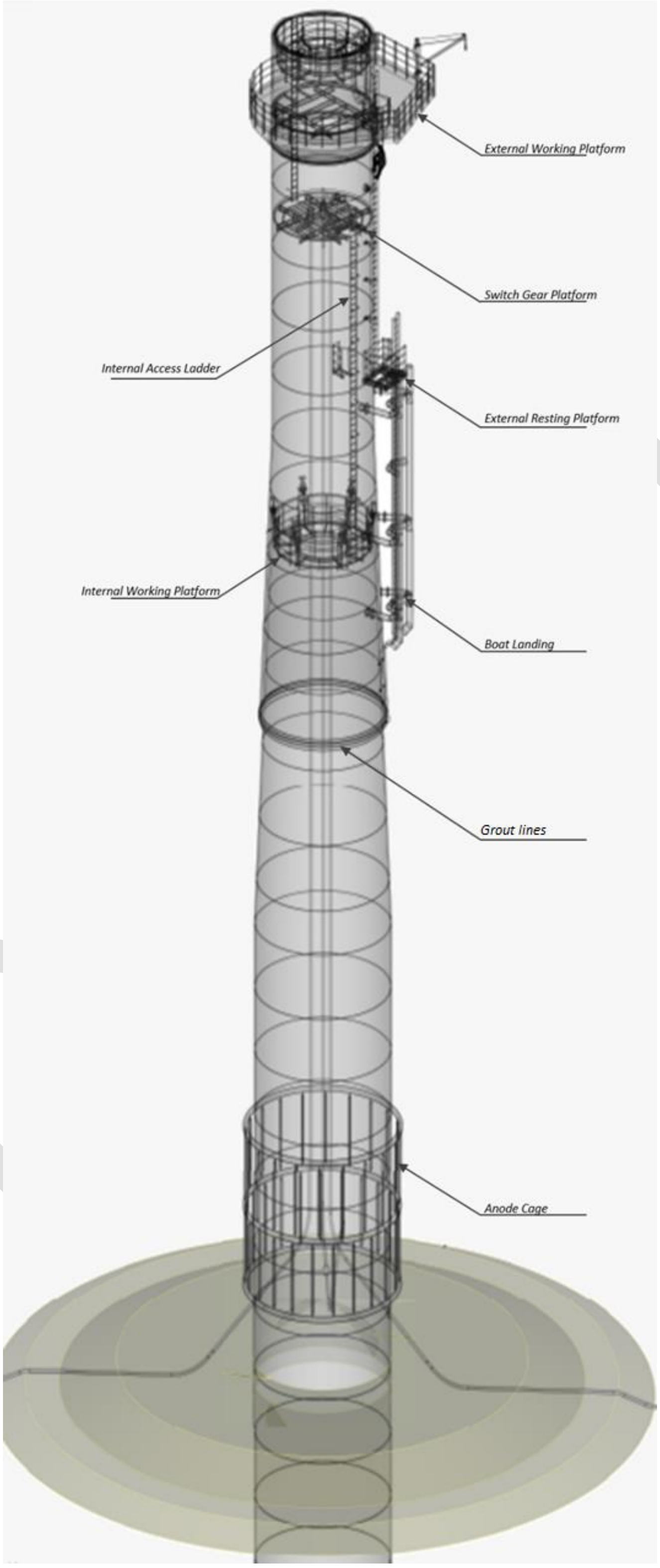


Figure 4-2 Example of monopile foundation including some secondary steel components (schematic illustration).

The connection between MP and TP may be either grouted, as in most of the current installed projects, or bolted, which is being frequently adopted in the latest European projects. For both cases, a skirt generally covers the interface with the MP.

In this regard, COWI sees the adoption of grouted connections for FOWPI as the most recommended practice. As a pioneer project of its type in India and allowing a wider tolerance range for the foundations' installation, it is understood to be the best approach, as designs that feature bolted connections do not present the same flexibility as those with grouted connections regarding the installation process.

In most cases, the pile is driven into the subsoil. If the subsoil does not allow an installation by pile driving, drilling a hole into which the pile is placed and the annulus grouted, or a combination of drilling and driving are possible, but costly and technically challenging.

Monopiles' rapid fabrication and installation processes lead to lower costs in relation to other foundation types. Furthermore, the water depth is relatively small. Hence, COWI perceives it to be the most suitable option at the FOWPI site.

4.2 Design process and loads

Foundation design is one of the most critical stages of offshore wind projects regarding the complexity of the investigations and the relevance for the stability of the wind turbine. The core of the design process is to determine the design loads applied on the structure in order to define its geometrical dimensions.

Some critical parts of the design and design process are listed in the following:

- The interface between the wind turbine base and the top of the foundation is a critical point through which the high dynamic loads from the turbine are transferred to the foundation structure via a flange connection.

- The soil is exposed to cyclic loading, which may lead to a degradation of soil properties and a reduction in soil bearing capacity during the lifetime of the foundation.

- Hydrodynamic loads from waves and currents on the main structural parts and on the appurtenances.

- Earthquake loading in regions especially with potential for soil liquefaction must be concerned in the calculations.

- The determination of the relevant design loads as such. This is generally done in a site-specific integrated load analysis, i.e. all relevant loadings from wind, waves etc. are combined in various load combinations and applied to a model of the entire system. Generally, time series of the loadings are applied to the system in a dynamic model. The governing sectional forces are determined over the modelled time span. As these are depending on the dynamic behaviour of the system which is likewise depending on the stiffness of the system, the dimensioning and load determination is an iterative process.

- The load iteration process is typically done for a limited number of representative WTG locations for one site. The loads have to be transferred to the other WTG locations in some way and the validity of the load application has to be verified.

As the wind turbine model for FOWPI has not yet been defined, COWI is developing this concept design for the two selected reference turbine models of 3MW and 6MW, respectively (Ref. /22/).

Design loads have been developed for FOWPI for the two selected turbines. The investigations as described in the following subsections are carried out accordingly.

4.2.1 Geotechnical design/assessments

The main geometry is assessed by geotechnical (and hydraulic) design, supplemented by structural design. The geotechnical design comprises soil strength and embedment length assessments. The design soil parameters are generally defined in a so-called soil and foundation expertise or geotechnical interpretation report based on site-specific soil investigations for all wind turbine positions. For FOWPI, the assessment at this stage will be based on the results provided in the geotechnical report (Ref. /20/).

4.2.2 Hydraulic design/assessments

The hydraulic design covers the interpretation of metocean data and thereof deduced load case definitions along with the actual generation of loads from waves and currents. For FOWPI, the assessment at this stage will be based on the results provided in the metocean study report (Ref. /18/).

4.2.3 Structural design

After a stable and quantitatively optimised solution is determined in the iterative integrated load analysis (ILA), the structural design is finalised generally based on a linear elastic structural analysis: geometry, material properties, structural verification principles, loads and load combinations are included in a finite element model in order to carry out the required design checks.

In general, structural elements are modelled as beam elements and only primary steel members are modelled explicitly, while the secondary steel (appurtenances) is taken into account by increased hydrodynamic force coefficients and additional weights. The foundation model extends from the tower top flange at the RNA to the tip of the embedded pile(s). The RNA is generally modelled as a point mass with rotary inertia.

Variations of water depth and soil profiles within the wind farm have to be considered.

The main design checks are described in the following sections.

4.2.4 Service limit state (SLS) checks

Deformation requirements are to be fulfilled, e.g. the maximum inclination of the turbine is limited to a certain value at the end of the planned lifetime in such a way that the turbine can still be operated. Further, there are certain limits to deflections of platforms and gratings in order to operate the facilities in a safe way.

4.2.5 Ultimate limit state (ULS) checks

The material utilisations are verified (stress checks and shell buckling). Furthermore, the overall structural integrity is verified (column buckling).

Applicable load combinations and partial safety factors are defined in Section 5.6 for this concept design, whereas general structural design issues are covered in Section 6. Some basic assumptions for such verifications are the following:

- ULS check of members and joints (circumferential welds)
- Lower bound soil resistance
- 100% corrosion allowance
- Full marine growth

4.2.6 Fatigue limit state (FLS) checks

Damage equivalent loads and/or Markov matrices can be used for the fatigue design checks. The standards and recommendations in (Ref. /1/) and (Ref. /4/) are typically used as a basis.

The design checks are generally done separately for the outside and for the inside of the TP and MP at the circumferential welds. The nominal stresses on the outer side of the MP are computed according to mechanical principles. The nominal stresses on the inside are reduced compared to the stresses on the outside due to the shorter distance to the pile axis. Attachments to the TP can be considered by stress concentration factors. Larger fabrication misalignments than included in the applied S-N-curves are covered by increasing the stress concentration factors for the specific situation according to (Ref. /1/), Section 4.11.3.

Some basic assumptions for such verifications are the following:

- FLS check of joints (circumferential welds)

- Characteristic (best estimate) soil resistance
- 50% corrosion allowance
- Full marine growth

4.2.7 Accidental limit state (ALS) checks

In general, accidental limit state (ALS) assessments are limited to boat impact checks. These are done for the overall structure, i.e. it is checked whether the wind turbine facilities can survive a boat impact without a collapse.

Furthermore, the boat impact of crew transfer vessels on the boat landing is checked. This is generally done for two scenarios: operational conditions and accidental conditions.

4.2.8 Frequency checks

The structures have to fulfil stiffness requirements to avoid resonance effects with the WTG production frequencies. The optimal range of first natural frequencies will be assessed during the course of the load iteration process in cooperation with the wind turbine supplier. Furthermore, the lower-bound and the upper-bound first eigenfrequencies will be checked against the frequency range given by the 1P and 3P frequencies including a safety margin defined by the wind turbine supplier with a corresponding lower-bound and upper-bound structural model.

Further, the mode shapes are determined and compared with those of the representative locations of the ILA together with the best estimate first eigenfrequencies in order to verify the applicability of the determined design loads to the specific WTG locations.

The three main conditions for the frequency checks with typical basic assumptions are the following:

Lower bound (LB) – check of lower bound eigenfrequency

- Lower bound modal frequency analysis of the entire foundation structure relevant for the verification of the eigenfrequency range defined by the WTG supplier
- Lower bound soil resistance based on lower bound soil parameters
- 100% corrosion allowance
- Stiffness of skirt (if any) and cementitious filler (if any) not considered, but mass
- Highest still water level and global sea water level rise
- Full marine growth

Best estimate (BE) – characteristic lower bound eigenfrequency relevant for load calculation

- Characteristic or best estimate lower bound modal frequency analysis of the entire foundation structure relevant for the load calculation and the load transfer from the load calculation MP location to other MP locations within a cluster, i.e. it is checked that the determined design loads can be basically applied to the location-specific designs.

- Lower bound soil resistance based on characteristic (best estimate) soil parameters
- 100% corrosion allowance
- Stiffness of skirt (if any) and cementitious filler (if any) not considered, but mass
- Highest still water level and global sea water level rise
- Full marine growth

Upper bound (UB) – check of upper bound eigenfrequency

- Upper bound modal frequency analysis of the entire foundation structure relevant for the verification of the eigenfrequency range defined by the WTG supplier
- Upper bound soil resistance based on upper bound soil parameters
- 0% corrosion allowance (nominal wall thicknesses)
- Stiffness and mass of skirt (if any) and cementitious filler (if any) fully considered within model
- Lowest still water level and no global sea water level rise
- No marine growth

4.3 Appurtenances

Appurtenance concepts are described in the following on the bases of European experience. Such attachments that comprise the secondary steel category are installed in its majority inside and on the external surface of the transition piece, and must be also addressed during the foundation design stage (primary steel) due to their respective impact on the structure. This class includes, between others, the external working or service platform, the internal platforms, boat landing structure, anode cage (or any other structural component for cathodic protection against corrosion), access ladders and resting platforms, inter-array cable frame (support for cables), and J-tube (if any; external cable protection and guidance).

Furthermore, it must be remarked that while most of the assumptions taken for the design of the referred structures are based in European standards, some specific cases might also take into account requirements established by the wind turbine supplier, which mostly leads to some difference on the considered values. In this regard, concrete values given in the following sections are general examples described merely to indicate the order of magnitude, and therefore should not be taken as fixed for FOWPI.

4.3.1 Internal working platform

The internal working platform is the first one inside the foundation from the seawater level. It is installed under the airtight deck and shall provide access for inspections and work operations (see Figure 4-2).

In the example layout shown below (see Figure 4-3), the platform additionally provides horizontal support for cables and is supported by the TP through so-called stopper-levelling-guide (SLG) units. Furthermore, the work area on it is divided in two parts bounded by the hand rail: the area between the hand rail and the TP wall, and the area within the hand rails, where the cables run through their respective guides.

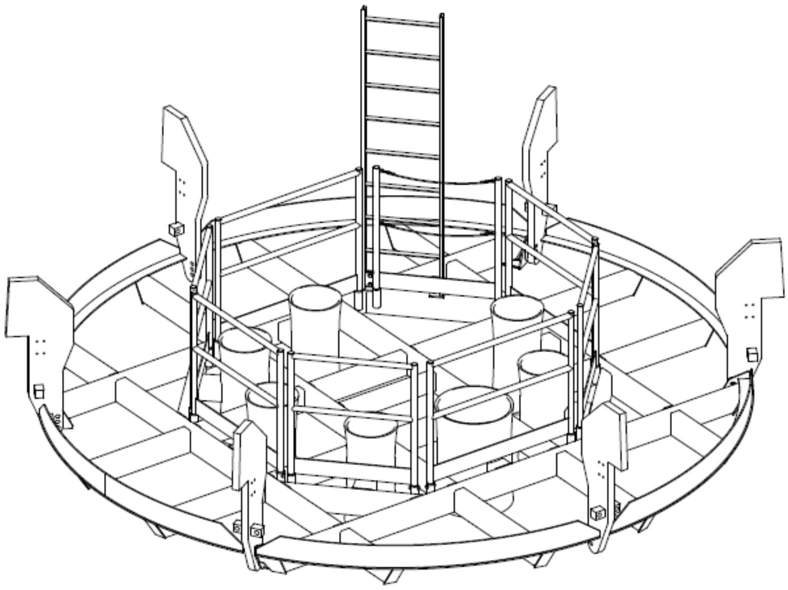


Figure 4-3 Example of internal working platform layout, gratings not shown

4.3.2 Airtight deck

The airtight deck is typically the second platform within the foundation from the seawater level, which is conventionally positioned under the flange connection between the monopile and the transition piece in case of a bolted MP-TP connection in order to limit its exposure to corrosive environmental conditions from inside the pile (see Figure 4-4).

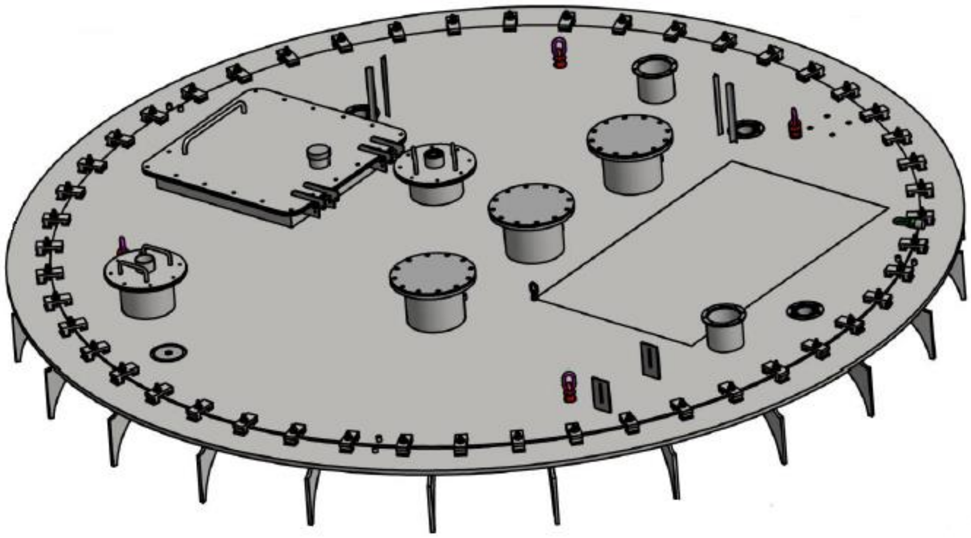


Figure 4-4 Example of airtight deck layout

The design is developed in accordance with the ultimate limit state (ULS) (strength and capacity) and the serviceability limit state (SLS) requirements (e.g. deflection criterion) for:

- dead load of the structure
- cable hang-off loads
- dead load of the bolts for the flange connection (in case of bolted connection between the monopile and transition piece)
- variable distributed (e.g. p = 5 kN/m²) and concentrated life loads applied in the most unfavourable position (e.g. P = 2 kN at 200 x 200 mm²)
- atmospheric loads due to volume changes of air below the airtight platform caused by tidal variations (considered as an accidental load case, as the pressure will only occur if the ventilation system is out of order)
- friction loads from cables during cable pull-in
- cable pull-out loads (if any)
- For railings, a distributed load of e.g. 1.00 kN/m acting in all directions is generally considered.

4.3.3 Boat landing and (external) resting platforms

The boat landing structure, resting platforms and ladders are installed on the external surface of the transition piece for facilitating the access to the WTG (see Figure 4-2). There are strict HSE requirements for this access system in order to provide a safe access to the WTG facilities. Generally, the boat landing is positioned on the opposite side of the face hit by the main wave direction in order to avoid the boat to collide with the TP during the service operations. There are different general concepts for the boat landing structure, being it either replaceable or non-replaceable. The first one is the most commonly adopted in European projects.

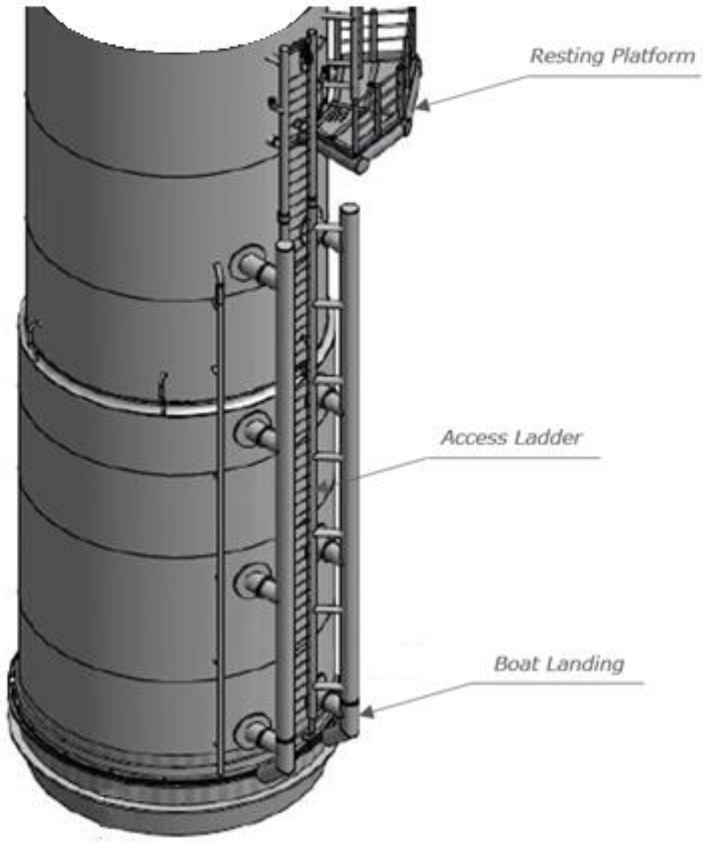


Figure 4-5 Example of transition piece with boat landing structure and external access system

The boat landing structure is designed to be able to resist permanent loads from e.g. dead loads, life loads, wave and current loads, vessel impact loads and thrust forces experienced during normal vessel operation for the ULS, SLS, FLS and ALS load combinations. Furthermore, the additional global loads introduced into the boat landing due to the stiffness of the boat landing are also considered ("master-slave-effect").

The boat landing, resting platform and access ladder typically consist of beam elements, which are supported at the TP. The structures are verified by spatial framework analysis. In general, the following loads are considered for their design:

- dead loads of the structure including marine growth
- boat impact and thrust forces, considered from bottom to top of the boat landing
- wave loads (horizontal and vertical)

- variable distributed (e.g. p = 5 kN/m²) and concentrated life loads applied in the most unfavourable position (e.g. P = 2 kN at 200 x 200 mm²)

- load on ladder rungs applied in the most unfavourable position (e.g. P = 2 kN)

- loads on ladder rungs, which can be used as personal anchor points as requested by WTG supplier or required by HSE considerations applied in the most unfavourable position (e.g. P = 10 kN).

Due to the typically removable characteristic of the boat landing, plastic deformations of the replaceable parts are allowed in the case of accidental boat impact. Specific dimensions of the boat landing structure for FOWPI are presented in Section 5.3.

Generally, there is a resting platform at the top of the boat landing in order to limit the climbing height. Depending on the remaining climbing height between this resting platform and the external working platform, an intermediate resting platform might be required following HSE requirements.

The intermediate resting platform (if any) is likewise generally designed for wave, gravity and live loads for ULS and SLS load combinations.

The ladders of this external access system shall be equipped with a suitable fall arrest system and a number of hook-on points following HSE requirements.

4.3.4 Internal decks and access ladders

Between the airtight platform and the top of TP there are generally a number of internal decks/platforms depending on the equipment installed in the TP, e.g. switch gear platform for switch gear and further control cabinets, or bolting platform for bolting of flange connection between TP and WTG tower (see Figure 4-2).

The internal decks are designed to be able to resist permanent loads from dead loads, live loads and relevant equipment loads for the ULS, SLS and ALS (if applicable) load combinations.

Access ladders are designed to be able to resist permanent loads from e.g. gravity loads and live loads for the ULS and SLS load combinations.

The internal platforms typically consist of beam elements with gratings, which are supported at the TP wall. The structures will be verified by spatial framework analysis. Sometimes, a modular design in which all internal platforms are fixed to a separate structural framework can be seen. It is in this case installed into the TP tube in one go.

The following loads are generally considered for the design of the internal platforms:

- dead load of the structure
- loads due to cable pull-in
- dead load of stored bolts (if any) for the flange connection
- variable distributed (e.g. p = 5 kN/m²) and concentrated life loads applied in the most unfavourable position (e.g. P = 2 kN at 200 x 200 mm²)
- loads on ladder rungs applied in the most unfavourable position (e.g. P = 1.5 kN).
- For railings, a distributed load of e.g. 1.00 kN/m acting in all directions is considered for design.

Internal access ladders above the airtight deck are generally standard aluminium or steel ladders, which meet the required safety regulations. Internal ladders below the airtight deck are generally steel ladders. They are equipped with a suitable fall arrest system following HSE requirements.

4.3.5 External working platform

The external working platform, or service platform is the main structural component for WTG access as well as for inspection, maintenance and repair work (see Figure 4-2). It generally consists of (steel) beam elements supported at the top of the transition piece. Alternatively, a concrete service platform is a viable solution.

At this level, a service crane is typically installed to assist in maintenance or repair operations. Further, the platform can serve as storage for maintenance equipment. In this regard, the platform is designed to resist dead loads, live loads, live loads on lay-down areas, service crane loads, vertical wave run-up loads and also temporary loads from equipment at lay-down areas in accordance to the ultimate limit state (ULS) (strength and capacity) and the serviceability limit state (SLS) for the in-place analysis.

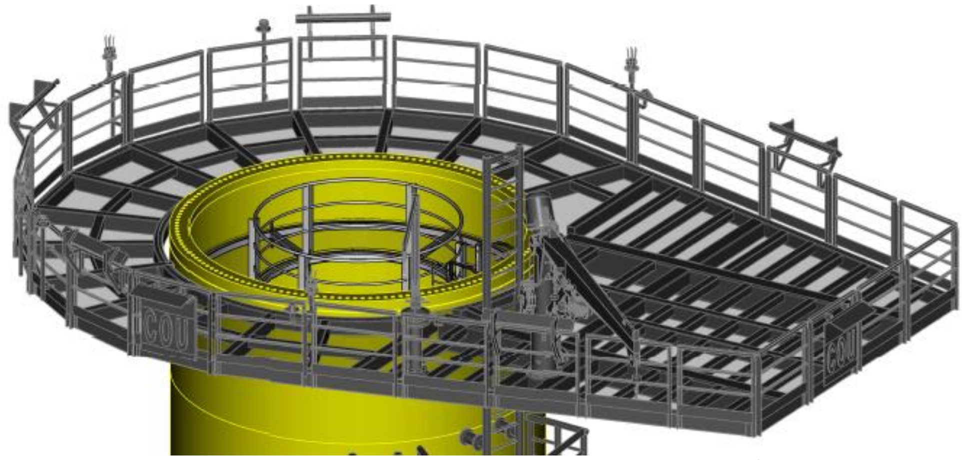


Figure 4-6 Example of service platform with service crane (gratings not shown)

The following loads are generally considered for the design of the external working platform:

- dead load of the structure
- variable distributed (e.g. p = 5 kN/m²) and concentrated life loads applied in the most unfavourable position at the walkway area (e.g. P = 2 kN at 200 x 200 mm²)
- variable distributed (e.g. p = 15 kN/m²) and concentrated life loads applied in the most unfavourable position at the lay-down area (e.g. P = 15 kN at 100 x 100 mm)*
- loads due to service crane operations*
- loads for transformer replacement or any other replacement services for WTG parts*
- loads for placing a gangway from an installation or maintenance vessel*
- wave loads (as the working platform is typically placed well above the 50-year design wave, slamming forces can be neglected and only wave run-up forces need to be considered according to Section 5.2.7)
- For railings, a distributed load of e.g. 1.00 kN/m acting in all directions is considered for design.

*Note: These values are normally defined according to specific requirements from the WTG supplier and/or the O&M concept.

4.3.6 Appurtenances for corrosion protection

Distinct options of corrosion protection appurtenances are commonly used in offshore wind projects. The main standard systems considered are ICCP anodes, sacrificial anodes (GACP), and anode cages (see Figure 4-2). A detailed description of corrosion protection systems is provided in Section 4.4.

ICCP anodes provide corrosion protection through an impressed current system. Figure 4-7 shows a generic layout of the system installed at the bottom of the TP.

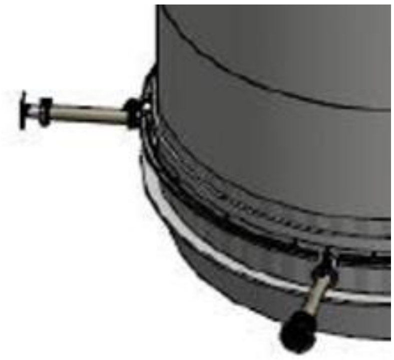


Figure 4-7 Example of ICCP system installation at the TP bottom

For sacrificial anodes, electrochemical elements (typically aluminium or zinc) are installed on the bottom of the TP, and they are the current source for the GACP system, being therefore continuously consumed (see Ref. /8/). A schematic view of an example layout is shown in Figure 4-8.

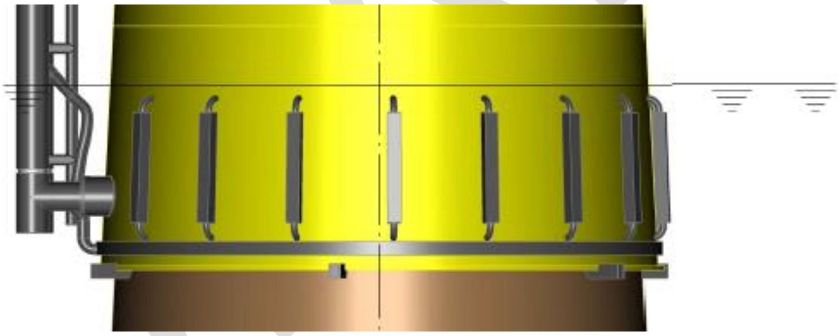


Figure 4-8 Example of GACP system installed on TP bottom

A corrosion protection cage (if required in case the anodes at TP bottom do not provide sufficient corrosion protection around sea floor level) is proposed to be positioned as close as possible to the seabed (see Figure 4-9). The functionality of the cage is to support the additional external anodes. This is to be obtained in accordance with Ref. /5/ over the design service life of the wind farm.

The anode cage is installed over the monopile, once the monopile has been driven into the seabed, as schematized in Figure 4-9.

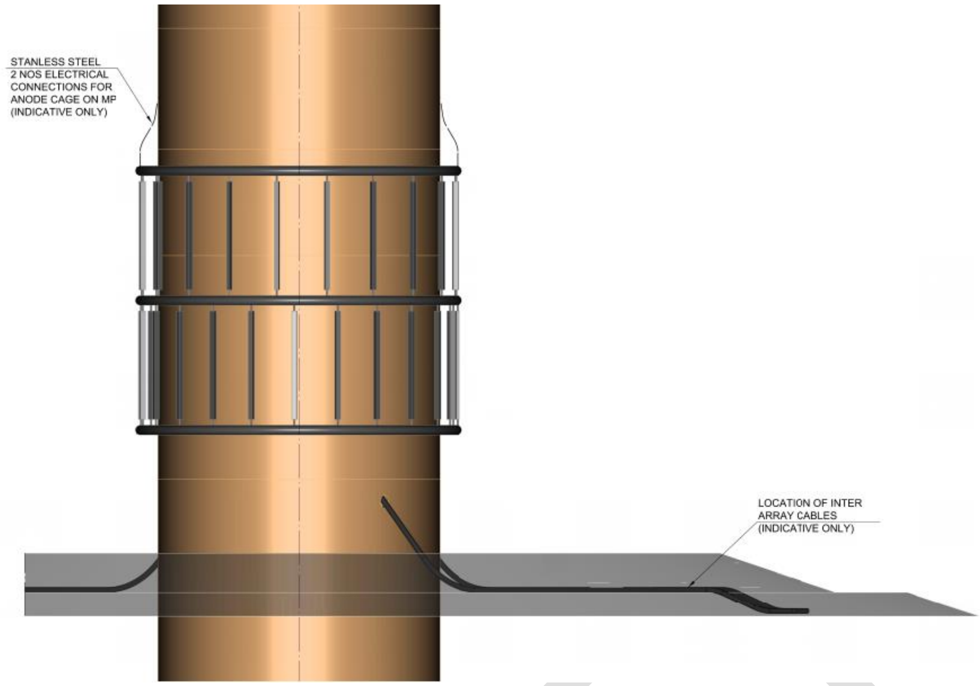


Figure 4-9 Example of anode cage on monopile – Schematic view

Further requirements regarding the structure, material properties, position and distribution of electrical connection wires, as well as coatings are to be defined according to Ref. /5/.

4.3.7 Cable protection

In order to limit or prevent the action of corrosive conditions from the seawater and mechanical actions by waves and currents, a protection system is required for the interarray and export cables, especially at the interface between monopile and seabed. This can be carried out through different concepts.

In one of the common practices, each end of the interarray cables is connected to the WTGs and OSS, through an open-ended "J" shaped tube or pipe, which can be either internally or externally attached to the structure. The J-tube extends from a platform deck or hang-off at the tower base down to the bottom bend near the seabed, and can be either made of steel or some flexible material.

Likewise, a free hanging cable system is also feasible, combined with a flexible protection system installed around the cable entry hole in the MP and down to the scour protection on the seabed.

4.4 Corrosion protection

Foundation structures are exposed to a corrosive marine environment. Furthermore, certain bacteria might be present in the subsoil leading to microbial induced corrosion (MIC). Therefore, the structures have to be protected against corrosion by an adequate corrosion protection system.

The design of the corrosion protection system can, for instance, follow the European requirements and guidelines (Ref. /5/). In general, such system is composed of coatings, sacrificial anodes or ICCP, or a value of corrosion allowance (CA), being also a common practice in European projects the combination of two or more options for a single design. Regardless of the adopted methodology, it shall be designed in order to attend the whole design lifetime of the project.

For designing the corrosion protection system, the structure is approached from two distinct perspectives: External and internal surfaces. A generic model of protection strategy is as it follows:

EXTERNAL

- Atmospheric zone: Coating
- Splash zone: Coating and CA, partly protected by anodes (if submerged)
- Submerged zone: Coating and anodes
- Mudline zone: CA, coating and anodes
- Embedded zone: CA

INTERNAL

- Atmospheric zone: Coating
- Tidal zone: Coating and CA, partly protected by anodes (if submerged).
- Submerged zone: Anodes (CA and coated welds for temporary protection until the anodes are installed or energized).

The coating systems usually includes either paint systems, hot-dip galvanizing or thermal metal spraying. Components delivered with a corrosion protection need to fulfil a high durability to the environment they are exposed to.

The cathodic protection system is either an impressed current system (ICCP) with inert anodes for protection or a galvanic anode cathodic protection (GACP) system with sacrificial anodes of aluminium or zinc. A typical layout for GACP is presented in the previous section (4.3.6).

A value of corrosion allowance shall also be considered in accordance with the European standards, especially in the splash zone, which is included in the determination of the thicknesses of the shells of the foundations.

5 Basis of design

A description of the basic aspects to be covered, as well as main considerations taken into account in FOWPI foundation concept design with the relevant regulating standards is presented in the following sections.

5.1 Design lifetime

Offshore Wind projects in Europe feature a design operational lifetime for the substructure that usually ranges around 25 years. This period is divided into operational and non-operational windows, according to environmental conditions on-site and availability of the turbine for production.

Furthermore, a period of 6-12 months shall be considered for the installation of foundations. Likewise, an additional idling time of up to 6 month shall be considered for a time period without grid connection after WTG installation and the time before commissioning, so as further 6 months after the operational lifetime for decommissioning services.

During the time period without WTG installed, wave loads are acting on the substructure only. Wind loads are relatively small. The resulting fatigue loading from wave loading can be assumed to be rather small or even negligible, due to the missing WTG mass (tower and RNA mass).

Therefore, a total design lifetime of 27 years is established for FOWPI, including 12 months for installation, 6 months for commissioning, and 6 months for decommissioning.

5.2 Environmental conditions

The following site conditions are covered in the following sections:

- Temperatures for steel selection
- Water depth and water levels
- Wind

- Waves
- Currents
- Splash zone
- Wave run-up
- Marine growth
- Air temperature
- Earthquakes
- Soil conditions
- Typhoons

The respective European standards and recommended practices to guide each condition are referred on the respective sections.

5.2.1 Temperatures for steel selection

In European projects, the steel material selection is carried out in accordance with the DNV GL standard (Ref. /1/), Section 4.2.

According to the code, the design temperature is defined as the lowest daily mean temperature, from which materials in structures above the LAT shall be designed. Furthermore, materials below the lowest astronomical tide do not need not be designed for service temperatures lower than 0°C.

5.2.2 Water depth and water levels

A preliminary bathymetric analysis has shown that the depths on the project site vary between 14 and 18 m LAT. Therefore, a depth of 16 m with relation to LAT has been adopted for the concept design accordingly.

The applicable water level is determined according to the parameters defined in the metocean report (Ref. /18/), Section 11, as well as the requirements from the European standard (Ref. /2/). From the determined Highest Astronomical Tide (4.12 m LAT), a sea level rise of 0.40m is considered for a 50-year return period. The positive storm surge in this case is dismissed for operational conditions due to the incidence of intense typhoon events predicted for the project site during the design lifetime, and therefore the highest sea water level (HSWL) is determined according to the extreme (Typhoon) conditions presented in the metocean study, Section 13 (Ref. /18/). The determined value for HSWL can therefore be found in Section 5.2.12 of this report.

The lowest sea water level (LSWL) is calculated based on the operational parameters, having therefore been considered a negative storm surge of 0.26m for the same 50-year return period. The determined value for LSWL can be found in Section 5.2.12 of this report.

For FLS, half of the sea level rise is considered, whereas for ULS, high still water level and the full sea level rise is to be taken into account. For ULS cases with low still water level, no sea level rise shall be applied.

5.2.3 Wind

Wind conditions are defined in the metocean report (Ref. /18/), Section 9. The rose plot of the wind speed at a height of +10mMSL (+12.11mLAT) is given in Figure 5-1.

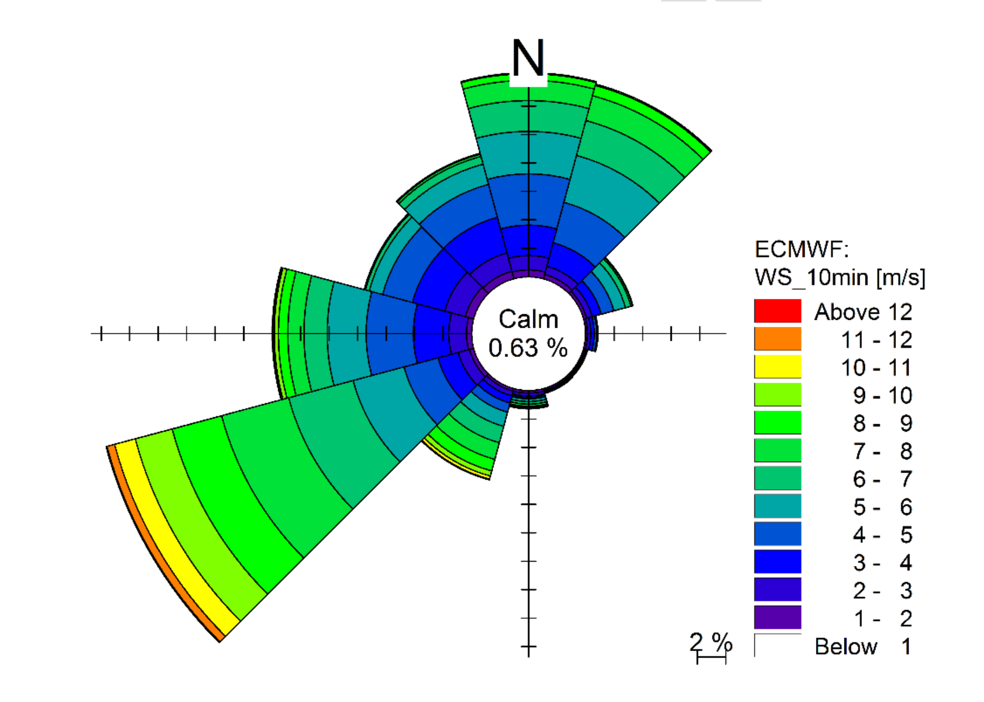


Figure 5-1 Rose plot of wind speed, U_{10}, 2010-2014.

The extreme value analysis for the same elevation is provided in Table 5-1.

Table 5-1 Extreme value analysis of wind speed, U_{10}, 2010-2014.

Wind Speed, U_{10} [m/s]	Return Period [Years]		
	1	5	10
Central estimate	12.6	13.9	14.4
Standard deviation	0.4	0.5	0.6

5.2.4 Waves

Wave conditions are defined in the metocean report (Ref. /18/), Section 10. The nearshore wave climate at three distinct locations within the proposed site in the form of wave rose plots of Hm0 and T_p for a 5-year period (2010-2014) are presented in Figure 5-2 and Figure 5-3.

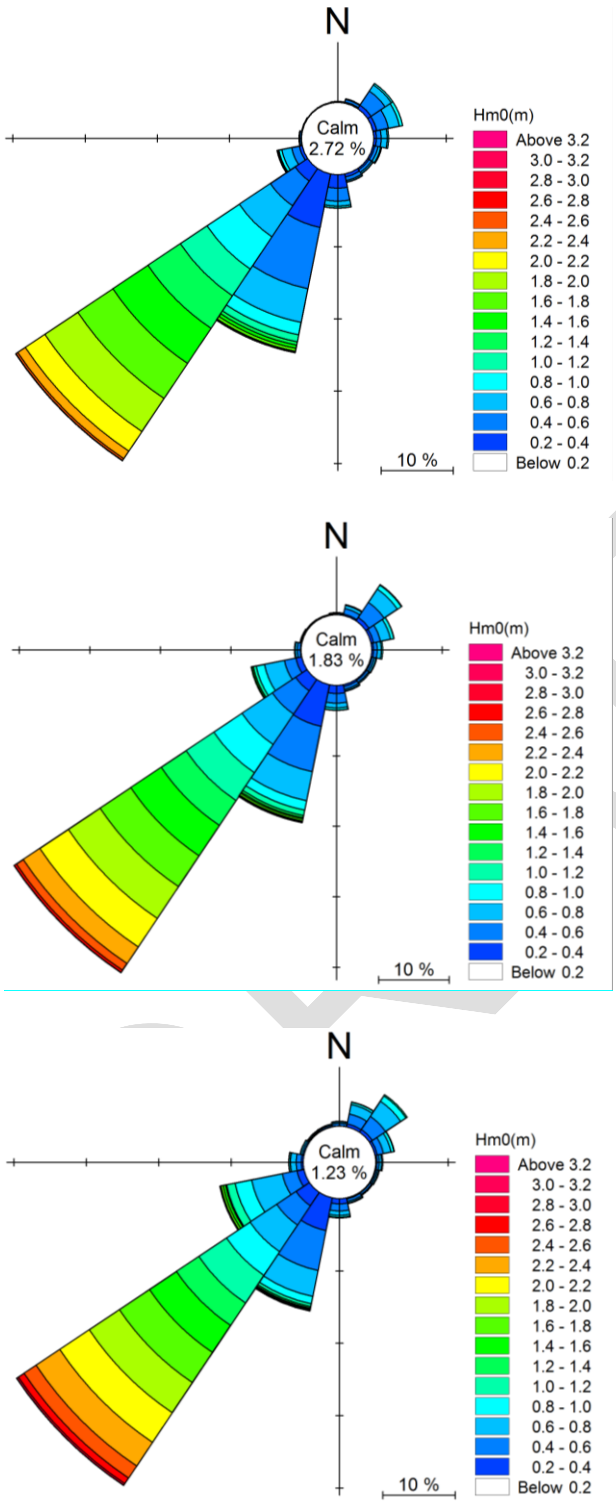


Figure 5-2 *Rose plots of significant wave height (H_m0) at three extraction points during 2010 to 2014; P1 (Top), P2 (Middle) and P3 (Bottom)*

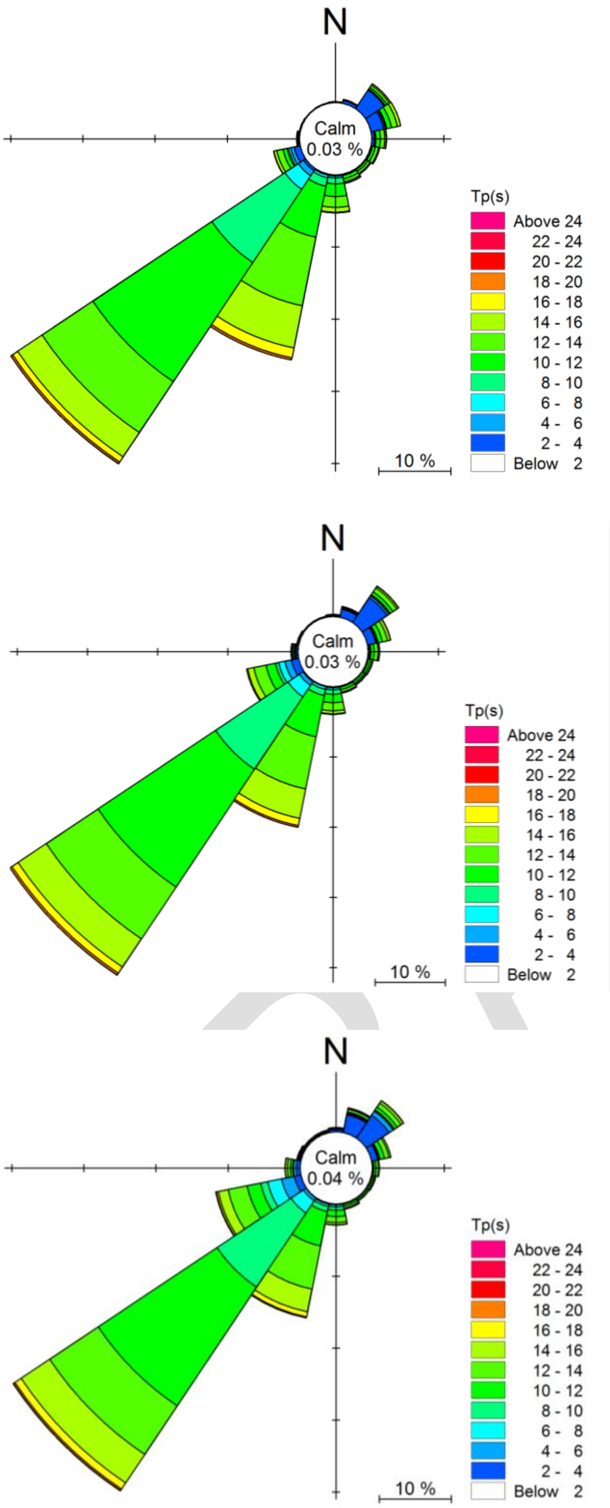


Figure 5-3 Rose diagram of peak wave period (T_p) at three extraction points during 2010 to 2014; P1 (Top), P2 (Middle) and P3 (Bottom)

The omni-directional extreme wave parameters calculated on basis of the recommended estimates of the extreme values of Hm0 are taken into account, i.e. for the calculation of fatigue loads. The corresponding values of the above presented analysis are presented in Table 5-2.

Table 5-2 Omni directional design wave parameters

Parameter	Return Period [Years]		
	1	5	10
H_{m0} [m]	2.7	3.1	3.2
H_{max} [m]	5.0	5.8	6.0
T_{Hmax} [s]	6.6	7.1	7.2
η_{max} [m]	3.0	3.6	3.8

For the foundation design, an extreme value analysis is carried out, on which the extreme wave height is determined for a 50-year return period, according to the European standard (Ref. /2/). For FOWPI concept design, however, this value is obtained from a typhoon study, also presented in the metocean report, as this event is not accounted for in operational conditions.

A brief introduction to the typhoon study results is presented in section 5.2.12.

5.2.5 Currents

Currents are considered according to Section 12 of the metocean report (Ref. /18/). The results of the extreme value analysis of total and residual current speeds are provided in Table 5-3 and Table 5-4.

Table 5-3 Results of extreme value analysis of Total Current Speed (notice that for design purposes it is recommended to add one standard deviation to the central estimates)

Total Current Speed [m/s]	Return Period [Years]		
	1	5	10
Central estimate	1.42	1.45	1.46
Standard deviation	0.01	0.01	0.02
Recommended value	1.43	1.46	1.48

Table 5-4 Results of extreme value analysis of Residual Current Speed (notice that for design purposes it is recommended to add one standard deviation to the central estimates)

Residual Current Speed [m/s]	Return Period [Years]		
	1	5	10
Central estimate	0.14	0.17	0.19
Standard deviation	0.01	0.02	0.02
Recommended value	0.15	0.19	0.21

5.2.6 Splash zone

According to the European standards (Ref. /1/ and Ref. /5/), the splash zone is the part of a support structure which is intermittently exposed to seawater due to the action of tides, waves, or both. Due to this action, the corrosive environment is severe, and maintenance of corrosion protection is not practical.

The upper bound of the splash zone is defined by the highest still water level for a 1-year return period, increased by the crest of the significant wave height for the same return period. An installation tolerance of 0.10 m at the interface level is also added. Based on the metocean data (Ref. /18/), this level is established at +7.00 mLAT.

The lower bound is defined by the lowest still water level for a 1-year return period, reduced by the trough depth of the significant wave height for the same return period. The same installation tolerance from the upper bound is considered, this time downwards, and therefore this level is defined at -1.50 mLAT.

5.2.7 Wave run-up

The impact of waves against the foundation leads to the so called wave run-up effect, on which the seawater splashes vertically upwards. In extreme cases, it can even reach the service platform on top of the transition piece, or theoretically even higher levels in events such as the extreme design wave.

The level of the service platform is determined in a way that the extreme design wave height still leaves an air gap of at least 1.50m from it in its occurrence. Nonetheless, there are further considerations to be accounted for in order to limit the effect of the wave run-up on the external platforms. Between these measures, a common practice is to design the platforms facing the opposite direction of the one from the extreme wave, in a way that it does not cause direct impacts on them. For instance, in FOWPI, according to the metocean study (Ref. /18/), the proposed OWF site is primarily exposed to waves from SW (225°), due to the exposure of the site during the southwest monsoon, whereas the northeast monsoon has a minor effect due to the limited fetch towards NE. In this way, a reasonable strategy would be to design the platforms facing NE, so as to avoid the run-up effect to reach those.

Furthermore, additional structural parts on the foundation structure itself are also viable. In this sense, the addition of appurtenances that guide the seawater from the run-up effect away from the platforms, the installation of replaceable gratings, or even designing a concrete platform, are common measures adopted in European projects.

5.2.8 Marine growth

Warm water conditions combined with several nutrients tend to generate extensive marine growth on the outer surface of the foundations. The rate of such growth varies from one region to another. Typically, about 0.1m can be expected between the mudline and the mean sea level, and up to 0.3m in the splash zone, according to Ref. /12/, and as provided in Table 5-5.

Unless more accurate data is available, the density of the marine growth may be set equal to 1325 kg/m³. This density is then used to determine the loads from that on the foundation structure.

Table 5-5 Marine growth thickness and elevation for primary steel

Level w.r.t. LAT	Thickness	Attribute
[m]	[mm]	[-]
above 7.00	0	smooth
7.00 to -1.50	300	rough
below -1.50	100	rough

5.2.9 Air temperature

Temperatures can be extreme in the Gulf of Khambhat, where the project site is located, reaching down to 8.4°C during January and up to 43.7°C during May (Ref. /18/). Statistically, due to its equatorial location, Gujarat can encounter temperatures varying between 23 and 33°C in average, according to Ref. /12/.

5.2.10 Earthquakes

Gujarat is located near two plate boundaries, therefore featuring a high potential for seismic activity. Figure 5-4 provides an overview of the earthquake risk zones in Gujarat according to their hazard levels (Ref. /20/). The proposed site location is accordingly situated within an area under moderate risks of earthquake damages.

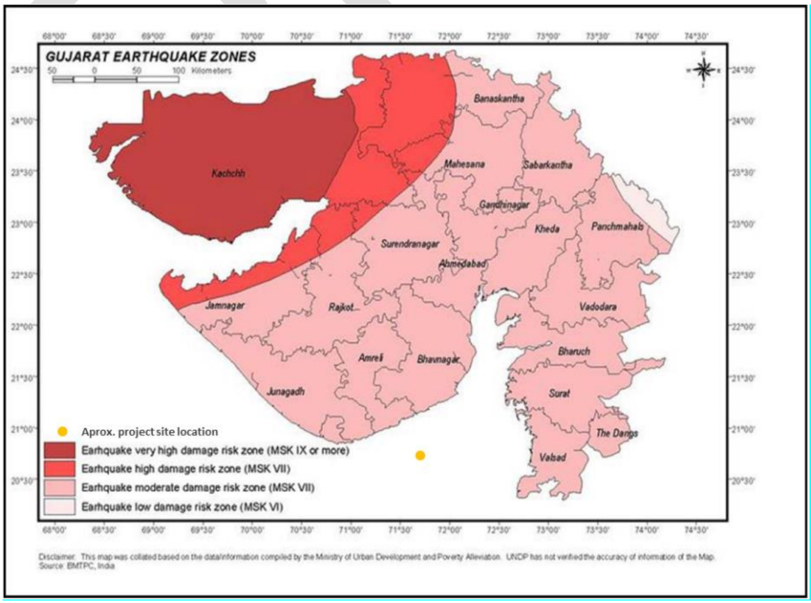


Figure 5-4 Earthquake Hazard Risk Zonation: Seismo-techtonic features of Gujarat

The consideration of earthquake requirements shall be carried out in accordance with the latest version of IEC 61400-1 (Ref. /10/), as well as the Guideline for the Certification of Offshore Wind Turbines (Ref. /9/).

For this conceptual design, the impact of earthquake events is not verified in detail. Nevertheless, in order to address such conditions, an additional embedment length of 1 time the bottom diameter of the monopile is considered in this concept design.

5.2.11 Soil conditions

In general, for each WTG location, a specific soil design profile is developed. Nevertheless, a single profile is established for FOWPI's primary design, according to the provided geotechnical report (Ref. /20/). A summary of the main characteristics from the soil at the project site is provided in Table 5-6. A schematic layout is further presented in Figure 5-5.

Table 5-6 Soil profile features at the project site

Soil profile					
Top of layer	Soil type	Consistence	γ'	φ	c_u
[m below sb]	-	-	[kN/m^3]	[°]	[kPa]
0.00	Clay	Very soft	6		3
2.50	Clay	Very soft	6		6.2
5.00	Clay	Very soft	6		6.8
7.50	Clay	Very soft	6		7.2
9.50	Sand	Very dense	10	41	
10.50	Clay	Stiff	10.5		111
11.50	Sand	Very dense	10	35	
14.00	Sand	Very dense	10	41	
15.50	Sand	Very dense	10	37	
20.00	Sand	Very dense	10	38	
21.50	Sand	Very dense	10	37	
23.00	Sand	Very dense	10	39	
24.50	Sand	Very dense	10	36	
26.00	Sand	Very dense	10	41	
27.50	Sand	Very dense	10	37	
29.00	Sand	Very dense	10	41	

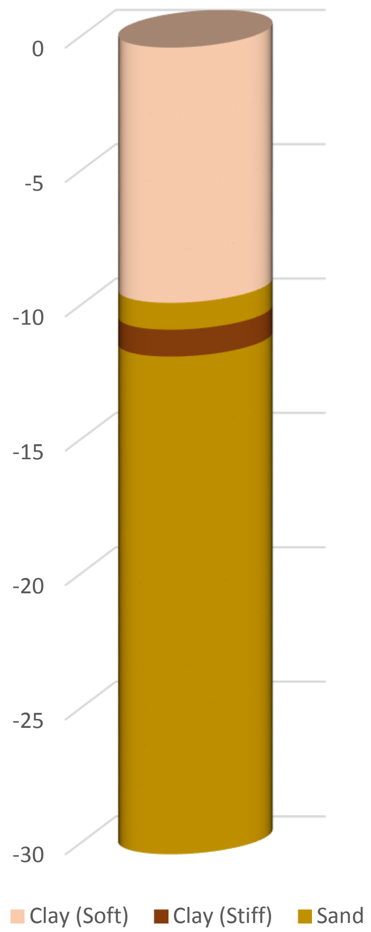


Figure 5-5 Schematic view of soil profile at the project site (0m at mudline)

The defined soil profile will be applied for the concept design. A pile drivability assessment is not executed by COWI due to the limited data available. However, the impact of pile driving on the design is accounted for in this concept design. For the fatigue design checks, a damage of 0.1 due to pile driving is included. This value is conservative based on European experiences.

5.2.12 Typhoons

Typhoon conditions are defined in the metocean report (Ref. /18/), Section 13.

Data from Ref. /19/ reveal that the west coast of India has been struck by 27 cyclones during the period of 1975 to 2015. The metocean conditions during cyclones exceed the conditions caused by monsoons and tropical storms, and the random nature of the cyclone tracks in the region statistically means that the project site will inevitably experience the full-blown impact of a cyclone sometime in the future.

In this regard, a storm surge of 3.26m and an extreme significant wave height of 9.5m are expected for a 50-year return period. From the determined HSWL (+7.80mLAT), the crest from the extreme wave height shall be added in order to determine the highest level reached by the seawater during that same return period (without accounting for the run-up effects). This value is calculated based on Ref. /9/, Section 4.2.3.3, and corresponds to approximately 78% of the wave height, or around 13.80m in this case. The highest level would be, therefore, +21.60m LAT. A summary of the main values is provided in Table 5-7.

Table 5-7 Extreme values for typhoon conditions

Return period (years)	50
Highest sea water level (m LAT)	+ 7.80
Storm surge (m)	3.26
Significant wave height Hm0 (m)	9.5
Extreme (design) wave height (m)	17.7
Extreme (design) wave crest (m)	13.8
Highest wave level (m LAT)	+ 21.60

This last calculation is fundamental for determining the level of the external working platform and consequently the interface level between the tower and the transition piece. As previously mentioned in Section 5.2.7, an air gap of at least 1.50 m is required between the lowest level of the working platform and the highest level reached by the water.

5.3 Functional requirements

The layout of the foundation components is defined in accordance with the European standards and guidelines (Ref. /1/ and Ref. /2/). Nonetheless, WTG suppliers may have different specific requirements which depend on the O&M concept, meaning that the design may vary whether the operation and maintenance is provided by the supplier himself, by the wind farm owner, or by a third party.

Regarding the boat landing structure dimensions, the guiding factor on determining the levels of the upper and lower bounds is the splash zone limits, as well as the dimensions of typical maintenance vessels (bigger ones for the upper bound, and small ones for the lower bound). Furthermore, additional verifications include the positioning of the boat landing supports on the TP walls in a way that these do not match with the welding between the TP shells. In this regard, the upper and lower bounds of the boat landing for FOWPI are defined as +13.00m LAT and -2.00m LAT, respectively.

Furthermore, the interface level between the tower and the transition piece, and consequently the level of the external working platform are determined according to the environmental conditions on the project site, previously described in Section 5.2. Under this consideration, the bottom of the external working platform is defined at +23.30m LAT and assuming a platform height of 0.5 m the top of the external working platform (top of grating) is at 23.80m LAT. The interface level is defined at +24.00m LAT for FOWPI.

5.4 Materials

A description of the requirements for the materials to be used in the foundations fabrication is provided in the following subsections. The proposed concept design takes as basis the European standards for the materials selection, and therefore its optimal functionality is also subject to the usage of the specifications here mentioned.

5.4.1 Primary steel

The material properties of the steel plates for the primary steel structure follow the guidelines of EN 10025 (Ref. /13/, Ref. /14/ and Ref. /15/). The steel quality for the shells of the MP and TP is either S355 NL/ML, S420 NL/ML or S460 NL/ML, as respectively presented in Table 5-8.

Table 5-8 Applied steel yield strength f_{yk} (without safety factor)

Material	Yield strength subject to thickness in mm [MPa]					
	≤ 16	]16-40]	]40-63]	]63-80]	]80-100]	]100-150]
S355 NL/ML	355	345	335	325	315	295
S420 NL/ML	420	400	390	370	360	340
S460 NL/ML	460	440	430	410	400	380

Note: the stated values were determined from a combination of parts 3 and 4 of the above mentioned code.

5.4.2 Secondary steel

Generally for secondary steel, components are selected according to EN 10025 (Ref. /13/Ref. /14/Ref. /15/). COWI is, however, not designing such elements for the concept design of FOWPI foundations.

5.4.3 Grout

The material properties to be used for the grouted connection between TP and MP is defined for the grout and skirt verification. For the purpose of the lumped mass determination as input to the frequency checks, a density of 2.4 t/m³ is used. A young´s modulus of 50.900 MPa is applied to consider the stiffness contribution of the grout layer between TP and MP.

5.4.4 Stainless steel

Generally, steel types for stainless steel are selected according to EN 10088 (Ref. /16/). COWI is, however, not executing such verifications for the concept design of FOWPI foundations.

5.5 Wind turbines

In order to develop the primary concept design for FOWPI, two distinct reference models of WTG were considered for setting up the tower geometry and their relative design loads applied on the foundations, being one of 3MW and one of 6MW capacity (Ref. /22/). The main features of the two turbines are presented in Table 5-9.

Table 5-9 Main wind turbine parameters

Main turbine data		
Reference Model	3 MW	6 MW
Rotor diameter (m)	112	154
Swept area (m²)	9852	18600
Hub height (m LAT)	86	107
Tower top - Hub height (m)	Approx. 1.90	Approx. 2.20
TP-Tower interface (m LAT)	24	24
Blades	3	3
RNA mass (tonnes)	Approx. 200	Approx. 410
Wind class IEC	IB	IA
Nominal power (kW)	3000	6000
Frequency (Hz)	50 / 60	50
Cut-in wind speed (m/s)	3	4
Rated wind speed (m/s)	12.5	13.5
Cut-Out wind speed (m/s)	25	25

The tower geometry for each WTG is a project-specific element. In this case, a preliminary tower design was made according to the specific characteristics of FOWPI regarding the hub height and tower-TP interface level.

5.6 Design loads

Design loads are derived for the two selected turbines based on the site-specific conditions, and the determined geometries for ULS and FLS conditions. For FLS, damage-equivalent loads are derived and applied in the design checks.

5.7 Grouted connection

Generally, there are specific verifications developed according to Ref. /1/, Section 6, in order to define the main properties of the grouted connection. Previous experiences have shown that a grouted connection length of 1.5 times the diameter of the TP provides enough resistance. For both configurations, a conical grouted connection with a length of 9.5m is considered for FOWPI on the safe side.

6 Foundation design

The main technical aspects of the concept design for FOWPI are covered in the following subsections. The applied design methodologies and the main results of the design checks are provided.

6.1 Design process

No load iteration process with the WTG supplier is carried out for this concept design. The design loads are derived based on the site-specific conditions and the initial geometry of the foundations. On this basis the concept designs are finalised and all relevant design checks are carried out.

6.1.1 Design software

The design software applied for the design of the primary structure of the monopile foundation are listed in the following:

- COPILOD: COWI in-house design software
  Design calculations of the primary structure of the monopile foundation including stress distribution, code checks and natural frequency analysis.

- COSPIN: for geotechnical design checks and for the determination of the non-linear springs for the soil structure interaction.

6.1.2 Structural model

A three-dimensional finite element model representing the foundation structure including monopile, transition piece, tower and RNA is established.

Modelling of the nacelle and rotor is done by discrete masses and mass moments of inertia at centre of gravity of the RNA.

The model incorporates pile-soil interaction in terms of P-y, T-z and Q-z curves for modelling of lateral and vertical soil-structure interaction.

6.1.3 Soil modelling

Two distinct methods for analysing the soil-structure interaction are applied for different design checks. For the geotechnical design checks and determination of the distribution of sectional forces in the subsoil non-linear soil springs are used. For the frequency checks, the non-linear soil springs have to be linearized. The initial stiffness method is used for this linearization.

6.2 Ultimate limit state (ULS)

The ultimate limit state corresponds to a maximum load-carrying resistance. The features and checks presented in the following subsections are considered accordingly.

6.2.1 Modelling

The basis for the ULS models is presented in Table 6-1 in accordance with (Ref. /2/).

Table 6-1 Basis for ULS modelling.

Item	ULS
Soil-structure interaction properties	Non-linear
Soil properties	Lower bound
Corrosion allowance	Fully corroded
Water level	Design high water level (HSWL)
Marine growth	Fully developed
Scour	Scour not considered
Structural analysis	Linear elastic
Load input format	Distributed static loads
Environmental conditions (wind, wave and current)	Embedded in distributed loads

6.2.2 Additional moments from inclination of structure

The ULS bending moments are increased by 10% on the safe side in order to account for second order effects in the ULS checks.

6.2.3 Design check equivalent stresses

Design checks based on equivalent stresses are carried out for all levels with a circumferential weld.

6.2.4 Buckling

The buckling checks are done according to Ref. /3/.

The buckling length which is applied in the buckling checks for the TP is the distance between interface level (+24.00m LAT) and top of MP (+6.00m LAT), i.e. the length is 18.00m for the TP.

The buckling length which is applied in the buckling checks for the MP is the distance between top of MP (+6.00m LAT) and 7.00m (6MW turbine) or 5.50m (3MW turbine), corresponding to one pile diameter long, below local sea floor level.

The buckling checks are done for every level of the TP and MP with a circumferential weld.

6.3 Fatigue Limit State (FLS)

The Fatigue Limit State corresponds to a failure due to the effect of cyclic loading. The considerations and checks presented in the following subsections are concerned accordingly.

6.3.1 Modelling

The basis for the FLS models will be in accordance with Table 6-2.

Table 6-2 Basis for FLS modelling.

Item	FLS
Soil-structure interaction properties	Non-linear
Soil properties	Best estimate
Corrosion allowance	Half corroded
Water level	MSL
Marine growth	Fully developed
Scour	Scour not considered
Structural analysis	Linear elastic
Load input format	Distributed damage equivalent loads
Environmental conditions (wind, wave and current)	Embedded in distributed loads

6.3.2 Pile driving fatigue

The pile driving fatigue is accounted for by adding 10% damage to the total fatigue damage.

6.3.3 Stress concentration factors

The SCF at circumferential welds and at conical transitions are calculated based on formulas from Ref. /4/.

6.3.4 SN curves

For practical fatigue design, welded joints are divided into several classes, each with a corresponding design S-N curve.

The basic design S-N curve is given as:

$$\mathrm{Log}_{10}(N) = \mathrm{Log}_{10}(\bar{a}) - m\,\mathrm{Log}_{10}(\Delta\sigma)$$

Where:

N = predicted number of cycles to failure for stress range $\Delta\sigma$
$\Delta\sigma$ = stress range (MPa)
m = negative inverse slope of S-N curve
$\mathrm{Log}_{10}(\bar{a})$ = intercept of log N-axis by S-N curve

The thickness effect is taken into account applying:

$$\mathrm{Log}_{10}(N) = \mathrm{Log}_{10}(\bar{a}) - m\ \mathrm{Log}_{10}[\Delta\sigma(t/t_{ref})^k]$$

Where:

t_{ref} = reference thickness (25mm)
t = thickness through which a crack will most likely grow
k = thickness exponent on fatigue strength

The S-N curves are based on Ref. /4/. The primary structure is mainly checked using S-N curve D for both conditions (in air and in seawater with cathodic protection). Grinded circumferential welds are checked against the S-N curve C1. Fatigue life calculations have been carried out based on corroded members (half corrosion allowance). S-N curves for free corrosion are used in the splash zone after coating life is exceeded (assumed to be after 15 years).

Fatigue life can be increased by grinding (if required) to produce a flush transition between the cans. The grinding procedure should ensure that all defects in the weld are removed.

Longitudinal welds have not at present been checked for fatigue, but are normally not governing for design.

6.3.5 Design fatigue factor

The substructure is assumed designed as "not accessible for inspection" requiring by Ref. /7/:

- Atmospheric zone (above respective splash zone):
  - DFF = 2.0
- All other parts of substructure:
  - DFF = 3.0

6.4 Analysis results

The foundation structure investigated in this study is a traditional monopile with a conical upper part and variable wall thicknesses supporting TP, tower and WTG. The TP and MP outer surface is flush, i.e. wall thickness steps are taken on the inside.

The substructure comprises a monopile from pile toe to top elevation +6.00m LAT and a transition piece from -4.00m LAT to the interface with the WTG tower at elevation +24.00m LAT. The transition piece is assumed grouted on the monopile with a conical grouted connection. Further specific properties are presented in the following subsection.

6.4.1 Overall dimensions and weights

The weight and length of the investigated monopile foundation cases are presented in Table 6-3 .

Table 6-3 Basic result table for weights and lengths of monopile foundations.

Turbine model	[-]	3MW	6MW
Weight of TP	[MT]	191.5	306.4
Length of TP	[m]	28	28
Top diameter of TP	[m]	4.5	6.0
Bottom diameter of TP	[m]	5.5	7.0
Wall thickness of TP	[mm]	55-65	60-90
Weight of MP	[MT]	529	874
Length of MP	[m]	57.6	63.0
Embedment length of MP	[m]	35.6	41.0
Top diameter of MP	[m]	4.4	5.9
Bottom diameter of MP	[m]	5.5	7.0
Wall thickness of MP	[mm]	55-80	60-100
Length of cylindrical section of MP	[m]	41.85	47.25
Length of conical section of MP	[m]	15.75	15.75

6.4.2 Ultimate limit state (ULS)

The ULS utilization ratios (UR) stays below 1 for all design checks.

6.4.3 Cable hole

The cable holes are not investigated in full detail at this stage of design.

The cable holes are assumed not to cause a need for increased wall thicknesses regarding ULS design checks. Local reinforcements of the cable holes will be applied if needed. Nevertheless, the steel grade is increased to S420 NL/ML for the can with the cable hole.

The required minimum angle between cable holes have to be determined with a detailed analysis. Generally, a minimum angle of 30 degrees should be sufficient considering the expected size of the cable hole of 340mm, considering two cable holes. In case that there are more than two cable holes per foundation, the angle between them should be 45 degrees as a minimum.

6.4.4 Fatigue limit state (FLS)

The results of the fatigue checks for the foundation for the 3MW reference turbine are shown in Table 6-4 to Table 6-7.

Table 6-4 Fatigue check results for outside of MP for 3MW reference turbine

MP Outside				
Elevation [mLAT]	Curve	Dam DEL	Dam PDA	DLife [y]
2.5	D-protected	0.189	0.1	90.22
-1	D-protected	0.165	0.1	98.34
-2.75	D-protected	0.207	0.1	84.63
-6.25	D-protected	0.181	0.1	92.72
-9.75	D-protected	0.446	0.1	47.17
-11.75	D-protected	0.168	0.1	97.37
-15.95	D-protected	0.139	0.1	109.45
-20.15	D-protected	0.324	0.1	60.94
-24.35	D-protected	0.479	0.1	44.53
-28.55	D-protected	0.697	0.1	32.23
-32.75	D-protected	0.386	0.1	53.16
-36.95	D-protected	0.069	0.1	156.1
-41.15	D-protected	0.002	0.1	264.21
-45.35	D-protected	0	0.1	270.99
-49.55	D-protected	0	0.1	271

Table 6-5 Fatigue check results for inside of MP for 3MW reference turbine

MP Inside				
Elevation [mLAT]	Curve	Dam DEL	Dam PDA	DLife [y]
2.5	D-air+free	0.515	0.1	41.87
-1	D-protected	0.249	0.1	74.35
-2.75	D-protected	0.33	0.1	60.13
-6.25	D-protected	0.29	0.1	66.42
-9.75	D-protected	0.4	0.1	51.58
-11.75	D-protected	0.653	0.1	34.13
-15.95	D-protected	0.188	0.1	90.5
-20.15	D-protected	0.438	0.1	47.9
-24.35	D-protected	0.413	0.1	50.3
-28.55	D-protected	0.601	0.1	36.66
-32.75	D-protected	0.333	0.1	59.75
-36.95	D-protected	0.139	0.1	109.52
-41.15	D-protected	0.005	0.1	256.05
-45.35	D-protected	0	0.1	270.98
-49.55	D-protected	0	0.1	271

Table 6-6 Fatigue check results for outside of TP for 3MW reference turbine

TP Outside				
Elevation [mLAT]	Curve	Dam DEL	Dam PDA	DLife [y]
23.8	D-air	0.085	0	300.95
20.8	D-air	0.108	0	235.52
17.3	D-air	0.142	0	179.51
13.8	D-air	0.185	0	137.62
10.3	D-air	0.785	0	32.48
7.7	D-air	0.191	0	133.36
5.2	D-air+free	0.132	0	193.3
3	D-air+free	0.159	0	160.49
-0.4	D-air+free	0.125	0	203.85

Table 6-7 Fatigue check results for inside of TP for 3MW reference turbine

TP Inside				
Elevation [mLAT]	Curve	Dam DEL	Dam PDA	DLife [y]
23.8	D-air	0.075	0	340.59
20.8	D-air	0.096	0	266.54
17.3	D-air	0.126	0	203.15
13.8	D-air	0.164	0	155.75
10.3	D-air	0.694	0	36.76
7.7	D-air	0.301	0	84.58
5.2	D-air	0.199	0	127.91
3	D-protected	0.385	0	66.2
-0.4	D-protected	0.113	0	225.53

The results of the fatigue checks for the foundation for the 6MW reference turbine are shown in Table 6-8 and Table 6-11.

Table 6-8 Fatigue check results for outside of MP for 6MW reference turbine

MP Outside				
Elevation [mLAT]	Curve	Dam DEL	Dam PDA	DLife [y]
2.5	D-protected	0.215	0.1	82.61
-1	D-protected	0.256	0.1	72.84
-2.75	D-protected	0.305	0.1	63.89
-6.25	D-protected	0.255	0.1	73.01
-9.75	D-protected	0.635	0.1	34.98
-11.75	D-protected	0.157	0.1	101.72
-15.95	D-protected	0.186	0.1	90.9
-20.15	D-protected	0.437	0.1	48.01
-23.95	D-protected	0.523	0.1	41.34
-27.65	D-protected	0.683	0.1	32.83
-31.35	D-protected	0.727	0.1	31.07
-35.05	D-protected	0.453	0.1	46.6
-38.85	D-protected	0.221	0.1	80.95
-43.05	D-protected	0.028	0.1	208.24
-47.25	D-protected	0	0.1	269.69
-51.45	D-protected	0	0.1	271
-55	D-protected	0	0.1	271

Table 6-9 Fatigue check results for inside of MP for 6MW reference turbine

MP Inside				
Elevation [mLAT]	Curve	Dam DEL	Dam PDA	DLife [y]
2.5	D-air+free	0.562	0.1	38.86
-1	D-protected	0.498	0.1	43.07
-2.75	D-protected	0.437	0.1	47.99
-6.25	D-protected	0.367	0.1	55.26
-9.75	D-protected	0.57	0.1	38.39
-11.75	D-protected	0.294	0.1	65.68
-15.95	D-protected	0.245	0.1	75.31
-20.15	D-protected	0.786	0.1	28.98
-23.95	D-protected	0.65	0.1	34.27
-27.65	D-protected	0.591	0.1	37.24
-31.35	D-protected	0.629	0.1	35.28
-35.05	D-protected	0.563	0.1	38.79
-38.85	D-protected	0.717	0.1	31.42
-43.05	D-protected	0.091	0.1	137.96
-47.25	D-protected	0	0.1	269.8
-51.45	D-protected	0	0.1	271
-55	D-protected	0	0.1	271

Table 6-10 Fatigue check results for outside of TP for 6MW reference turbine

TP Outside				
Elevation [mLAT]	Curve	Dam DEL	Dam PDA	DLife [y]
23.8	D-air	0.233	0	109.51
21	D-air	0.284	0	89.92
18.8	D-air	0.329	0	77.42
15.9	D-air	0.293	0	87.08
13.1	D-air	0.349	0	73.07
10.3	D-air	0.777	0	32.83
7.7	D-air	0.205	0	124.13
5.2	D-air+free	0.159	0	160.14
3	D-air+free	0.671	0	38.01
-0.4	D-air+free	0.602	0	42.36

Table 6-11 Fatigue check results for inside of TP for 6MW reference turbine

TP Inside				
Elevation [mLAT]	Curve	Dam DEL	Dam PDA	DLife [y]
23.8	D-air	0.21	0	121.15
21	D-air	0.256	0	99.47
18.8	D-air	0.509	0	50.14
15.9	D-air	0.262	0	97.15
13.1	D-air	0.752	0	33.92
10.3	D-air	0.684	0	37.27
7.7	D-air	0.282	0	90.44
5.2	D-air	0.296	0	86.2
3	D-protected	0.837	0	30.45
-0.4	D-protected	0.187	0	136.05

The calculated minimum fatigue life for both models for TP and MP is not less than 27 years.

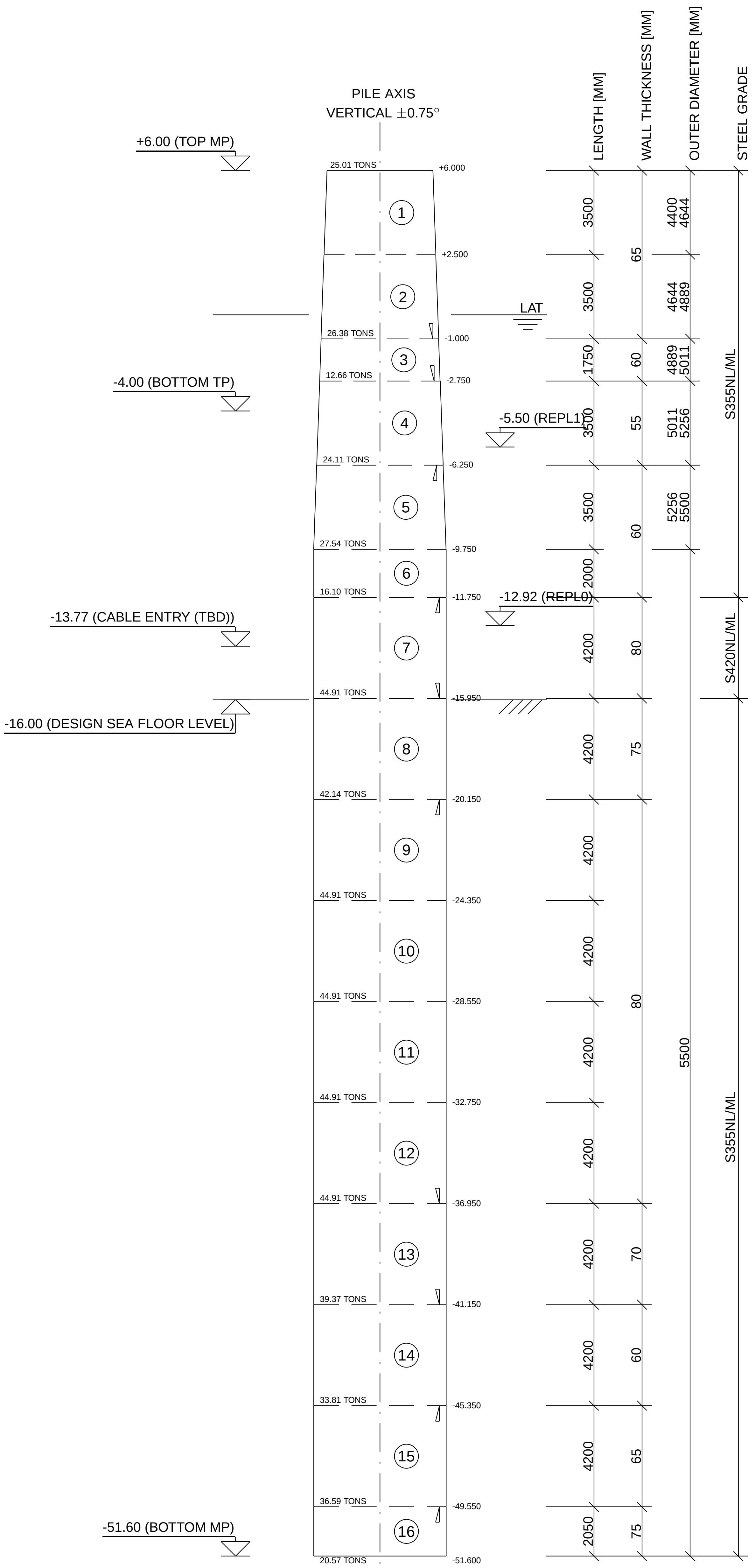
6.4.5 Natural frequency

The first natural frequencies for the complete foundation structures are shown in Table 6-12. The calculated natural frequencies are at a typical level for such monopile foundations and well above the expected lower bound frequencies as required by the WTG supplier.

Table 6-12 First natural frequencies for the complete foundation structures.

Turbine Model	[-]	3 MW	6 MW
Frequency (ULS model/LB)	[Hz]	0.294	0.245
Frequency (FLS model/BE)	[Hz]	0.295	0.247
Frequency (Upper Bound/UB)	[Hz]	0.298	0.248

Appendix A MP and TP dimensions for 3MW reference wind turbine



NOMINAL WEIGHT	[MT]	528.8
COG FROM TOP MP	[M]	30.00
NUMBER OF CANS	[-]	16
TOTAL PILE LENGTH	[M]	57.60
DESIGN EMBEDDED PILE LENGTH	[M]	35.60 -0.00/+0.10

LEGEND:

CIRCUMFERENTIAL WELDS:

NOT GROUND.

GROUND FLUSH TO SURFACE ON THE INSIDE.

GROUND FLUSH TO SURFACE ON THE OUTSIDE.

GROUND FLUSH TO SURFACE BOTH ON THE INSIDE AND THE OUTSIDE.

TAPERING

NOTES:

1. ALL DIMENSIONS ARE IN MM.
2. ALL LEVELS ARE IN M RELATIVE TO LAT.
3. CIRCUMFERENTIAL AND LONGITUDINAL WELDS SHALL BE FLUSH GROUND, IF GRINDING IS NECESSARY.
4. ALL WELDS SHALL BE FULL PENETRATION WELDS WELDED FROM BOTH SIDES.
5. CRUCIFORM WELDS ARE NOT ALLOWED.
6. THE MINIMUM ANGLE BETWEEN LONGITUDINAL WELDS SHALL BE AT LEAST 30 DEGREES FOR ADJACENT CANS.
7. ATTACHMENTS ARE NOT ALLOWED UNLESS NOTED OTHERWISE ON THE DESIGN DRAWINGS.
8. BEVELLING BETWEEN CANS OF DIFFERENT WALL THICKNESS SHALL BE 1:4 IF THICKNESS STEP IS LARGER THAN 4 MM. IF THE THICKNESS STEP IS NOT LARGER THAN 4 MM, THE THICKNESS STEP IS EQUALIZED WITHIN THE WELD.
9. THICKNESS STEPS BETWEEN CANS ARE ON THE INSIDE. THE OUTER SURFACE IS FLUSH.
10. THE PILE TOP SURFACE SHALL BE FABRICATED IN SUCH A WAY, THAT IT IS TRUE PERPENDICULAR TO THE PILE AXIS.
11. REPL MEANS REPLENISHMENT HOLE. REPLENISHMENT HOLES SHALL HAVE AN ELLIPTICAL SHAPE.

Rev.	Date	Description	Designed	Checked	Approved

Client

FOWPI
FIRST OFFSHORE WIND PROJECT OF INDIA

FOWPI First Offshore Wind Project of India

CONCEPT DESIGN
MONOPILE DIMENSIONS (3MW TURBINE)

Description

Designed	GAQI
Drawn	GAQI
Checked	MNKM
Approved	PEVL
Scale	1:200 (for A3)
Date	2017-11-30

COWI
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Drawing No.	Rev.
A073635-DWG-019-012	1.0

DWG generated 2017-12-19 by script create-drawing-mp.pl, version 2017-11-15 using configuration 00 (structural revision 00) in database INDB.

NOMINAL WEIGHT	[MT]	191.5
COG FROM TOP TP	[M]	14.88
NUMBER OF CANS	[-]	11
TOTAL TP LENGTH	[M]	28.00

LEGEND:

CIRCUMFERENTIAL WELDS:

- NOT GROUND.
- GROUND FLUSH TO SURFACE ON THE INSIDE.
- GROUND FLUSH TO SURFACE ON THE OUTSIDE.
- GROUND FLUSH TO SURFACE BOTH ON THE INSIDE AND THE OUTSIDE.
- TAPERING

NOTES:

1. ALL DIMENSIONS ARE IN MM.
2. ALL LEVELS ARE IN M RELATIVE TO LAT.
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4. ALL WELDS SHALL BE FULL PENETRATION WELDS WELDED FROM BOTH SIDES.
5. CRUCIFORM WELDS ARE NOT ALLOWED.
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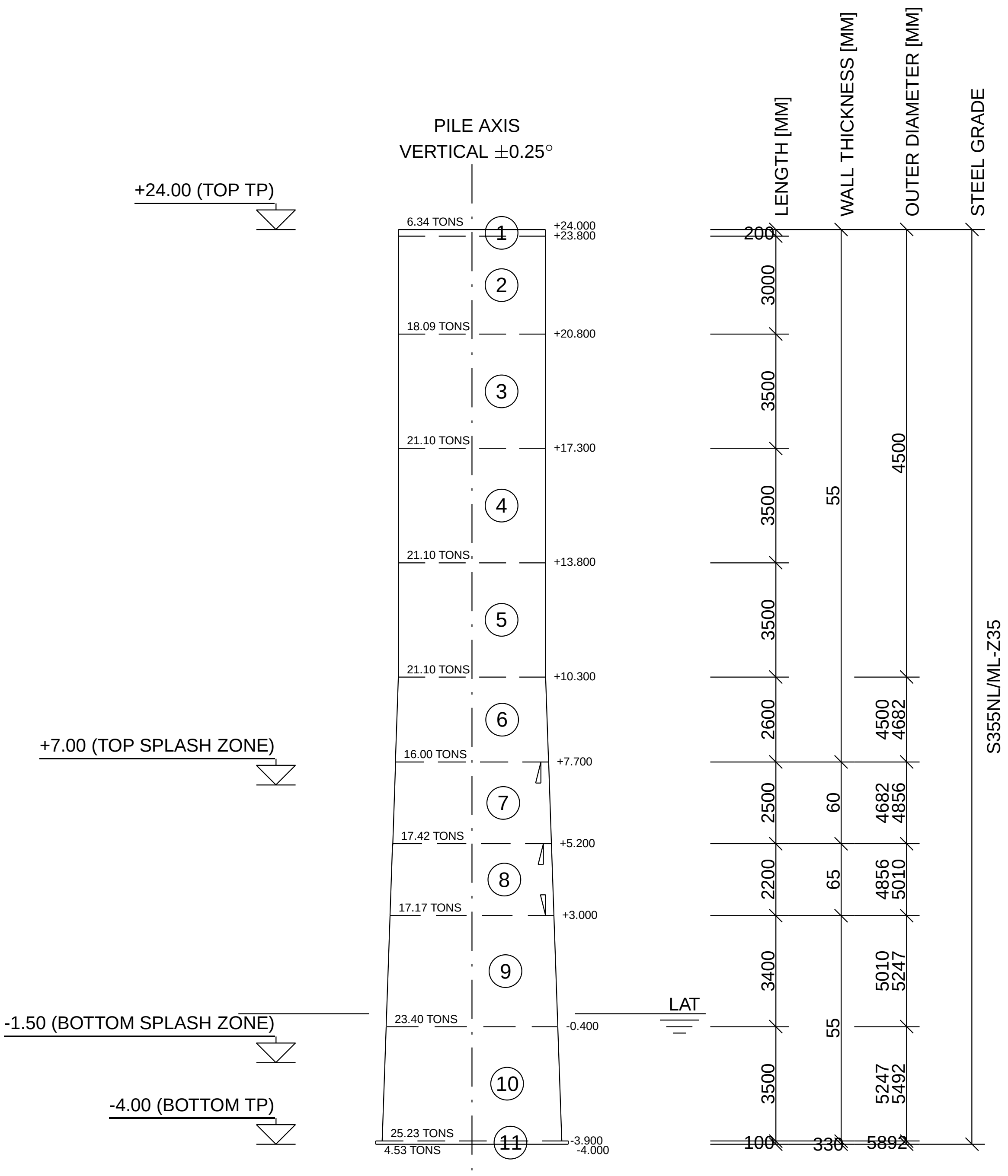
7. BEVELLING BETWEEN CANS OF DIFFERENT WALL THICKNESS SHALL BE 1:4 IF THICKNESS STEP IS LARGER THAN 4 MM. IF THE THICKNESS STEP IS NOT LARGER THAN 4 MM, THE THICKNESS STEP IS EQUALIZED WITHIN THE WELD.
8. THICKNESS STEPS BETWEEN CANS ARE ON THE INSIDE. THE OUTER SURFACE IS FLUSH.
9. THE WELD TOES OF SOME ATTACHMENTS TO THE TP MAY HAVE TO BE GROUND.



Rev.	Date	Description	Designed	Checked	Approved

Client

FOWPI
FIRST OFFSHORE WIND PROJECT OF INDIA

FOWPI First Offshore Wind Project of India

CONCEPT DESIGN
TRANSITIONPIECE DIMENSIONS
(TP2, 3MW TURBINE)

Description

Designed	GAQI
Drawn	GAQI
Checked	MNKM
Approved	PEVL
Scale	1:200 (for A3)
Date	2017-11-30
Drawing No.	Rev.
A073635-DWG-019-002	1.0

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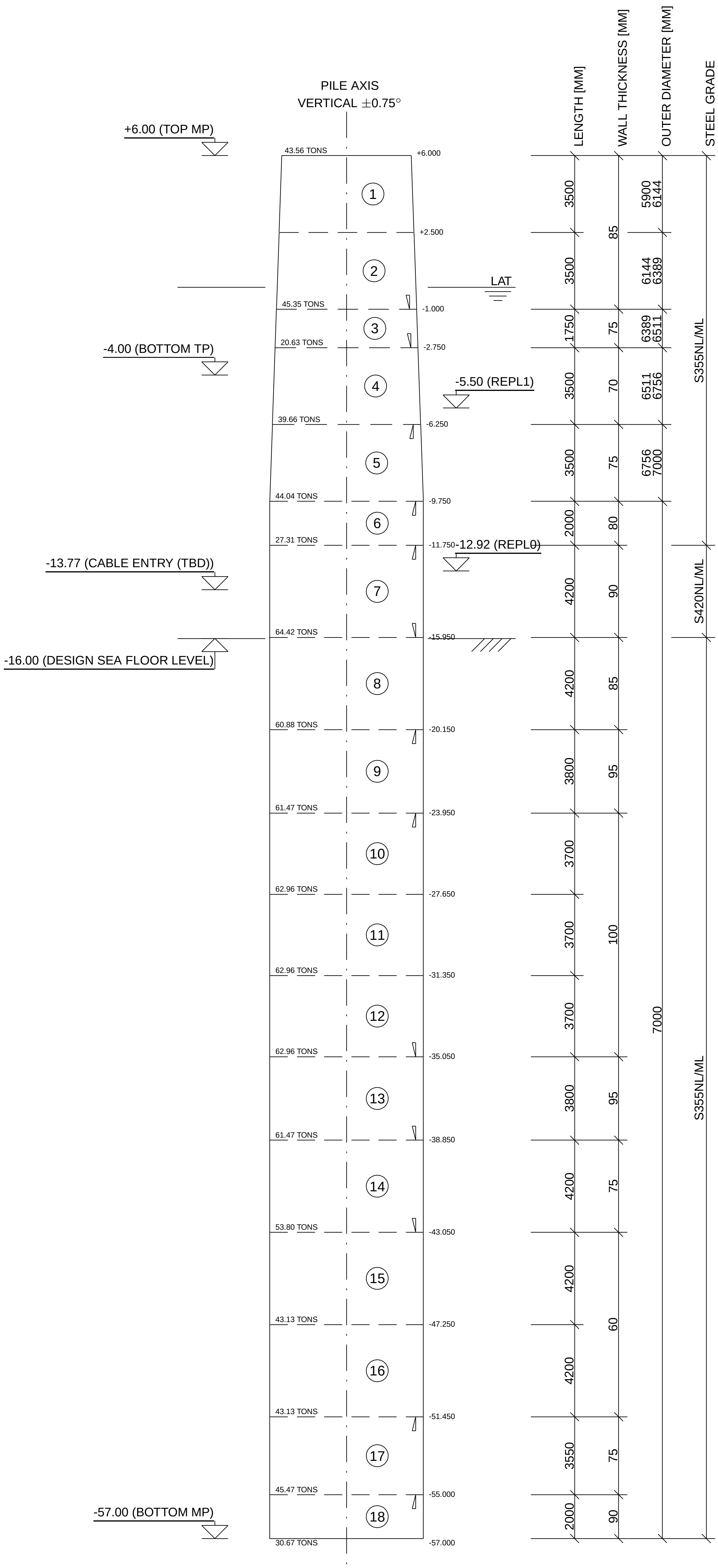
DWG generated 2017-12-19 by script create-drawing-tp.pl, version 2017-11-15 using configuration 00 (structural revision 00) in database INDB.

Appendix B MP and TP dimensions for 6MW reference wind turbine



NOMINAL WEIGHT	[MT]	873.9
COG FROM TOP MP	[M]	31.47
NUMBER OF CANS	[-]	18
TOTAL PILE LENGTH	[M]	63.00
DESIGN EMBEDDED PILE LENGTH	[M]	41.00 -0.00/+0.10

LEGEND:

CIRCUMFERENTIAL WELDS:

- NOT GROUND.
- GROUND FLUSH TO SURFACE ON THE INSIDE.
- GROUND FLUSH TO SURFACE ON THE OUTSIDE.
- GROUND FLUSH TO SURFACE BOTH ON THE INSIDE AND THE OUTSIDE.
- TAPERING

NOTES:

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2. ALL LEVELS ARE IN M RELATIVE TO LAT.
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4. ALL WELDS SHALL BE FULL PENETRATION WELDS WELDED FROM BOTH SIDES.
5. CRUCIFORM WELDS ARE NOT ALLOWED.
6. THE MINIMUM ANGLE BETWEEN LONGITUDINAL WELDS SHALL BE AT LEAST 30 DEGREES FOR ADJACENT CANS.
7. ATTACHMENTS ARE NOT ALLOWED UNLESS NOTED OTHERWISE ON THE DESIGN DRAWINGS.
8. BEVELLING BETWEEN CANS OF DIFFERENT WALL THICKNESS SHALL BE 1:4 IF THICKNESS STEP IS LARGER THAN 4 MM. IF THE THICKNESS STEP IS NOT LARGER THAN 4 MM, THE THICKNESS STEP IS EQUALIZED WITHIN THE WELD.
9. THICKNESS STEPS BETWEEN CANS ARE ON THE INSIDE. THE OUTER SURFACE IS FLUSH.
10. THE PILE TOP SURFACE SHALL BE FABRICATED IN SUCH A WAY, THAT IT IS TRUE PERPENDICULAR TO THE PILE AXIS.
11. REPL MEANS REPLENISHMENT HOLE. REPLENISHMENT HOLES SHALL HAVE AN ELLIPTICAL SHAPE.

Rev.	Date	Description	Designed	Checked	Approved

Client

FOWPI
FIRST OFFSHORE WIND PROJECT OF INDIA

FOWPI First Offshore Wind Project of India

CONCEPT DESIGN
MONOPILE DIMENSIONS (6MW TURBINE)

Description

Designed	GAQI
Drawn	GAQI
Checked	MNKM
Approved	PEVL
Scale	1:200 (for A3)
Date	2017-11-30

COWI
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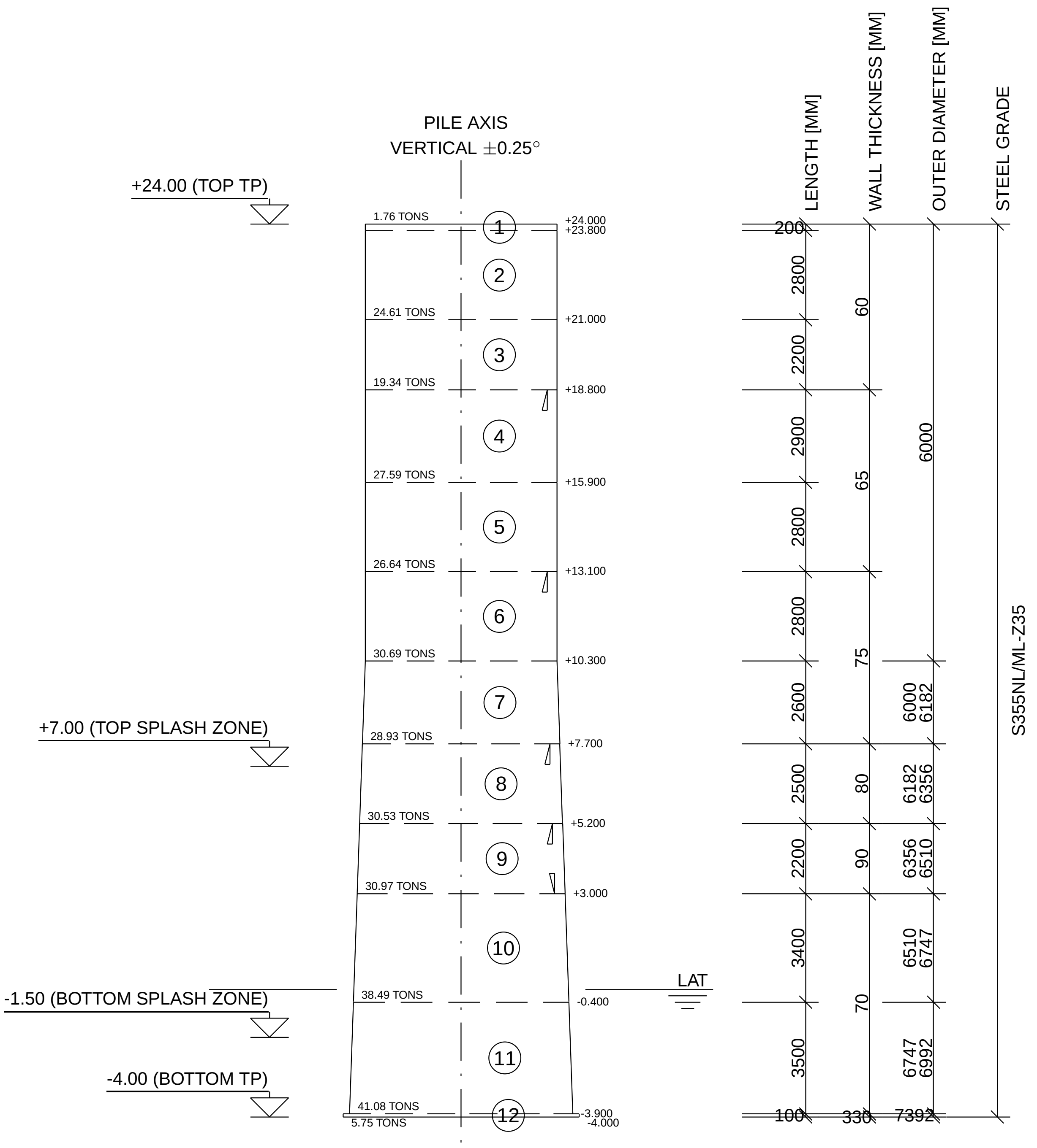
Drawing No. A073635-DWG-019-011 Rev. 1.0

DWG generated 2017-12-19 by script create-drawing-mp.pl, version 2017-11-15 using configuration 00 (structural revision 00) in database INDB.

NOMINAL WEIGHT	[MT]	306.4
COG FROM TOP TP	[M]	15.11
NUMBER OF CANS	[-]	12
TOTAL TP LENGTH	[M]	28.00



LEGEND:

CIRCUMFERENTIAL WELDS:

- NOT GROUND.
- GROUND FLUSH TO SURFACE ON THE INSIDE.
- GROUND FLUSH TO SURFACE ON THE OUTSIDE.
- GROUND FLUSH TO SURFACE BOTH ON THE INSIDE AND THE OUTSIDE.
- TAPERING

NOTES:

1. ALL DIMENSIONS ARE IN MM.
2. ALL LEVELS ARE IN M RELATIVE TO LAT.
3. CIRCUMFERENTIAL AND LONGITUDINAL WELDS SHALL BE FLUSH GROUND, IF GRINDING IS NECESSARY.
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8. THICKNESS STEPS BETWEEN CANS ARE ON THE INSIDE. THE OUTER SURFACE IS FLUSH.
9. THE WELD TOES OF SOME ATTACHMENTS TO THE TP MAY HAVE TO BE GROUND.

Rev.	Date	Description	Designed	Checked	Approved

Client

FOWPI First Offshore Wind Project of India

CONCEPT DESIGN
TRANSITIONPIECE DIMENSIONS
(TP1, 6MW TURBINE)

Description

Designed	GAQI
Drawn	GAQI
Checked	MNKM
Approved	PEVL
Scale	1:200 (for A3)
Date	2017-11-30
Drawing No.	A073635-DWG-019-001
Rev.	1.0

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