REPORT

MV2 Wind Farm Soft Sea Barrier

Concept Design - Monopile Foundation

Client: Eneco Wind B.V.

Reference: BG8375-RHD-ZZ-XX-RP-S-0001

Status: S4/P02

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1 Introduction

1.1 Project background

Eneco will develop the new Wind Farm at Maasvlakte (MV2) in Rotterdam. The wind turbines will be positioned along the outer perimeter of the MV2 reclamation, i.e. along the sea defence. The northern stretch of the sea defence is referred to as the Hard Sea Barrier (HSB) and consists of a revetment structure. The western and south-western stretches of the MV2 outer perimeter accommodate the Soft Sea Barrier (SSB) which consists of a beach-dune system. An overview of the project site is presented in Figure 1-1. The SSB perimeter is indicated in orange, with the anticipated turbine locations within the SSB indicated by blue dots. A total amount of 12 turbines on the SSB will be installed. The intermediate distance is 454 m. Only for ZZ1 and ZZ2 the intermediate distance is 427 m.

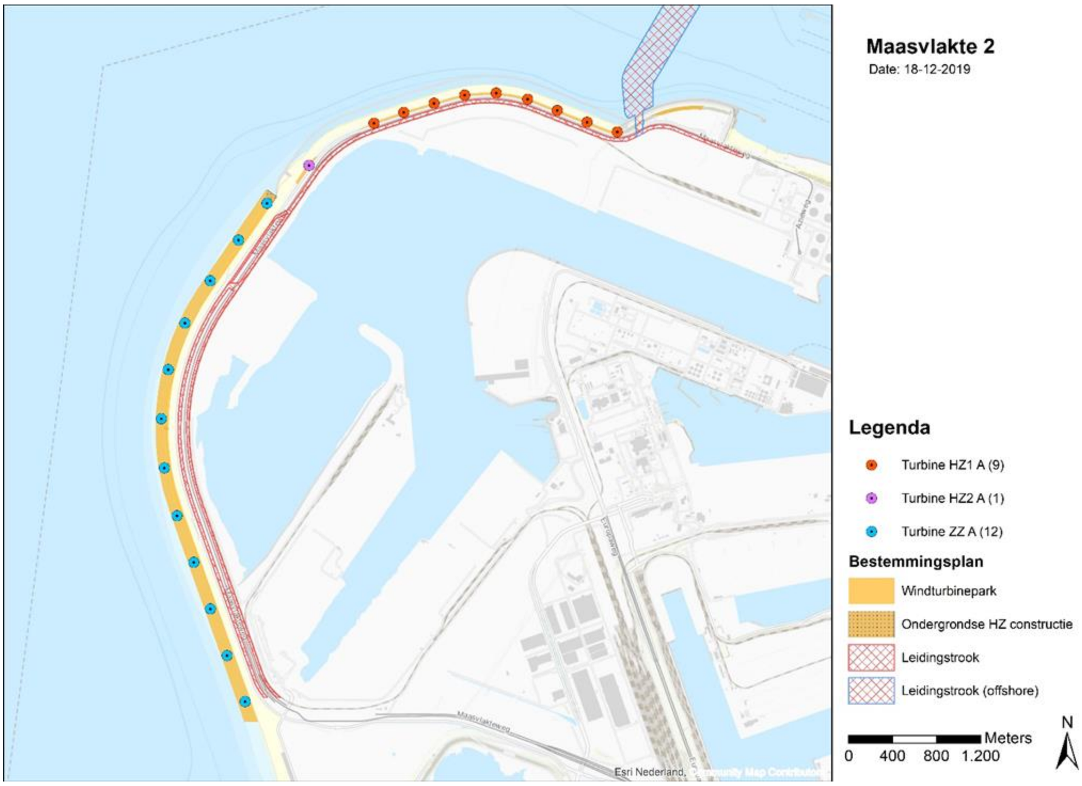


Figure 1-1: Project area overview

1.2 Scope of the report

The scope of this report is the description and verification of a conceptual design of the foundation of the wind turbines positioned along the SSB. It is anticipated that the turbines along the SSB will be founded on large diameter monopiles. Chapters 2, 3, 4 will present the design approach for the concept evaluation and the obtained results, as part of the structural integrity assessment. In chapter 5 the conceptual aspect of the monopile transportation, installation, and grid connection will be presented to the reader.

1.3 Concept design loop

The monopile foundation design will be developed in three steps: concept design, preliminary design and final design, visualised in Figure 1-2. This report documents the concept design of the monopile foundation.

The aim of the concept design report is to prove the technical feasibility of the monopile foundation type for the new wind farm development along the MV2 SSB.
An envelope of sizes and dimensions will be given. Principles of techniques are presented and used as verification of the technical feasibility.

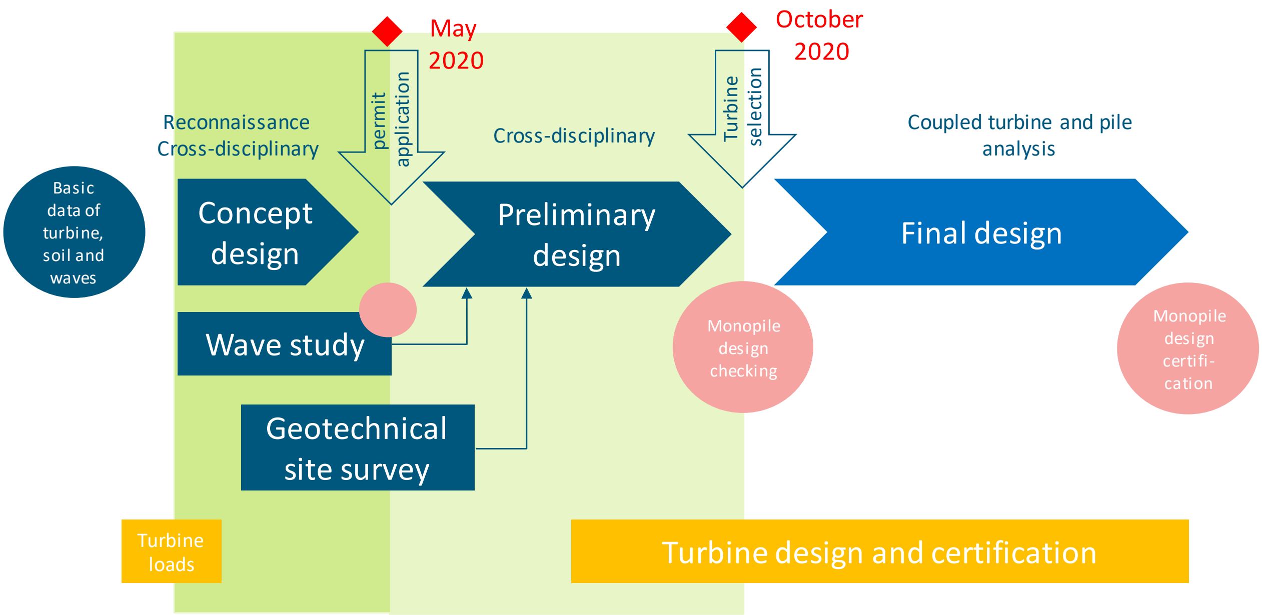


Figure 1-2: Concept design loop in relation to monopile foundation design

The preliminary design step will be based on data that will be generated in the concept design phase as a parallel activity. A wave study will be carried out. Also, a geotechnical survey will be undertaken to investigate the properties of the subsoil.

The concept design step is based on data available from previous studies and investigations, which are sufficient for a conceptual design verification. This concept design report together with the new investigations and studies will define the input and starting points of the next the preliminary design step. Along with the existing environmental conditions, basic input from four turbine suppliers is used for the concept design assessment. Care is taken in the analysis as the available turbine data is not suited for construction and is only meant as input for the concept design of the fixed concrete turbine foundations of the HSB. Further details on the starting points for detailed design are explained in Chapter 2.

Later, the design of the individual monopiles will be customised and optimised for their own location, their specific wave load and subsoil condition. At this stage no clustering of turbines is considered. In the concept design typical soil profiles were selected to envelope the conditions for all piles. Based on the outcomes of the erosion study, new geotechnical investigations and soil interpretations as well as wave studies, clustering and/or customization will be considered for the next phase.

A most probable optimistic and pessimistic scenario are considered now in terms of general erosion as well the most unfavourable soil conditions from available data. These items are further detailed and explained in upcoming sections of this report.

2 Starting points concept design loop

2.1 General – Design Codes and Standards

DNV-GL:

- DNVGL-SE-0190: Project certification of wind power plants
- DNVGL-SE-0074: Type and component certification of wind turbines according to IEC 61400-2
- DNVGL-ST-0126: Support structures for wind turbines
- DNVGL-ST-0437: Loads and site conditions for wind turbines
- DNVGL-ST-0054: Transport and installation
- DNVGL-ST-0359: Subsea power cables

Eurocodes, European Norms and their respective Dutch National Annexes:

- EN 1990 Eurocode – Basis of structural design
- EN 1991-1-4: Actions on structures – wind actions
- EN 1993-1-1 Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings
- EN 1993-1-5 Eurocode 3: Design of steel structures - Part 1-5: Plated structural elements
- EN 1993-1-6 Eurocode 3: Design of Steel Structures, Part 1-6: Strength and Stability of Shell Structures
- EN 1993-1-7 Eurocode 3: Design of steel structures - Part 1-7: Plated structures subject to out of plane loading
- EN 1993-5 Eurocode 3: Design of steel structures – Part 5: Piling
- EN 1997-1 Eurocode 7: Geotechnical design – Part 1: General Rules
- EN 1997-2 Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing
- EN 10088-1 Stainless steels - Part 1: List of stainless steels
- EN 10225 Weldable structural steels for fixed offshore structures – technical delivery conditions
- EN 10228-3 Non-destructive testing of steel forgings - Part 3: Ultrasonic testing of ferritic and martensitic steel forgings
- EN 12495 Corrosion Protection of Fixed Offshore Structure
- EN 14399 (All parts) High-strength structural bolting assemblies for preloading

Publications, design manuals, recommended practices

- DNV-RP-C201: Buckling Strength of Plated Structures
- DNVGL-RP-C202: Buckling strength of shell
- DNVGL-RP-C203: Fatigue design of offshore steel structures
- DNVGL-RP-C204: Design against accidental loads
- DNVGL-RP-C205: Environmental conditions and environmental loads
- DNVGL-RP-C207: Statistical representation of soil data
- DNVGL-RP-C208: Determination of structural capacity by non-linear finite element analysis methods
- DNVGL-RP-C210: Probabilistic methods for planning of inspection for fatigue cracks in offshore structures
- DNVGL-RP-0360 Subsea power cables in shallow water

Publications, articles, manuals, journals

- *Fatigue estimation methods comparison for wind turbine control,* J.J. Barradas, Berglind and Rafael Wisniewski, 2014

- *Dynamics of offshore wind turbines supported on two foundations,* Subhamoy, Bhattacharya, James A. Cox, Domenico Lombardi, David Muir Wood, Institute of Civil Engineers – Geotechnical Engineering vol 166 Issue GE2

2.2 Design Criteria

2.2.1 Design reliability and design lifetime

The reliability of the foundation of the wind turbines is that it can be equal to what is normally required for onshore foundations: CC2. The failure probability associated with this category is 1.4 x 10^{-6} per year.

This chance is also sufficiently small to meet the simple test of 1% chance of failure in a storm condition with a return period of 10^{-4} per year. The foundation is hence a stable element in the vicinity of the flood defence. Consequential failure mechanisms associated with turbine foundation failure could be (a) a large break out of a soil cone: a slip circle failure in case of a too short pile, or (b) a local buckling or bending capacity failure of the pile, resulting in a turbine tower falling on the beach and/or the dune. The conditional failure probability associated with those mechanisms to happen simultaneous to a 10^{-4} storm condition is very low (< 1%) and will hence not reduce the safety level of the flood defence system.

The design service life of the wind turbine assembly is 25 years. The support structure is as a minimum designed for 25 year service life. The selected turbine supplier is to confirm this value for the other components, including the tower.

Based on the outcomes of the erosion and wave studies, zones of severity of corrosion can be distinguished and it will be decided what type of corrosion protection will be adopted.

2.2.2 Material factors

- *Soil:*

In the following sections and detailed in paragraph 2.5.5, it is explained how the safety factors and load combinations have been used in estimating the monopile structural and geotechnical capacity.

- *Steel*

For the Concept Design Phase, the standard EN1993-1-1 + Dutch Annex have been used as indicated in Table 2-1, with a high level of underutilization of the cross-section.

Table 2-1: Steel material partial safety factors

Cross-section class	γ_{M0}	γ_{M1}
1…3	1.0	1.0
4	1.0	1.1

In accordance with DNV-ST-0126, buckling factors do not distinguish between Cross section Classes. In the next project phase these will be considered, once the preliminary Fatigue analyses can be performed and the Cross-section further optimized.

- *Bolts*: γ_{M2} = 1.25

2.3 Design concept

The design concept of the wind turbines along the Soft Sea Barrier (SSB) with monopile foundation is sketched in Figure 2-1.

The monopile foundation consists of an open-ended steel tubular pile. The turbine tower (provided by the turbine supplier) is connected to the monopile foundation at interface level. The interface level is initially defined at NAP +7.75m on basis of extreme water levels, wave action and a minimum airgap of 1 meter.

The mudline level is dependent on the extent of beach and dune erosion and development of local scour around the monopile. The native mudline level is initially assumed at average low water elevation, i.e. at NAP -0.6m. The embedment of the monopile foundation into the soil must be sufficiently deep to provide the required lateral and axial bearing capacity under the prescribed maximum load demands and the considered erosion and scour scenarios.

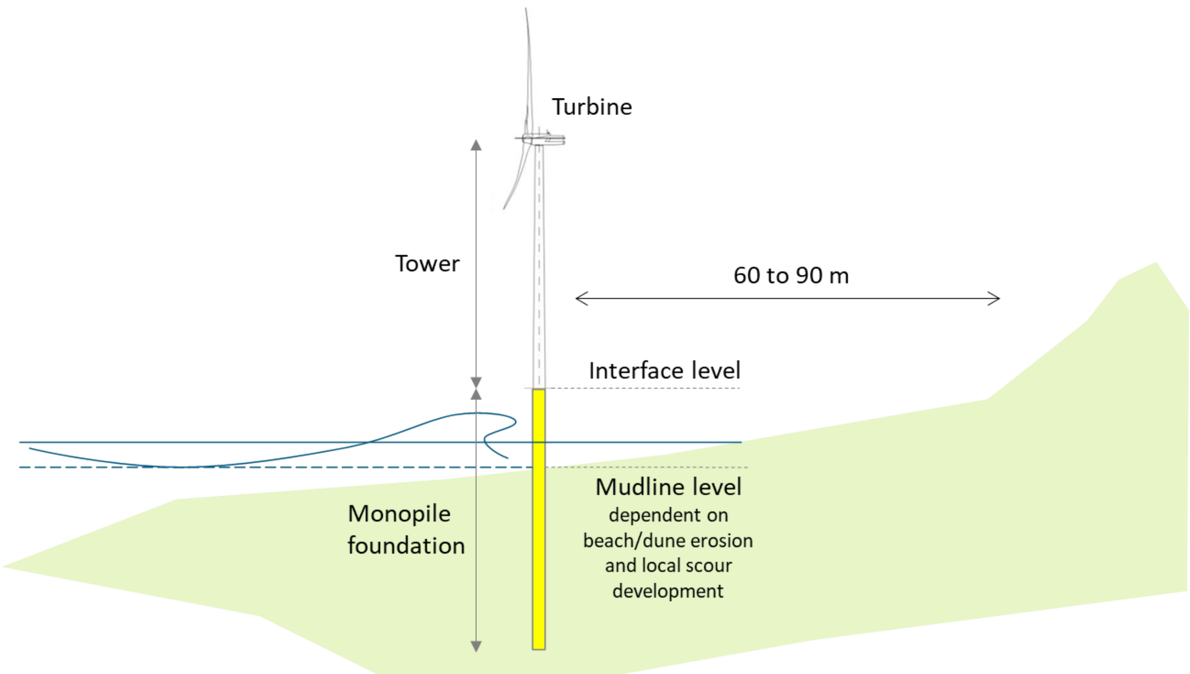


Figure 2-1: Wind turbine with monopile foundation concept in MV2 Soft Sea Barrier

2.4 Choice of design concept

The choice for the monopile foundation concept is directly related to the specific mitigation measures following from the risk management plan [1] and corresponding verification matrix [2]. The relevant mitigation measures prescribing the monopile foundation concept, including their specific ID, are listed in Table 2-2.

Table 2-2: Risk mitigation measures prescribing the monopile foundation concept

Risk 2.2-M10 [1] Risk 2.4-M1 [1] Measure M2.4.1.1 [2]	Hard elements in the SSB negatively influence the	Application of monopile foundation with maximum diameter of 5.5m at non-eroded mudline level. Scour	Minimised number of hard elements in SSB compared to traditional wind turbine foundation so that	We apply a scour protection locally around the monopiles, but below the minimum long-term bed level. This allows us to

	morphology of the SSB	protection is not applied. Daily inspection of scour holes and regular maintenance will be executed by local asset manager.	disturbance of sand transport / morphology is prevented.	enable cable entry into the monopile whilst still minimizing hard elements at the SSB surface. This is further explained in section 5.
Risk 2.3-M11 [1] Risk 2.4-M7 [1] Measure M2.3.11.1 [2]	Recreationists can showcase unpredictable and unsafe behaviour	Due to application of monopiles with a high interface level the turbines will be hard to climb	Additional safety on top of minimum legal requirements to prevent unsafe situations	The monopiles are designed with a smooth surface at human reach, no external cable ladders starting at current seabed level. and the wind turbine entry is several meters above the ground. See drawing BG8375-RHD-ZZ-XX-DR-S-0004-2of1

2.5 Geotechnical starting points

2.5.1 General aspects – geotechnical interpretation

The existing soil data comprise results of a site investigation carried out before the Maasvlakte 2 was created and a site investigation after completion of the Maasvlakte 2 area. Site investigations consisted of CPT's and a few boreholes including laboratory testing. Relevant data used for the current phase are included in sections below. Locations of existing CPT's and boreholes on plot below, along with tentative locations of ZZ wind turbines.

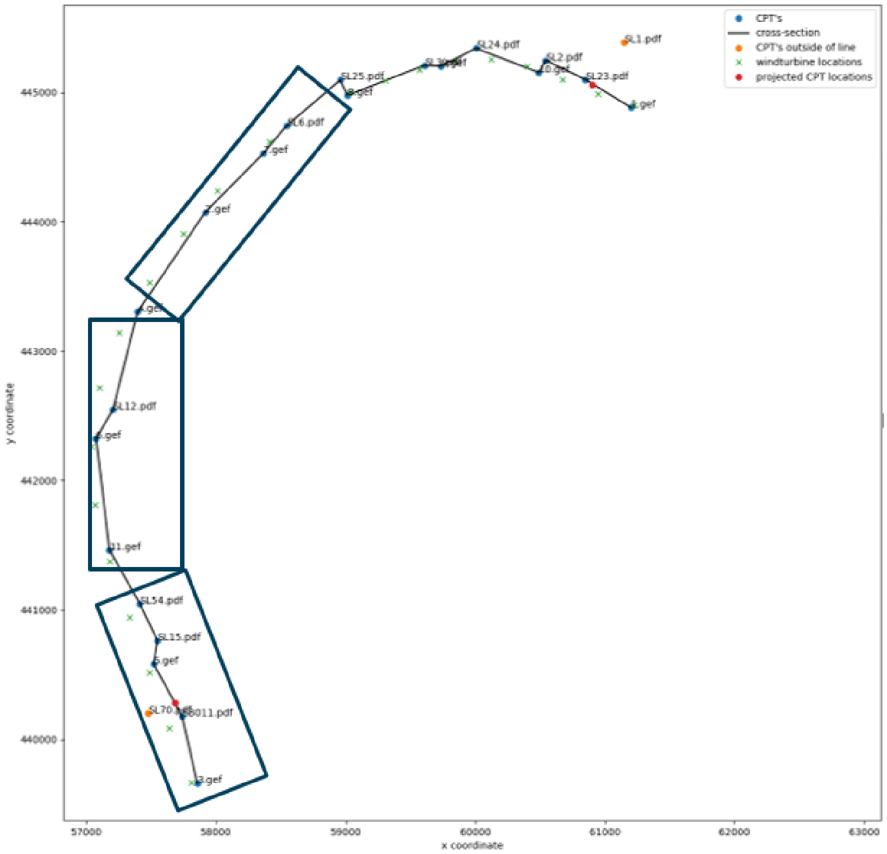


Figure 2-2 Locations of available CPT's from previous soil investigations

2.5.2 Preliminary ground profiles

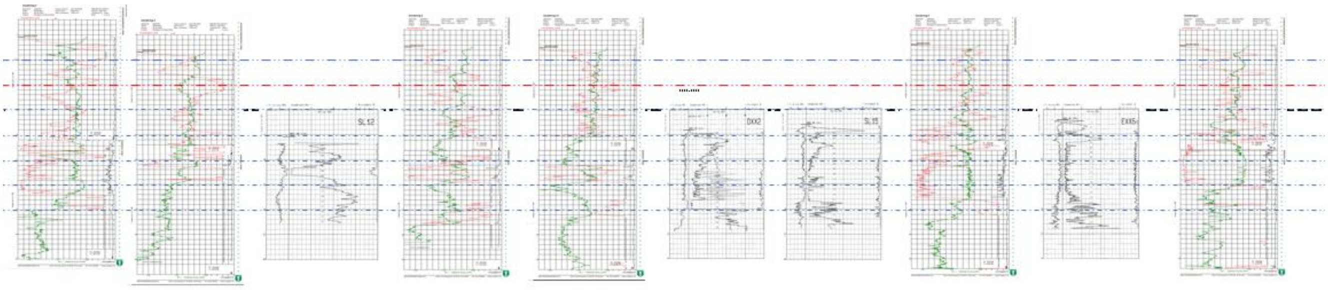


Figure 2-3 Available CPT results from previous soil investigations (left to right: north to south)

Available data show:

- Variable degree of compactness of fill, also locally silty layers
- Large variation in thickness, nature and properties of the compressible layers on the original seabed
- Medium to extremely dense sand at greater depth

2.5.3 Soil parameters

Based on the available CPT data two preliminary soil profiles are defined for concept design: a North profile (based on MOS CPT2) and a South profile (based on PUMA CPT SL70). The PUMA CPT was carried out from original seabed but is governing with respect to the presence of clayey deposits. The upper sand soils are reclaimed soils. Given the relatively large distance of the CPT data points to the monopile locations and the variation shown it was considered prudent for the concept design phase to use one governing sand profile for the upper sands. In the next design phase, with the CPT data close to the monopile locations, representative clusters will be defined accounting for the variations and thereby reducing uncertainties for the design.

The locations of these profiles are presented in Figure 2-4.

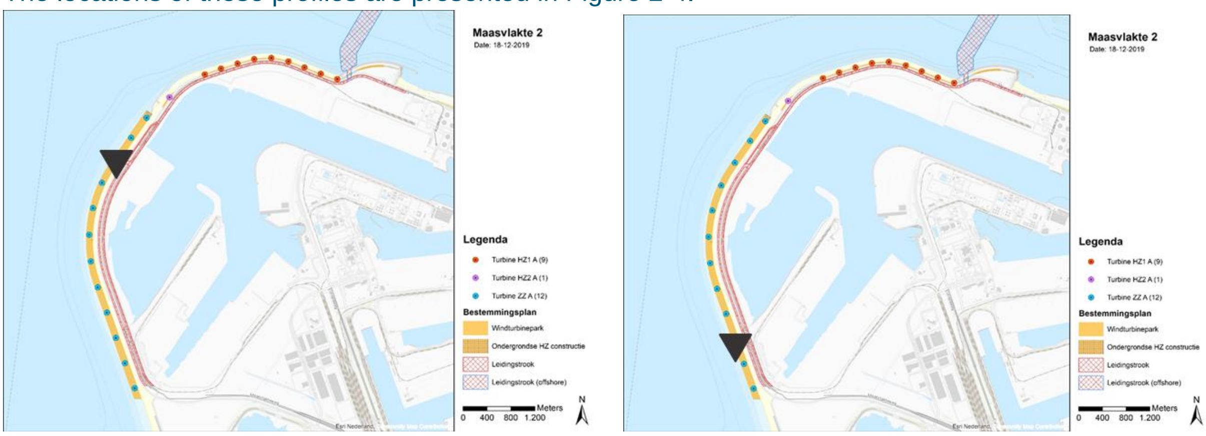


Figure 2-4: Location of North soil profile (left) and South soil profile (right) along the SSB

The preliminary soil profile parameters are summarised in Table 2-3. The presented soil parameters are for modelling the interaction between soil and monopile structure in the form of lateral soil springs (p-y curves). The preliminary p-y curves will be defined according to API standard [3]. The presented parameter values are interpreted as representative values.

The symbols in Table 2-3 represent the following soil properties:

- q_c = CPT cone resistance [MPa]
- γ' = effective volume weight [kN/m^3]
- φ = angle of internal friction [°]
- S_u = undrained shear strength [kPa]
- k_h = initial modulus of horizontal subgrade reaction [kN/m^3]
- ε_{50} = strain at 50% of the maximum deviator stress in laboratory undrained compression tests of undisturbed soil samples

Table 2-3: Preliminary soil parameters for monopile foundation concept design: north profile (left) and south profile (right)

Depth z	Soil Type	qc	γ'	ϕ	Su	kh	ε50
[m NAP]		[MPa]	[kN/m³]	[°]	[kPa]	[kN/m³]	[-]
4.50	sand	30.0	21	37.5	0	31100	0
0.00	sand	30.0	11	37.5	0	31100	0
0.00	sand	23.0	11	37.5	0	31100	0
-2.00	sand	23.0	11	37.5	0	31100	0
-2.00	sand	12.0	9	30.0	0	7400	0
-3.50	sand	12.0	9	30.0	0	7400	0
-3.50	sand	20.0	10	32.5	0	15400	0
-7.00	sand	20.0	10	32.5	0	15400	0
-7.00	sand	9.5	9	30.0	0	7400	0
-8.00	sand	9.5	9	30.0	0	7400	0
-8.00	sand	18.0	10	32.5	0	15400	0
-12.50	sand	18.0	10	32.5	0	15400	0
-12.50	sand	12.0	9	30.0	0	7400	0
-15.00	sand	12.0	9	30.0	0	7400	0
-15.00	sand	17.0	10	32.5	0	15400	0
-16.50	sand	17.0	10	32.5	0	15400	0
-16.50	silt	1.2	10	27.5	0	1300	0
-18.00	silt	1.2	10	27.5	0	1300	0
-18.00	sand	18.0	10	32.5	0	15400	0
-20.00	sand	18.0	10	32.5	0	15400	0
-20.00	silt	4.0	11	30.0	0	7400	0
-24.00	silt	4.0	11	30.0	0	7400	0
-24.00	clay	1.2	8	0.0	60	1500	0.006
-26.00	clay	1.2	8	0.0	60	1500	0.006
-26.00	sand	30.0	11	37.5	0	31100	0
-27.00	sand	30.0	11	37.5	0	31100	0
-27.00	sand	30.0	11	37.5	0	31100	0
-45.00	sand	30.0	11	37.5	0	31100	0

Depth z	Soil Type	qc	γ'	ϕ	Su	kh	ε50
[m NAP]		[MPa]	[kN/m³]	[°]	[kPa]	[kN/m³]	[-]
4.50	sand	30.0	21	37.5	0	31100	0
0.00	sand	30.0	11	37.5	0	31100	0
0.00	sand	23.0	11	37.5	0	31100	0
-2.00	sand	23.0	11	37.5	0	31100	0
-2.00	sand	12.0	9	30.0	0	7400	0
-3.50	sand	12.0	9	30.0	0	7400	0
-3.50	sand	20.0	10	32.5	0	15400	0
-7.00	sand	20.0	10	32.5	0	15400	0
-7.00	sand	9.5	9	30.0	0	7400	0
-8.00	sand	9.5	9	30.0	0	7400	0
-8.00	sand	18.0	10	32.5	0	15400	0
-12.00	sand	18.0	10	32.5	0	15400	0
-12.00	clay	1.0	8	0.0	50	1200	0.007
-13.00	clay	1.0	8	0.0	50	1200	0.007
-13.00	sand	17.0	10	32.5	0	15400	0
-14.50	sand	17.0	10	32.5	0	15400	0
-14.50	clay	1.0	8	0.0	50	1200	0.007
-15.00	clay	1.0	8	0.0	50	1200	0.007
-15.00	sand	13.0	9	30.0	0	7400	0
-16.50	sand	13.0	9	30.0	0	7400	0
-16.50	clay	1.5	8	0.0	75	2000	0.005
-26.50	clay	1.5	8	0.0	75	2000	0.005
-26.50	sand	4.0	9	30.0	0	7400	0
-28.75	sand	4.0	9	30.0	0	7400	0
-28.75	sand	6.0	9	30.0	0	7400	0
-32.75	sand	6.0	9	30.0	0	7400	0
-32.75	sand	7.0	9	30.0	0	7400	0
-34.00	sand	7.0	9	30.0	0	7400	0
-34.00	sand	30.	11	37.5	0	31100	0

2.5.4 Static and dynamic behaviour

In the concept design loop of the monopile foundation the turbine design loads are introduced as static loads on top of the monopile (at interface level). The soil-structure behaviour that is computed with static p-y curves can be interpreted as pseudo-static behaviour. For the preliminary assessment of dynamic effects, the same calculation is also performed with cyclically degraded soil springs, compliant with API methodology described in [3]. In the calculation the maximum amount of load cycles is assumed for determining the degraded p-y curves. Through this approach the ultimate envelope condition for the foundation system is assessed.

2.5.5 Geotechnical design approach

In accordance with the applicable DNV-GL standards the geotechnical foundation design of wind turbine foundations is essentially based on a load and resistance factor design approach (LRFD). This is in conformity with Design Approach 3 (DA3) of Eurocode EN 1997 which prescribes the use of partial load factors and material factors to derive the design values for the loads and the various soil parameters from their respective characteristic (loads) and low representative (soil parameters) values. DA3 is commonly applied in the Netherlands for various geotechnical structures and foundation types. For the purpose of the wind turbine foundation, the partial material factors prescribed in Annex A of the Eurocode 7 apply [4].
More specifically the following applies for DA3:

It shall be verified that a limit state of rupture or excessive deformation will not occur with the following combination of sets of partial factors:

Combination: *(A1* or A2⁺)* "+" *M2* "+" *R3*

Where:

- A represents the partial factors for actions
  - * on structural actions (i.e. turbine loads at the interface level)
  - ⁺ on geotechnical actions
- M represents the partial factors for materials (i.e. the soil parameters)
- R represents the partial factor for resistance
- "+" implies: "to be combined with".

Note 1: In this approach, partial factors are applied to actions or to the effects of actions from the Structure (i.e. turbine loads at the interface level) and to ground strength parameters (i.e. the soil parameters).

Note 2: For slope and overall stability analyses, actions on the soil (e.g. structural actions, traffic load) are treated as geotechnical actions by using the set of load factors A2.

Note 3: For the concept design stage it is assumed that Consequence Class 2 (CC2) / Reliability Class 2 (RC2) as specified in Eurocode 0 applies for the required safety level of the monopile foundation. Partial factors reported in Annex A of the Eurocode 7, which apply to CC2/RC2, are in this stage temporarily multiplied by a factor 1.1 to implement additional margin for uncertainty in the soil.

In the concept design step, the available geotechnical data is limited. An appropriate and accurate parameter determination for DA3 is not sensible, and therefore the concept design step Design Approach 2 (DA2) is adopted for the monopile foundation assessment. In DA2 equivalent to DA3 partial load factors (A1) are applied on the turbine loads (refer to Section 2.7) but the partial material factors (M2) on the specific soil parameter values are replaced by an equivalent soil resistance factor ("$R_{M2,eq}$") on the overall lateral soil-pile behaviour. This simplified approach is possible in this concept design stage because the overall lateral behaviour appears to be dominated by the upper sand soils for which the resistance is determined by the single strength parameter φ, i.e. the angle of internal friction.

The value of the equivalent soil resistance factor $R_{M2,eq}$ will as such be based on the maximum of:
- Partial material factor on tangent of φ according to Eurocode 7 – DA3 [4]
- Partial material factor on tangent of φ according to DNV-GL-ST-0126 [5]
- Soil resistance factor according to Eurocode 7 – DA2 [4]

The partial material factor on the tangent φ according to EC7-DA3 equals 1.1 x 1.25 = 1.4. The partial material factor on the tangent φ according to DNV-GL-ST-0126 equals 1.15. The resistance on soil bearing capacity and earth resistance according to EC7-DA2 equals 1.1 x 1.4 = 1.5.

For the concept design phase the soil resistance factor $R_{M2,eq}$ is set to 1.5. Considering that the DNV-GL-ST-0126 standard is specifically developed for wind turbine support structures and mentions a considerably lower value for the partial material factor, it is concluded that adequate conservatism is implemented in the concept design phase with $R_{M2,eq}$ = 1.5.

As a final remark, it is noted that lateral bearing capacity of the soil-pile system is governing over axial bearing capacity. As such the focus of the geotechnical foundation design during concept design loop is focused on the verification of lateral bearing resistance. Axial bearing capacity will be verified in the preliminary design stage when the final soil investigation is available.

2.5.6 Geotechnical ground investigations – surveys

The available soil data show significant spatial variation in ground conditions including the presence of old gullies. For this reason, a site investigation is scheduled to comprise one seismic CPTU in the centre of each wind turbine location to a depth of approx. NAP-45 m or deeper if encountered ground conditions are poor.

A total of 6 boreholes will be executed in the near vicinity of selected CPT locations to a depth of approx. NAP -35 m. These boreholes are in particular for calibration purposes but also to obtain undisturbed samples for more advanced laboratory tests.

2.6 Erosion and scour scenarios

Beach erosion and local scour are subject to separate specialist studies. These studies are aimed at gaining insight and as a mean of risk assessment of the effect of the wind turbines on the flood defence system. Outcomes of these studies not necessarily are one-to-one design scenarios for the turbine foundations themselves.

The beach erosion and natural accretion follow a maintenance plan with guaranteed quantities of sand in vertical zones, i.e. between -8 and -4, between -4 and +3 and above +3 m NAP.

Individual storms may cause erosion and temporary relocation of sand volumes to a lower zone. An associated lower beach level will be used as a design scenario for the piles.

Local scour is understood to be caused a wave-driven long-shore currents of typically maximum 1.5 m/s, associated with the 1:100 yr storm conditions. Scour holes that might occur under these conditions will be estimated base on the applicable standards and specialist studies, if needed.

Prior to final results, to obtain insight in the sensitivity of the monopile foundation to the beach/dune erosion and local scour development, a set of erosion scenarios is considered in the concept design step. The scenarios are summarised in Table 2-4.

Table 2-4: Erosion scenarios considered for monopile concept design

Erosion scenario	Mudline level	Remarks
No erosion / scour	NAP -0.6m	In this scenario the target static foundation stiffness defined by the turbine suppliers is assessed with SLS loads (see section 2.7)
General erosion -4m	NAP -4.0m	For the push-over analysis both static capacity of the soil and cyclic degradation is considered. For the moment this is only done for this mudline level to verify the sensitivity of the pile response to cyclic degradation. Recommended pile toe levels for other scour scenarios will include the outcome of this verification.
General erosion -7m	NAP -7.0m	For push-over analysis the static capacity of the soil is considered. For the structural capacity check the effect of cyclic degradation is considered for a mudline level at NAP -7.0m as a most likely worst-case scenario.
General erosion -7m + additional local scour	NAP -10.3m with reduced spring capacity to a level of NAP -13.5m	In this scenario the redundancy of the foundation system is assessed in case of lacking maintenance after considerable beach / dune erosion Cyclic degradation is for this scenario specifically not taken into account as it is considered to be an unlikely scenario.

Measured coastal erosion profiles at cross-sections along the MV2 SSB (Figure 2-5) support the assumption that the present mudline level at the monopile locations is situated around NAP -0.6m (i.e. average low water level). Considering the historic development of coastal erosion and potential dune erosion during a storm the two scenarios of general erosion to NAP -4m and NAP -7m were selected. General erosion is defined as an overall lowering of the beach level resulting in a sloping bed level with an e.g. 1:30 slope. In the geotechnical calculations this is modelled as a horizontal level. The scenario with general erosion to NAP -7m plus an additional scour hole around the piles is to assess the redundancy of the foundation system in case of lacking maintenance after considerable beach and dune erosion.

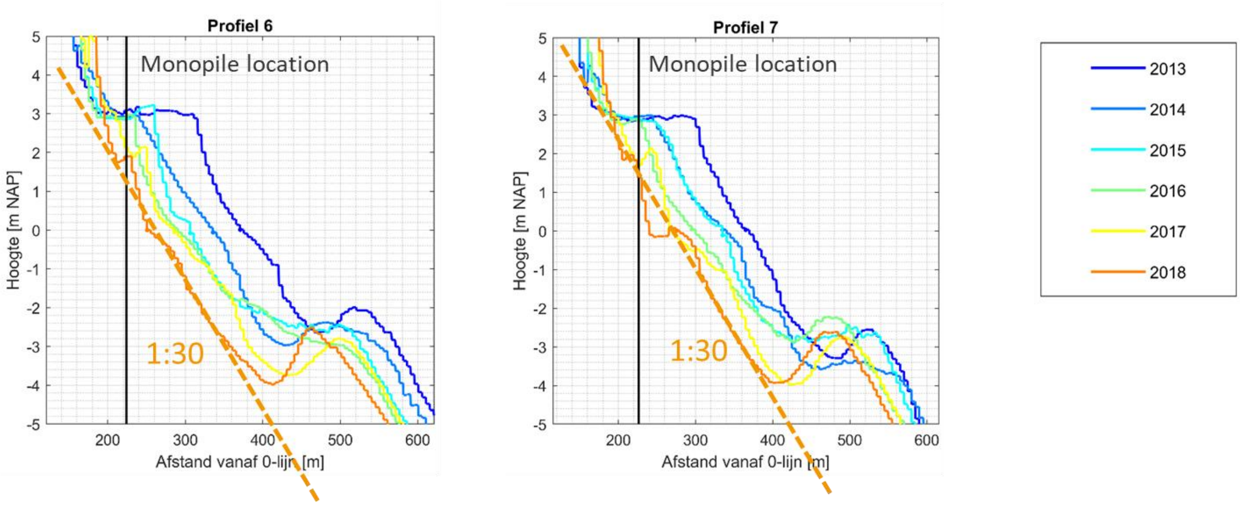


Figure 2-5: Measured coastal erosion at two cross-sections along the MV2 SSB

It is noted that along part of the MV2 SSB perimeter coastal accretion has been observed while at other locations coastal erosion has been measured (as shown in Figure 2-5). The locations of the soil profiles selected for design and the locations of coastal accretion / erosion zones are for concept foundation design not yet linked. In concept design phase the various erosion scenarios are assessed with the governing soil profile to secure that the design is also feasible in case conservative erosion scenario's would apply.

2.7 Turbine interface loads and foundation target rotational stiffness

The turbine interface loads for the monopile foundation concept design are selected the available non contractual data as shown in Annex A2 . Turbine loads are provided by four suppliers: Enercom, GE, Vestas and Siemens. Their data are based on turbines of approximately the same category and size. The terminology of the load combinations differs per supplier and in this early phase of the project the level of detail is insufficient to clearly distinguish which turbine load cases are specifically included in the load combinations. Therefore, the approach for turbine interface load definition during concept design phase is as follows:

- The selection of interface loads is predominantly based on the turbine loads provided by Enercom and GE as these are most comprehensive. The turbine loads provided by GE are considered representative for the Vestas and Siemens turbine loads.
- For the ULS condition an envelope set of loads is selected per supplier. It is observed from the information provided by Enercom and GE that ULS 'normal' and 'abnormal' design load conditions fall within the same envelope.
- For the SLS condition the SLS (unfactored) 'normal' design load conditions are selected. For Enercom the overall factor between ULS and SLS 'normal' bending moment is approximately 1.35. For GE the overall factor between ULS and SLS 'normal' bending moment is approximately 1.5.

In summary the implemented turbine loads for concept design are presented in Table 2-5. The loads are introduced at interface level (i.e. NAP +7.75m). For the preliminary design phase further definition of the load combinations is required.

Table 2-5: Turbine interface loads considered for monopile concept design, from ***Error! Reference source not found.***

Supplier	Axial force [kN]	Shear force [1] [kN]	Bending moment [1] [kNm]	Torsion [kNm]
		ULS envelope [2]		
Enercom	-12848	1878	228911	19428
GE / Vestas / Siemens	-8695	1717	158776	13534
		SLS 'normal'		
Enercom	-8727	1391	166806	14391
GE / Vestas / Siemens	-5885	1240	106532	10025

[1] The presented shear forces and bending moments are the quadratically combined components in main horizontal directions
[2] Assumed to be factored in accordance with DA3

As presented inA2, Enercom and GE provide static target rotational stiffnesses of the foundation system:

- $k_{\varphi,Enercom}$ > 26.666 GNm/rad
- $k_{\varphi,GE}$ > 30.000 GNm/rad

2.8 Wave loads

For the preliminary design a separate, site specific wave load study will be performed. This study includes the transformation of offshore wave and wind and water level data into near-shore data. This conditions are further analysed to calculate static and dynamic wave loads in the piles, for all sea states and load scenario's to be considered, whether or not corelated with wind.

The methodology of determination of wave loads in this concept design step is as follows. A beach profile is chosen as a starting point. Initially the beach is around -0.6 m NAP, having a 1:30 slope toward the sea. Over the years the beach might erode, leading to a typical bed level of - 2 m NAP at the position of the monopile.
In case of a design storm a surge level of +3 m NAP can occur. This leads to a water depth of 5 m.

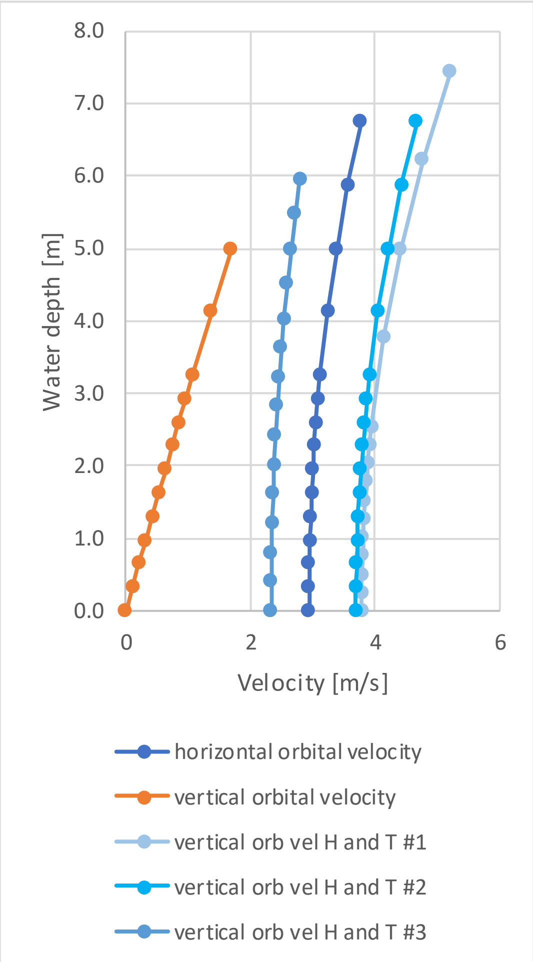
In storm conditions large waves can be generated. The wave conditions used for the design of the sea defences can be used as a reference. For the SSD, the underwater geometry of the foreshore and the beach will cause shoaling and depth-induced breaking, which processes transform the waves, leading to a reduced wave height.

	H_{m0}	T_{m-10}
1:100 yr condition, point MZ4	6.8 m	11.3 s
1:100 yr, depth-limited 5 m	3.5	11.3

The depth-limited wave parameters are given as spectral values. In order to determine extreme values of cyclic loads typical combinations of individual waves and periods can be given.

	H_i	T_i
Combinations for 'normal' Rayleigh distributed wave heights	1.8	0.9
	1.0	1.3
	0.55	1.5
Combinations for depth-limited wave heights	1.4	0.9
	1.0	1.3
	0.55	1.5

Using dispersion relations, based on the combination of depth-limited wave height and period, the wave orbital velocities and acceleration can be computed as a vertical profile at the location of the pile.



The ration between wave period and pile diameter (5 m) permits the use of Morison equations for calculation of drag and inertia forces. Conservatively they are summed, and the result of these calculations is shown below. The adopted Morison coefficients are 1.2 for drag and 2.0 for inertia. The coefficient of 1.2 is conservative and is based on a rough surface, cause by marine growth. Based on these results a maximum wave load value of 2000kN is estimated. This wave load value is conservatively introduced at the interface level (i.e. NAP +7.75m).

intermediate water waves				1	2	3
spectral wave parameters						
Wave height	H_{m0}	3.5				
Wave period	T_{-10}	11.3				
Water level (relative to Ref)		3.0				
Bottom level (relative to Ref)		-2.0				
Combinations of H and T	H_i	*1.0*		***1.4***	***1.0***	***0.55***
	T_i	*1.0*		***0.9***	***1.3***	***1.5***
Total Morison forces [kN]				1777	1250	615
z coordinate of the force (rel to bottom level)				4.2	4.3	4.4
Impact force Wifi JIP I		388				

2.9 Verification criteria

The performance of the monopile foundation is verified on three main items, as summarized in Table 2-6.

Table 2-6: Verification criteria for monopile concept design

Behaviour	Verification
SLS deformations [1]	Rotational stiffness in SLS: • $k_{\varphi,Enercom}$ > 26.666 GNm/rad • $k_{\varphi,GE}$ > 30.000 GNm/rad Normal operational load conditions in SLS: • Lateral deflection at mudline: 3% of pile diameter • Maximum inclination: 1mm/m Extreme operational ("abnormal") load conditions in SLS: • Lateral deflection at mudline: 6% of pile diameter • Maximum inclination: 3mm/m
ULS structural capacity	Unity check in the final design step to be based on CC2 / RC2 Eurocode safety level [2]. Working stress < 40% of yield stress to create sufficient margin for fatigue (which is not assessed in concept design yet) and to accommodate the equivalent soil resistance factor of 1.5.
ULS geotechnical stability (push-over)	Margin assessment: load demand and erosion scenarios in relation to push-over curves and minimum pile toe level

[1] A separate check is the effect of the initial rotation of the unloaded as-installed wind turbine immediately after construction (installation tolerance). This check will be performed during preliminary design phase when more details are available.

[2] For CC2 / RC2 the probability of exceedance is 10^{-4} (β = 3.8) for a design life of 50 years. This is considered conservative as the design lifetime of the wind turbines is only 25 years.

For structures in offshore wind farms, in the DNV-GL codes two safety classes are considered.

- Low safety class is used for structures, whose failures imply low risk for personal injuries and pollution, low risk for economic consequences and negligible risk to human life. This level complies to CC1 in the Eurocode system.
- Normal safety class is used for structures, whose failures imply some risk for personal injuries, fatalities and pollution and significant economic consequences. This level complies to CC2 in the Eurocode system.

DNV-GL guidance note: Support structures and foundations for wind turbines are usually to be designed to the lower end of the normal safety class. Given their location on a beach accessible by the general public a level of CC2 is recommended. Although in the circumstances where the design conditions occur the beach will be flooded and strong winds will blow, which will in practice mean that there will be no risk of injuries and casualties as a consequence of structural or geotechnical failure of the structure.

The overall safety implemented in the lateral geo-structural assessment in the monopile foundation concept design step is 1.35 (Enercom factor between envelope ULS and 'normal' SLS) x 1.5 (equivalent soil resistance) = 2.0.

3 Model setup

3.1 Model overview

L-pile software (developed by Ensoft Inc.) is applied for the monopile concept design. The L-pile program provides the capability to analyse individual piles for a variety of applications in which lateral loading is applied. The L-pile program is a commonly applied and proven tool in the offshore industry.

The L-pile analysis is based on a solution of a differential equation describing the behaviour of a beam-column supported by non-linear springs (i.e. the soil p-y curves defined from the parameters in Table 2-3). An overview of the monopile concept design model is presented in Figure 3-1.

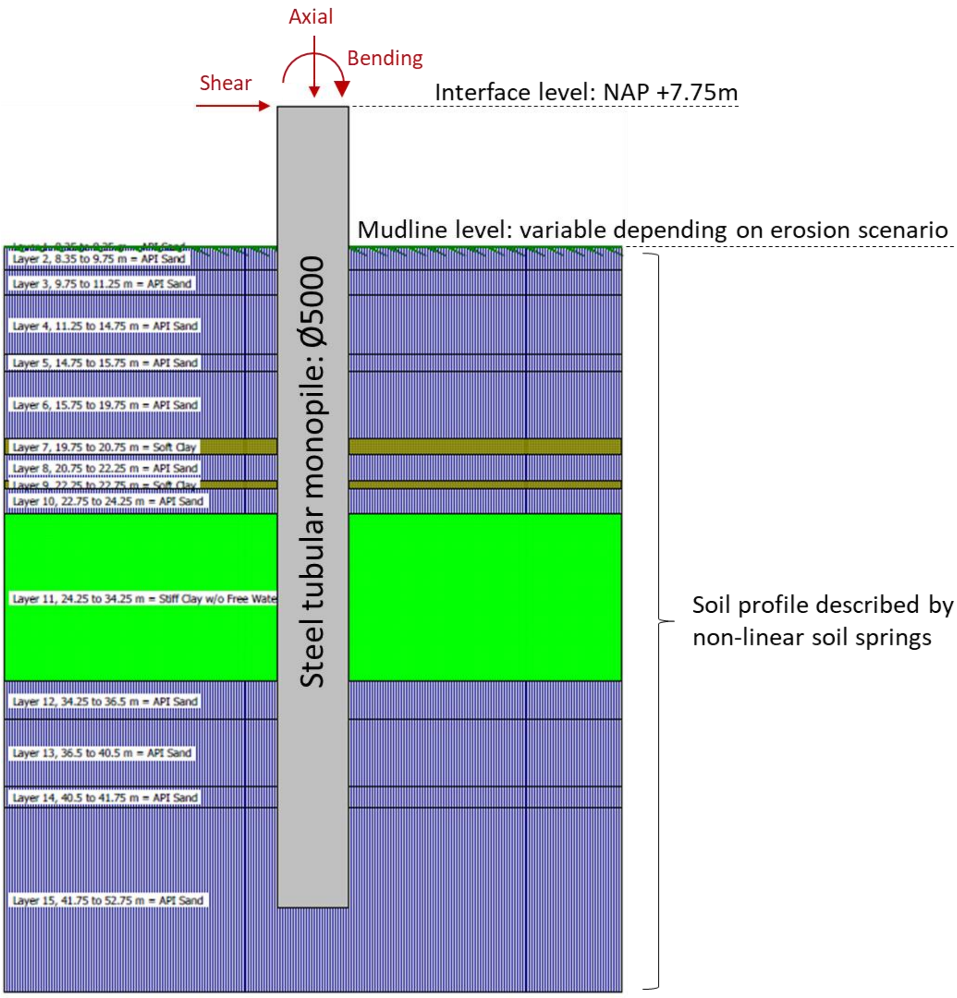


Figure 3-1: L-pile model overview for monopile concept design

It is noted that the use of the p-y curve methodology for single piles larger than 1.0m diameter should be validated with other models. This will be done during preliminary design phase.

3.2 Model input

3.2.1 Soil profiles and properties

The soil profiles introduced in the calculation model are presented in Figure 3-2 (North profile) and Figure 3-3 (South profile). The figures show the examples for the soil profiles without erosion and scour. In the erosion scenarios the top layers are accordingly removed from the calculation model. The soil properties (γ', φ, k_h for sand and γ', S_u, ε_{50} for clay) are introduced as stated in Table 2-3, resulting in the input presented in Figure 3-4.

	Select p-y Curve Type from Drop-down List	Vertical Depth Below Pile Head of Top of Soil Layer (m)	Vertical Depth Below Pile Head of Bottom of Soil Layer (m)	Press Button to Enter Soil Properties
1	API Sand (O'Neill)	8.35	8.35	1: API Sand
2	API Sand (O'Neill)	8.35	9.75	2: API Sand
3	API Sand (O'Neill)	9.75	11.25	3: API Sand
4	API Sand (O'Neill)	11.25	14.75	4: API Sand
5	API Sand (O'Neill)	14.75	15.75	5: API Sand
6	API Sand (O'Neill)	15.75	20.25	6: API Sand
7	API Sand (O'Neill)	20.25	22.75	7: API Sand
8	API Sand (O'Neill)	22.75	24.25	8: API Sand
9	API Sand (O'Neill)	24.25	25.75	9: API Sand
10	API Sand (O'Neill)	25.75	27.75	10: API Sand
11	API Sand (O'Neill)	27.75	31.75	11: API Sand
12	Stiff Clay w/o Free Water (Reese)	31.75	33.75	12: Stiff Clay without Free Water
13	API Sand (O'Neill)	33.75	34.75	13: API Sand
14	API Sand (O'Neill)	34.75	52.75	14: API Sand

Figure 3-2: Soil profile North (no erosion / scour) input in L-pile

	Select p-y Curve Type from Drop-down List	Vertical Depth Below Pile Head of Top of Soil Layer (m)	Vertical Depth Below Pile Head of Bottom of Soil Layer (m)	Press Button to Enter Soil Properties
1	API Sand (O'Neill)	8.35	8.35	1: API Sand
2	API Sand (O'Neill)	8.35	9.75	2: API Sand
3	API Sand (O'Neill)	9.75	11.25	3: API Sand
4	API Sand (O'Neill)	11.25	14.75	4: API Sand
5	API Sand (O'Neill)	14.75	15.75	5: API Sand
6	API Sand (O'Neill)	15.75	19.75	6: API Sand
7	Soft Clay (Matlock)	19.75	20.75	7: Soft Clay
8	API Sand (O'Neill)	20.75	22.25	8: API Sand
9	Soft Clay (Matlock)	22.25	22.75	9: Soft Clay
10	API Sand (O'Neill)	22.75	24.25	10: API Sand
11	Stiff Clay w/o Free Water (Reese)	24.25	34.25	11: Stiff Clay without Free Water
12	API Sand (O'Neill)	34.25	36.5	12: API Sand
13	API Sand (O'Neill)	36.5	40.5	13: API Sand
14	API Sand (O'Neill)	40.5	41.75	14: API Sand
15	API Sand (O'Neill)	41.75	52.75	15: API Sand

Figure 3-3: Soil profile South (no erosion / scour) input in L-pile

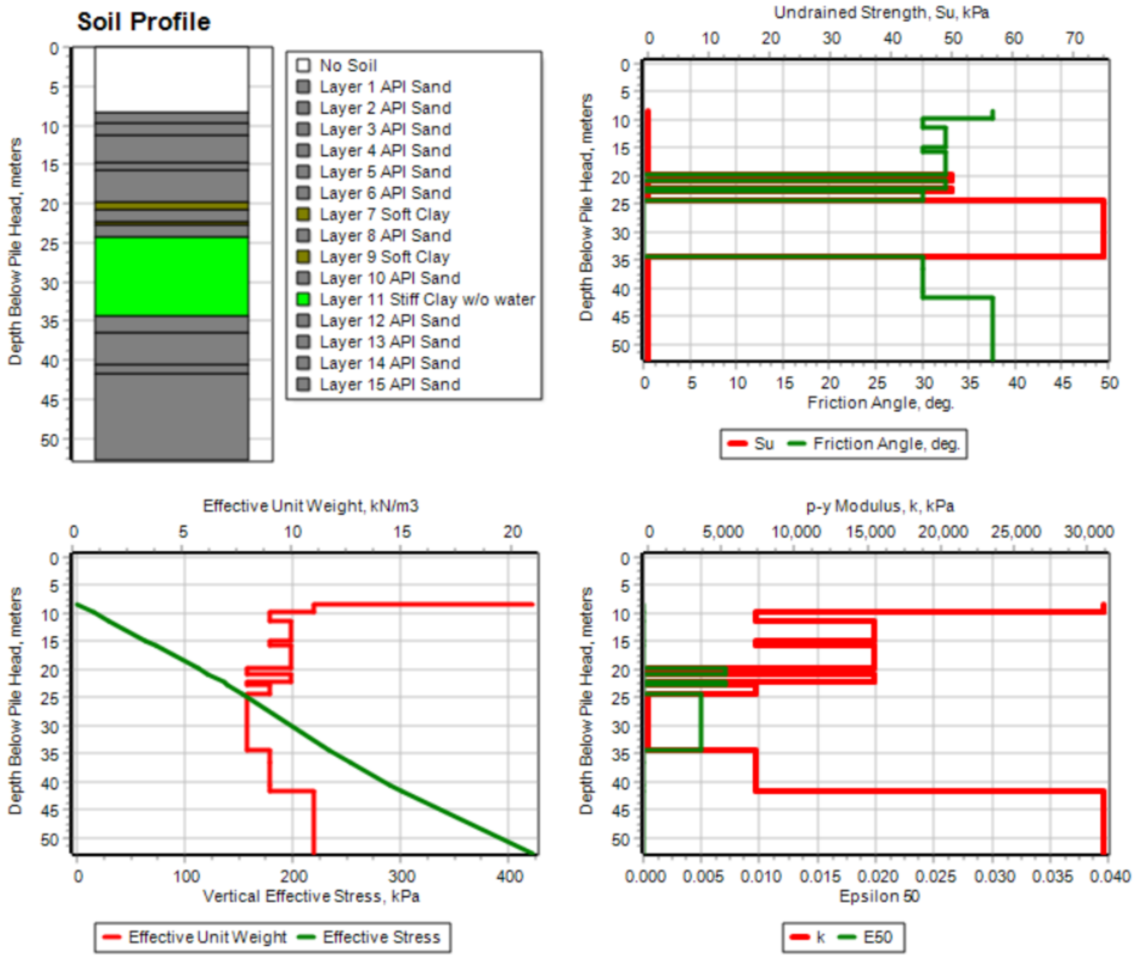


Figure 3-4: Resulting soil property input for South profile in L-pile

During the concept design phase, it was found that the overall lateral behaviour is dominated by the upper sand soils. As such the calculation results are rather similar for the North and South profile, as the top layers in both soil profiles consist of reclaimed MV2 sand. The South profile is somewhat more governing though due to thicker clay deposits (including soft clay) in the native soil stratigraphy underneath the reclamation. As such the final concept design is only assessed with the governing South profile.

3.2.2 Pile geometry

The monopile geometry properties introduced in the calculation model are presented in Figure 3-5. The monopile is an open-ended steel tubular pile with the following properties:

- Diameter = 5000 mm
- D/t ratio = 55 → Wall thickness = 90 mm → Class 3 cross-section
- Top level = NAP +7.75m
- Toe level = +7.75m – 47.75m = NAP -40m
- Steel quality = S355
- Young’s modulus = 2.1.10^{8} kN/m^{2}

Corrosion allowance of the monopiles is at this stage not yet specifically considered.

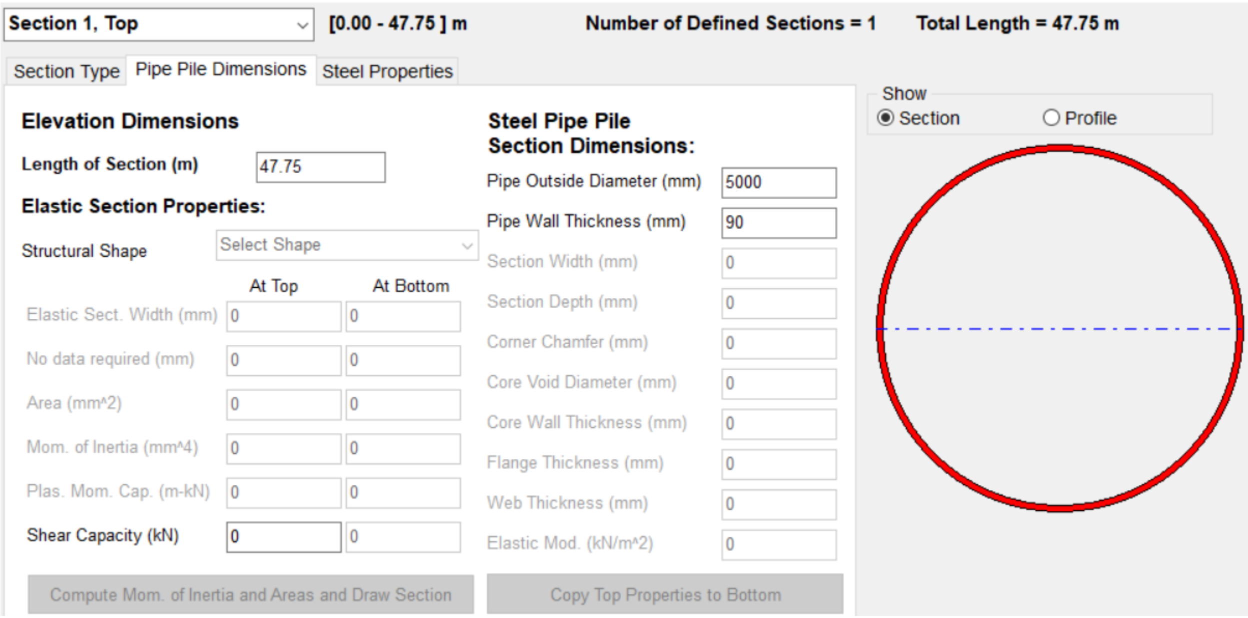


Figure 3-5: Monopile geometry input in L-pile

3.2.3 Pile loads

3.2.3.1 Deformation and structural capacity verifications

For the deformation and structural capacity verifications the concept design loads specified in Section 2.7 and 2.8 are introduced at interface level in the calculation model. The load input is presented in Figure 3-6. Load cases 1 and 3 refer to Enercom ULS and SLS loads (Table 2-5). Load cases 2 and 4 refer to GE ULS and SLS loads (Table 2-5). The ULS shear force values in load cases 1 and 2 consist of the turbine shear forces added with the 2000kN wave load.

Load Case	Pile-Head Loading Condition	Condition (1) for Loading Type	Condition (2) for Loading Type	Axial Load (p-delta) (kN)	Compute Top y vs. L?
1	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	3878	228911	12848	Yes
2	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	3717	158776	8695	Yes
3	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	1391	166806	8727	Yes
4	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	1240	106532	5885	Yes

Figure 3-6: Interface load input in L-pile for deformation and structural capacity verifications

3.2.3.2 Geotechnical stability (push-over) verifications

For the push-over analysis the lateral interface loads are stepwise increased until the software reaches non-convergence indicating instability / failure of the soil-structure system. Both the shear and bending interface loads are increased with a constant amount. Based on the available turbine load data in **Error! Reference source not found.**, an average ratio of 100:1 is assumed between shear and bending. The axial compression load is kept at a constant value. This approach is visualised in Figure 3-7.

Shear ← Factor 100 → Bending

Load Case	Pile-Head Loading Condition	Condition (1) for Loading Type	Condition (2) for Loading Type	Axial Load (p-delta) (kN)	Compute Top y vs. L?
1	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	0	0	8000	Yes
2	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	500	50000	8000	Yes
3	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	1000	100000	8000	Yes
4	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	1500	150000	8000	Yes
5	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	2000	200000	8000	Yes
6	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	2500	250000	8000	Yes
7	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	3000	300000	8000	Yes
8	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	3250	325000	8000	Yes
9	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	3500	350000	8000	Yes
10	(1) Shear [lb or kN] and (2) Moment [in-lb or kN-m]	3750	375000	8000	Yes

Constant value

Figure 3-7: Interface load input in L-pile for push-over analysis

4 Verification results

4.1 Deformation behaviour

The static target rotational stiffness values provided by Enercom and GE are verified against the respective SLS load conditions listed in Table 2-5, assuming the scenario without erosion. It is not specifically stated by the suppliers at which level the target rotational stiffness values are defined. It is expected that the values are defined around mudline level for a regular shallow foundation system. Nonetheless the target stiffness verification is in this design phase performed for both interface level and NAP level, with the interface level verification being more conservative. For verifying the target rotational stiffness at NAP level the bending moment values at interface level are lowered as such that the SLS loads at NAP level match the SLS loads listed in Table 2-5. The verification result is presented in Figure 4-1.

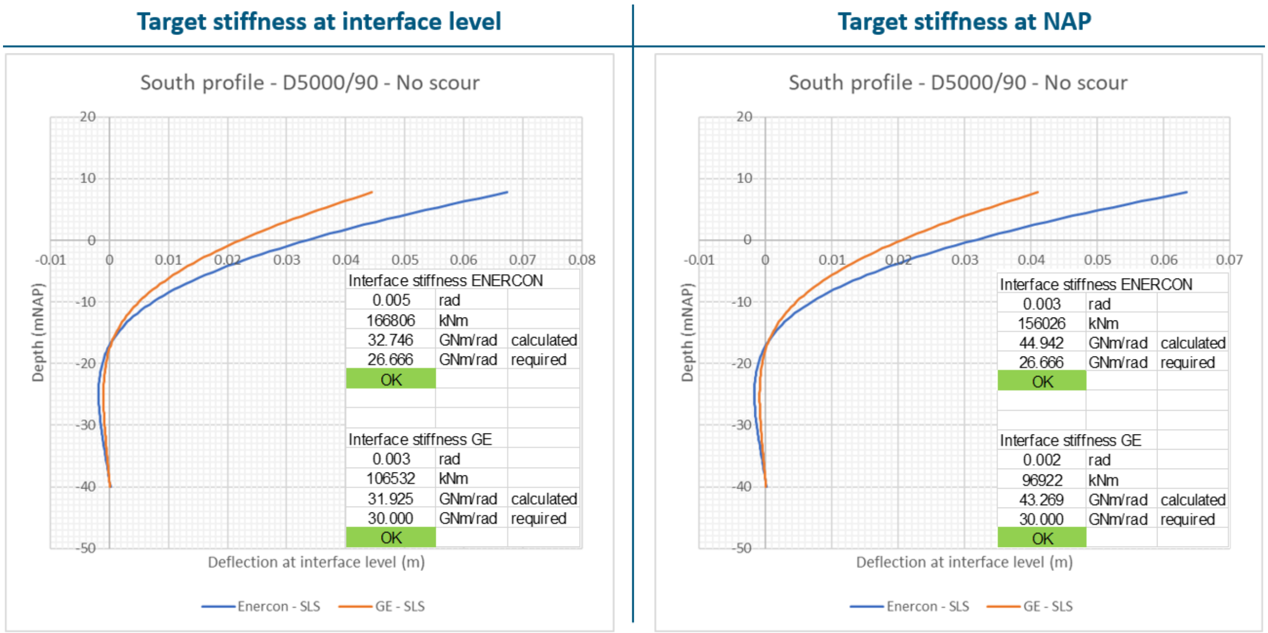


Figure 4-1: Static rotational stiffness verifications for D5000/90 monopile in South profile

It is concluded from Figure 4-1 that the soil-pile system with the D5000/90 monopile reaches sufficient static rotational stiffness:

- Static rotational stiffness at interface level:
  - Enercom: $k_{\varphi,calculated}$ = 32.7GNm/rad > $k_{\varphi,required}$ = 26.7GNm/rad → OK
  - GE: $k_{\varphi,calculated}$ = 31.9GNm/rad > $k_{\varphi,required}$ = 30.0GNm/rad → OK
- Static rotational stiffness at NAP level:
  - Enercom: $k_{\varphi,calculated}$ = 44.9GNm/rad > $k_{\varphi,required}$ = 26.7GNm/rad → OK
  - GE: $k_{\varphi,calculated}$ = 43.2GNm/rad > $k_{\varphi,required}$ = 30.0GNm/rad → OK

In addition it can directly be observed from the deformation graphs plotted in Figure 4-1 that the SLS lateral deflection around mudline level is sufficiently low: $u_{ENERCOM}$ = 3cm < 15cm (=3% of D_{pile}) → OK

The monopile overall inclination is estimated from the deformation plots in Figure 4-1 by calculating the inclination over the pile length between interface level and the first level of zero lateral deformation in the soil. As such an equivalent inclination of 2.5mm/m is estimated. This value is in between the normal operational and extreme operational SLS requirements (1mm/m and 3mm/m respectively, see Table 2-6).

As explained in Section 2.7 it is at this stage not yet sufficiently clear how the SLS 'normal design conditions' provided by the suppliers relate to the SLS normal operational and SLS extreme operational load conditions defined for the inclination requirement. The inclination requirement needs further assessment in the preliminary design phase.

4.2 ULS structural capacity

The ULS structural capacity of the monopile cross-section is verified against the maximum internal forces computed for the ULS load conditions listed in Table 2-5 for the following two outer ULS scenarios:

- No erosion and scour (mudline at NAP -0.6m) without cyclic degradation of soil springs
- Full erosion to NAP -7.0m with full cyclic degradation of soil springs

The internal force results for the two scenarios are plotted in Figure 4-2 and Figure 4-3 respectively.

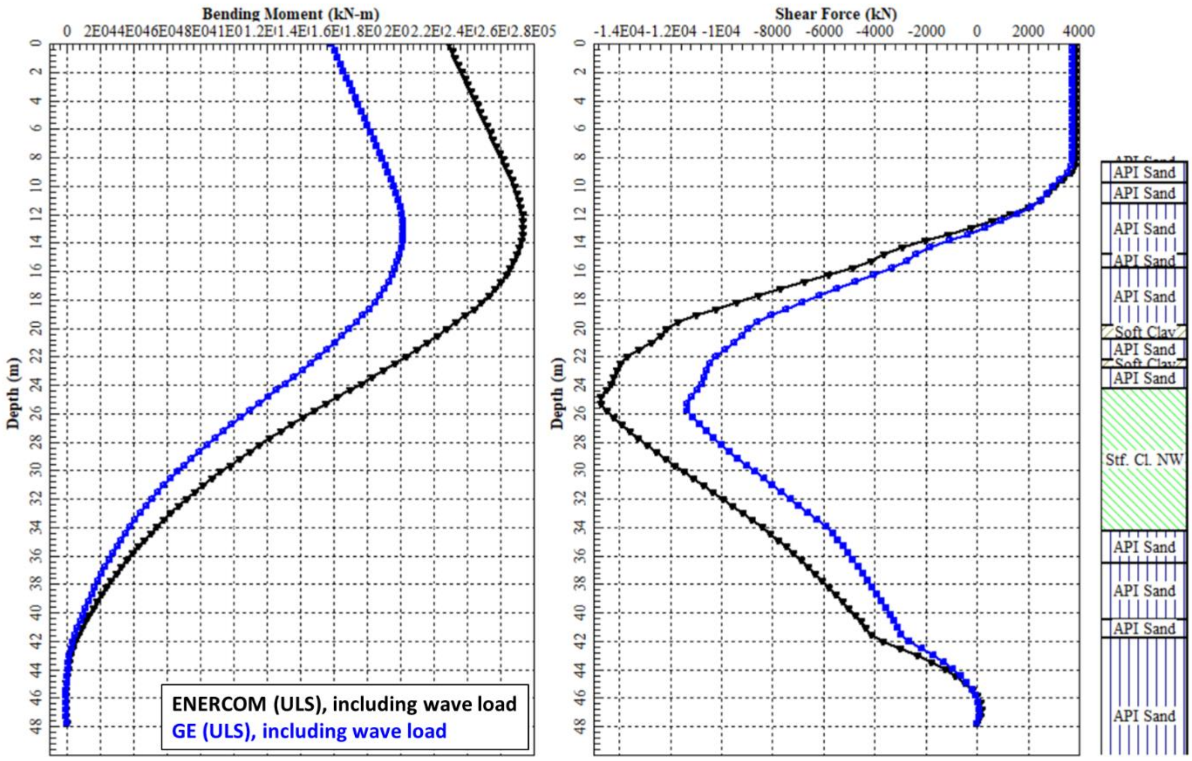


Figure 4-2: ULS internal forces for D5000/90 monopile in South profile for no-erosion scenario without cyclic degradation

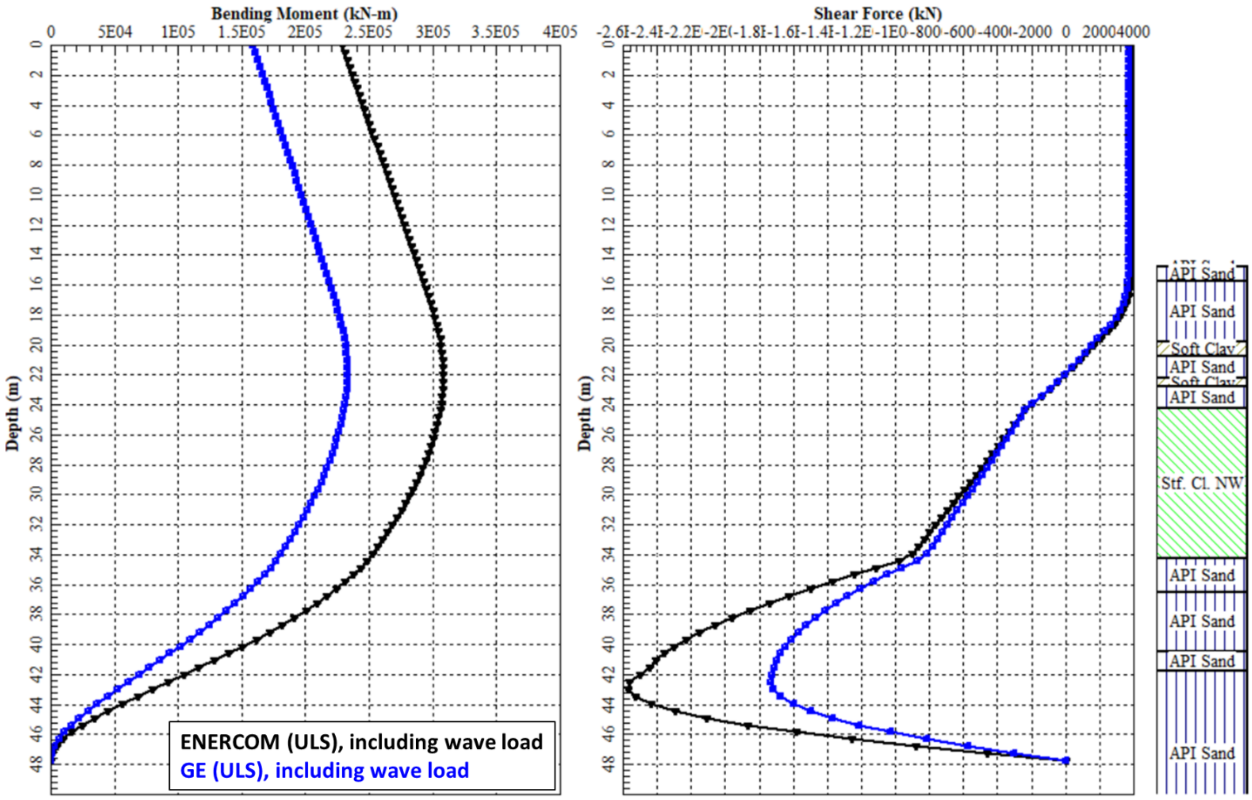


Figure 4-3: ULS internal forces for D5000/90 monopile in South profile for full erosion scenario (NAP -7m) with cyclic degradation

The internal force verifications according to Eurocode 3 ([6], [7]) are presented in Table 4-1. The structural capacity verification is elastic as the D5000/90 cross-section with S355 steel quality falls within Class 3. The yield stress is reduced from 355N/mm^2 to 315N/mm^2 because of the large wall thickness. The full calculation sheets are documented in Appendix 1.

Table 4-1: ULS structural capacity verifications according to Eurocode 3 for D5000/90 monopile in South profile

Internal force results	No erosion / scour (NAP -0.6m) without cyclic degradation	General erosion to NAP -7.0m with cyclic degradation
Maximum bending moment [kNm]	275000	310000
Maximum shear force [kN]	15000	26000
Maximum axial force [kN]	13000	13000
Torsion load [kNm]	19500	19500
Unity Check on bending moment	0.52	0.59
Unity Check on shear force	0.09	0.16
Combined Unity check	**0.55**	**0.59**

Although the presented unity checks are comfortably below 1.0, the target utilisation of 40% (to provide margin for fatigue and uncertainty in the soil) is not reached. If the wall thickness would be locally increased to 100mm the cross-section becomes a Class 2 profile. The locally thickened Class 2 cross-section has sufficient capacity to reach the 40% utilisation criterion.

4.3 ULS geotechnical stability (push-over and minimum pile toe level)

The push-over curves for the various erosion scenarios are presented in Figure 4-4. In the top figure the interface shear force is plotted against the interface horizontal deformation. In the bottom figure the interface bending moment is plotted against the interface horizontal deformation. For every scenario the push-over curves excluding and including the equivalent soil resistance factor of 1.5 are shown (except for the abnormal scenario with full erosion to NAP -7m with lacking maintenance). For the scenario with full erosion to NAP -4m the push-over results with both static and cyclically degraded soil springs are shown. In both figures the turbine envelope ULS load demands are plotted as well.

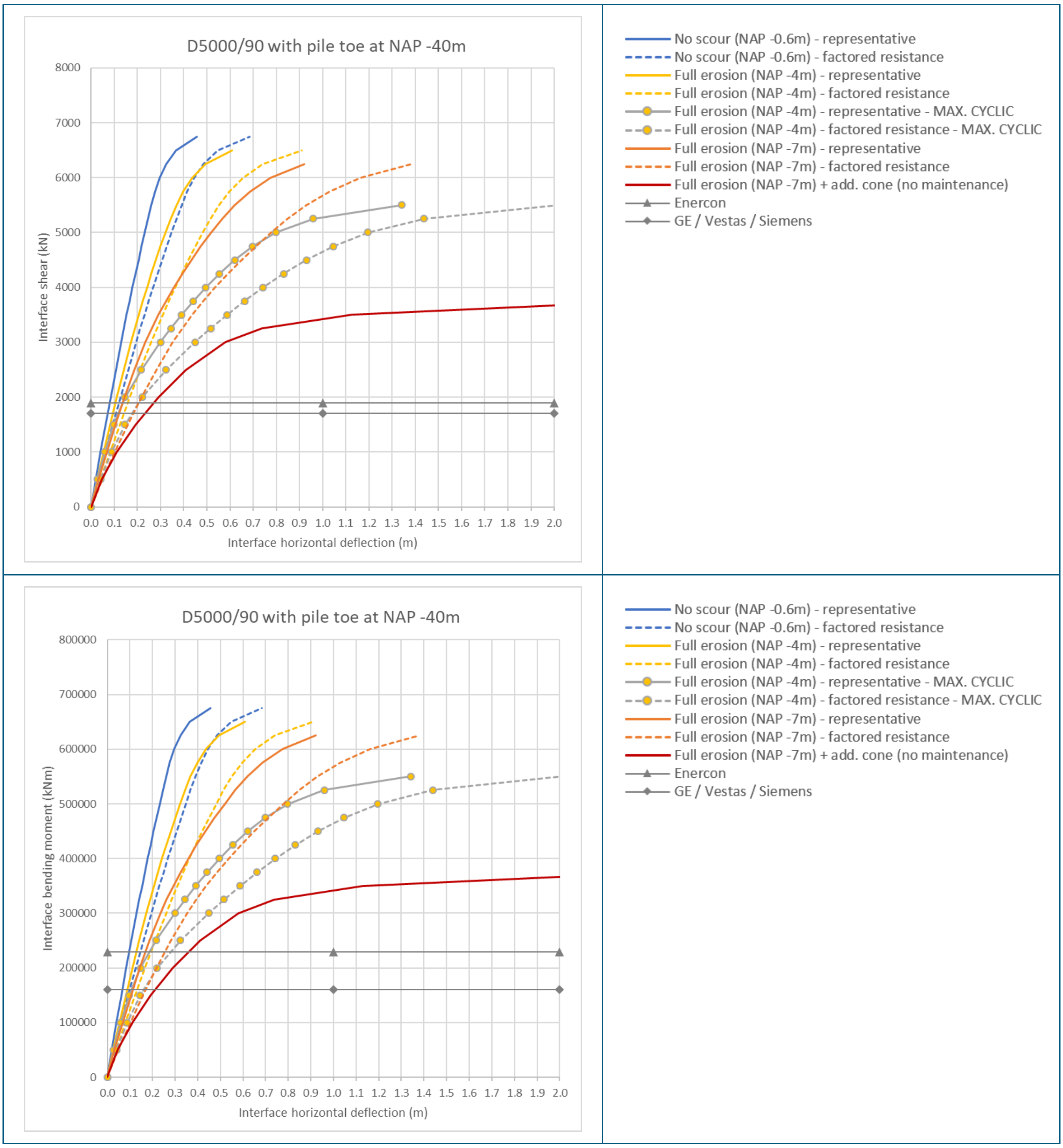


Figure 4-4: Push-over curves at interface level for D5000/90 monopile in South profile

In Figure 4-5 the push-over curve computed at mudline level is presented as well. The figure plots the interface shear force against the mudline horizontal deformation

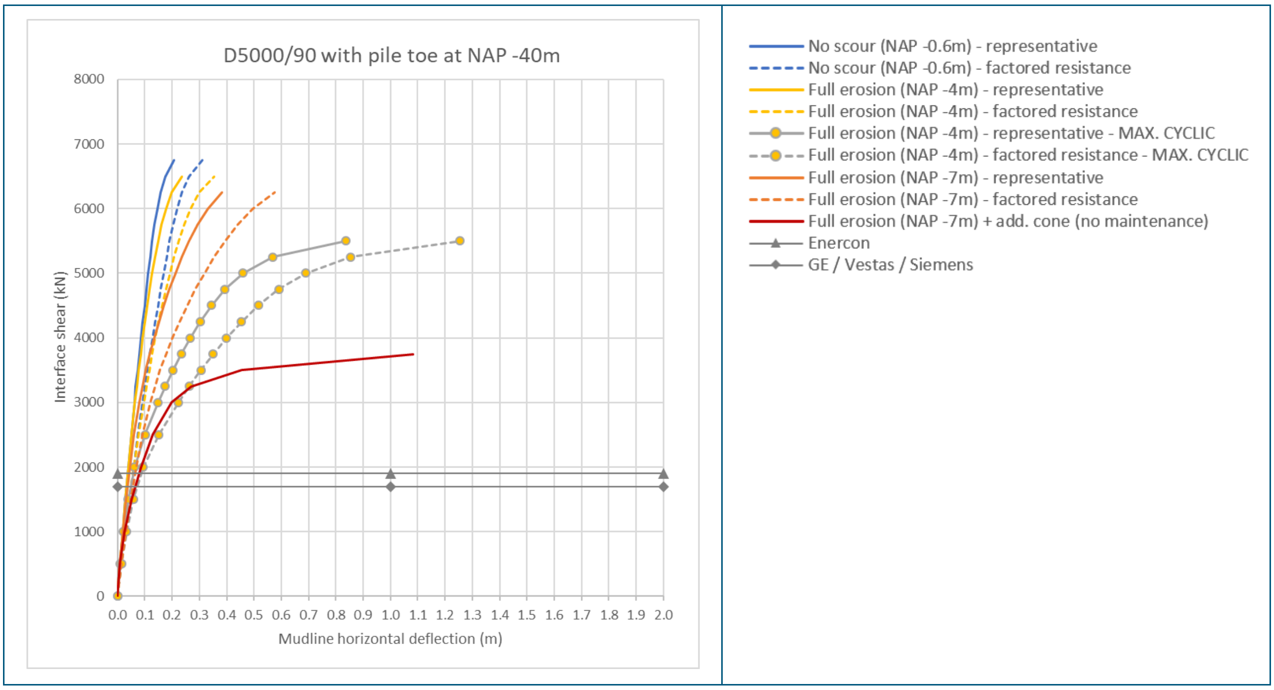


Figure 4-5: Push-over curves at mudline level for D5000/90 monopile in South profile

The following is concluded from the push-over curves presented in Figure 4-4 and Figure 4-5:

- The studied soil-pile system with the D5000/90 monopile is sufficiently stable under the ULS turbine load demands, even in the abnormal scenario with full erosion to NAP -7m and an additional scour hole around the pile due to lacking maintenance. The ULS load demands cross the push-over curves in the branch which is predominantly elastic due to limited plasticity in the soil.
- The abnormal scenario with full erosion to NAP -7m and successive lacking maintenance should be prevented. It can be derived from the top graph in Figure 4-4 that the system in that specific scenario would not have sufficient residual capacity to resist the additional incoming wave load of 2000kN.

The push-over curves have been calculated for a pile toe level at NAP -40m. In addition, a separate set of calculations is performed to assess the minimum required pile toe level for realising sufficient stability. For every scenario the ULS loads (including wave shear) as listed in Figure 3-6 are used. The result is presented in Figure 4-6 which plots horizontal deformation at interface level against pile toe level per erosion and load scenario. The pile is considered stable when the curve becomes a vertical line. The results do yet include partial safety on soil resistance.

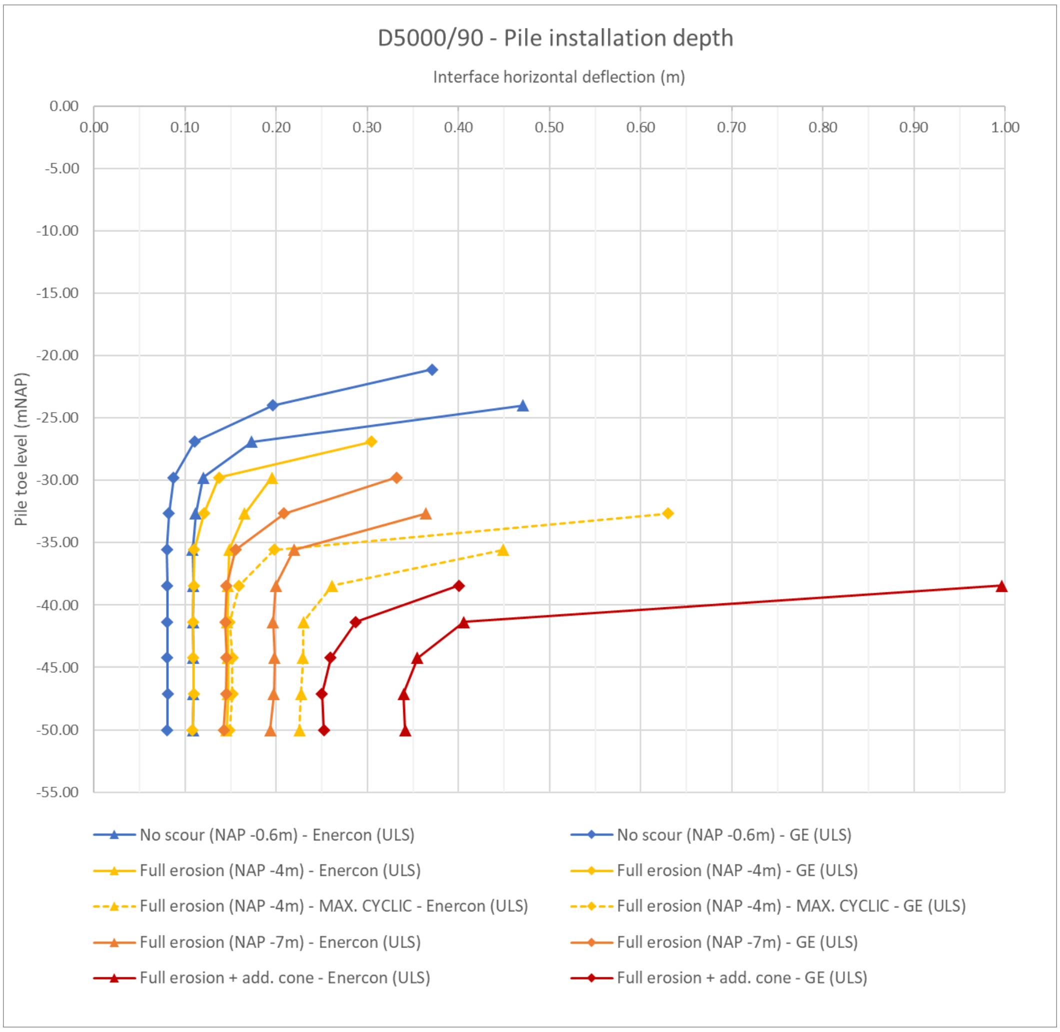


Figure 4-6: Minimum pile toe level assessment for D5000/90 monopile in South profile

Based on the set of observations in Figure 4-4, Figure 4-5 and Figure 4-6 minimum pile toe levels are recommended which include sufficient margin for cyclic degradation, local scour and uncertainty in the soil resistance:

- Mudline at NAP -0.6m → minimum pile toe level at NAP -40m
- Mudline at NAP -4.0m → minimum pile toe level at NAP -45m
- Mudline at NAP -7.0m → minimum pile toe level at NAP -50m

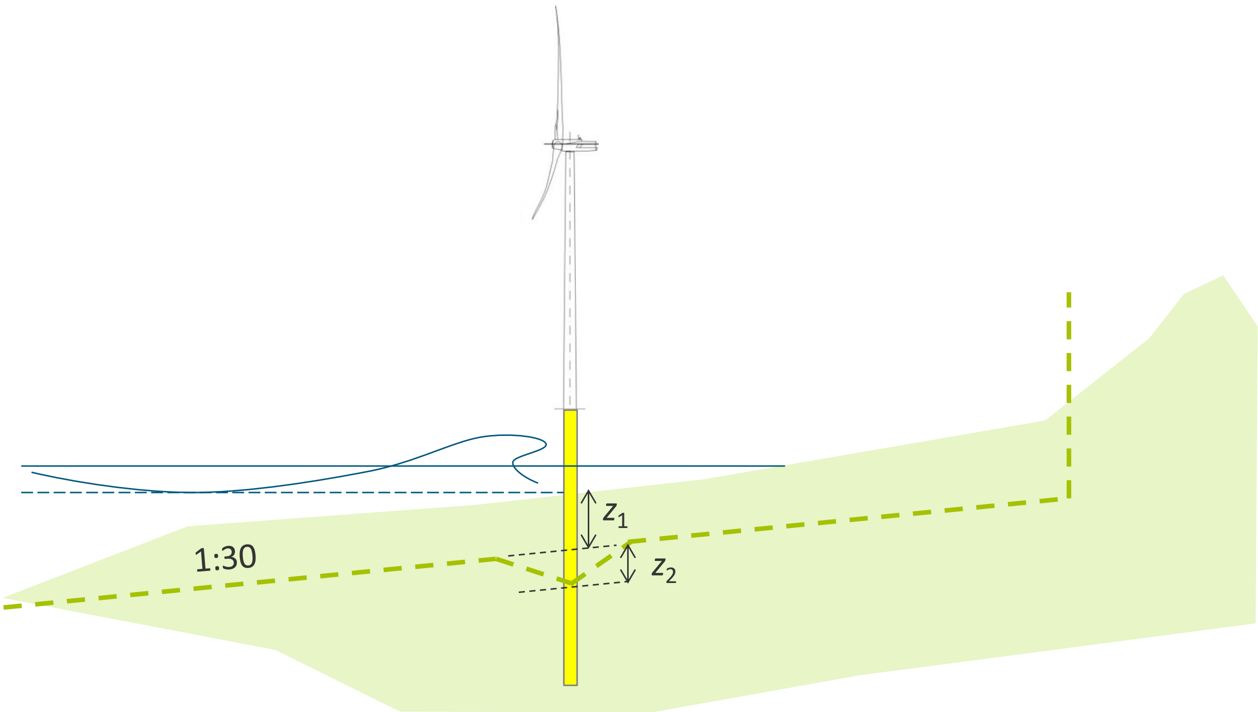
These values are based on calculations for the South soil profile. It should be noted that the larger scour risk occurs at locations ZZ1 to ZZ7, for which the North coil profile applies.

These values are further clarified in Table 4-2.

Table 4-2: Clarification of recommended minimum pile toe levels

General Erosion scenario	Pile toe level from static calculation	Pile toe level with cyclic degradation	Margin for local scour, soil disturbance and uncertainty in the soil	Minimum pile toe level recommendation
NAP -0.6m	NAP -30m	NAP -35m (estimated)	5m (estimated)	NAP -40m
NAP -4.0m	NAP -35m	NAP -40m (calculated)	5m (estimated)	NAP -45m
NAP -7.0m	NAP -40m	NAP -45m (estimated)	5m (estimated)	NAP -50m

These values are based on calculations for the South soil profile. It should be noted that the larger general beach erosion and larger scour risk occurs at locations ZZ1 to ZZ7, for which the North soil profile applies.

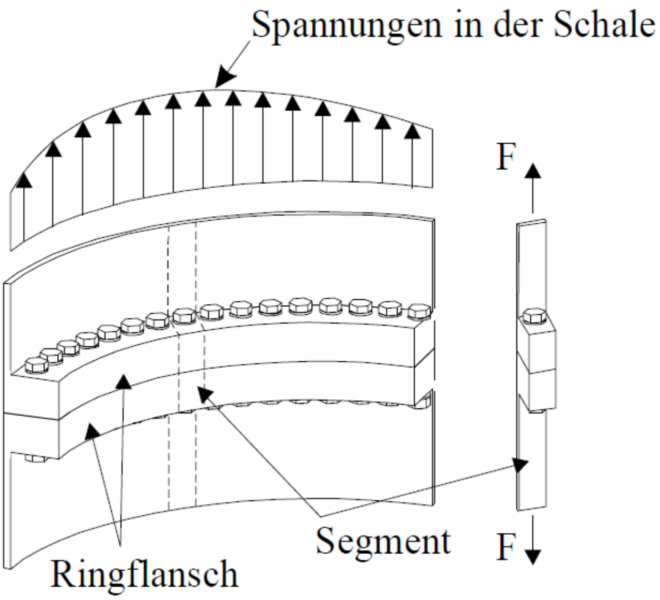


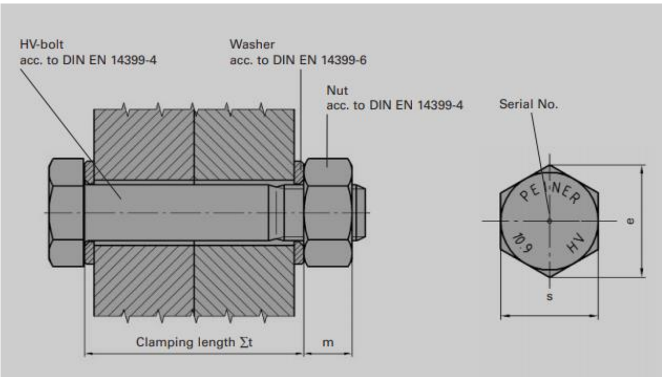
4.4 Conceptual verification of the bolted connection

Mono-pile tower connections in onshore situation shall have bolted connections. The accessibility of the location, and also the possibilities to give the monopile an interface with a flange, accurately dimensioned and levelled respecting strict tolerances lead to this preference.

Specific for the MV2 location close to the seashore and in permanent salt spray, the connection is ideally design as an interior flange, as indicated in the figure below. The flange can be designed with one bolt rows, dependent on the pile diameter, the wall thickness and stress levels in the steel shell.

For our case it appears that a connection with one bolt row is feasible.





The bending moments in the flange connection at the level of (approx. +7 m NAP) are:

	M_{Ed_0} [kNm]	V_{Ed} [kN]	M_{Ed_+7} [kNm]	Diameter [m]	Tension force = M_{Ed+7} / (π/4 x D^2) [kN/m]
GE	158776	1717	146757	5.0	7474
Vestas	154600	1474	144282	5.0	7348
Enercom	228911	1878	215765	5.0	10989
Siemens	153017	1509	142454	5.0	7255

The difference between ULS load and SLS load is a factor 1.5. With the SLS load the elastic cyclic capacity is verified. For the concept design the maximum load will be check, which corresponds to the Enercon available load sets.
The calculation below shows that one row of bolts is feasible, in the preliminary design stage the design of the bolts and flange can be optimize.

Used bolts:

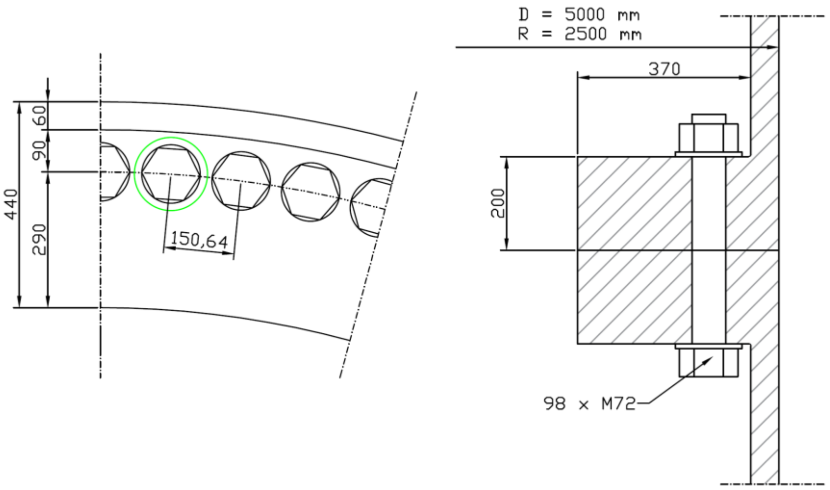
M72 10.9 (A_s = 3460mm²).
The bolt force capacity is:
$F_{t,Rd} = 0.9\, f_{ub}\, A_s \,/\, \gamma_{M2}$

$F_{t,Rd.\ M72}$ = 0.9 · 1000 · 3460 / 1.25 = 2491 kN

To tighten the bolt M72 the following equipment can be used: D-Flex 18, with a minimum head radius R = 78 mm. A tolerance of 10 mm will be added.

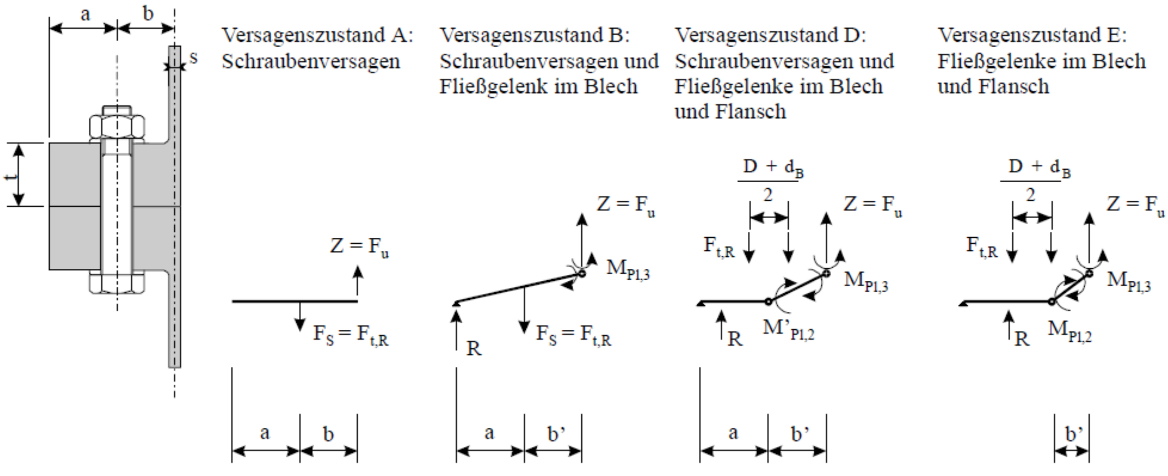
The minimum c.t.c. distance of the bolts is R + e/2 + tolerance
e = outer bolt diameter
ctc of an M72 = 78 + 125/2 +10 = 150.5 mm

The bolts will be placed as showed in picture below:



The green circle is the minimum required space to tighten the bolts.
The thickness of the flange plate is 200 mm, with $f_{y,flange}$ = 285 N/mm² (S355 for thick plates)

The ULS verification of the connection is done with formula for the bolt force capacity and the plastic bending capacity of the flange. Refer to [9].



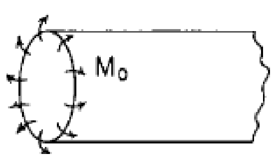
Versagens-mechanismus	A	B	D	E

In the concept design step failure mode B is verified, since there is freedom to choose for a thick plate. The failure load is computed with:

The SLS verification is done for the diameter 5.0m and F_{Ed} = 10989 kN/m, for the case with bolts M72. For the SLS check a FEM model is used. The model consists of two beams, connect together with only non-tension springs. The bolts are modelled as tension-only members.

At the shell end there is a flexible rotation support, with the stiffness calculated with:

10. End moment, M_o lb-in/in

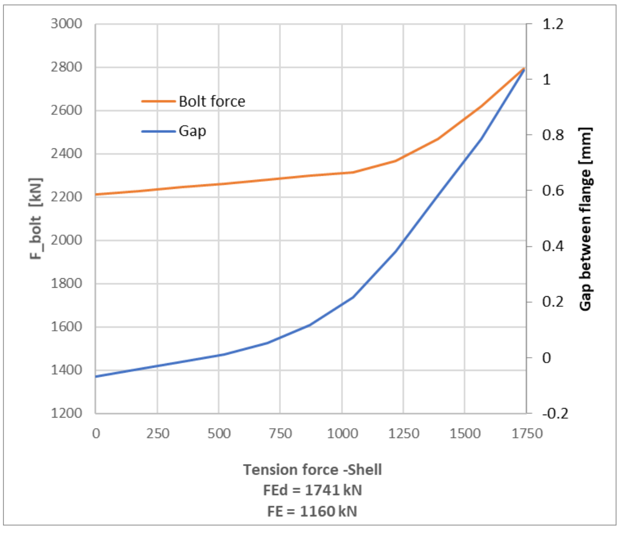


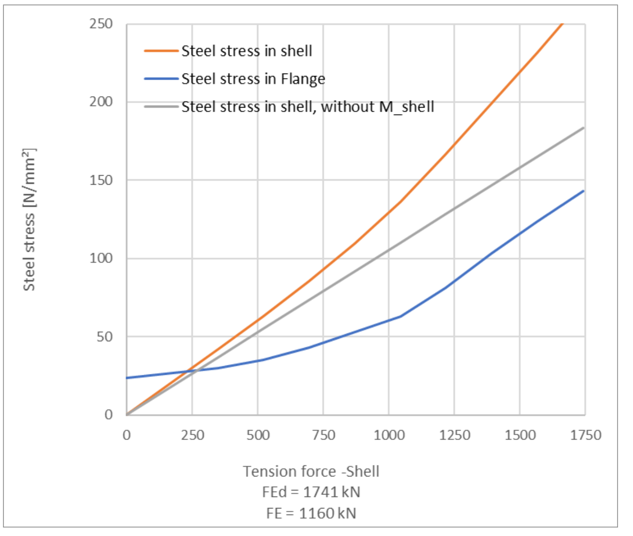
$$\psi_A = \frac{-M_o}{D\lambda}$$
$$y_A = \frac{M_o}{2D\lambda^2}$$

The bolt preload is modelled as a load.
The maximum preload is: $F_{p;\,M56}$ = 0.7 · 1000 · 3460 = 2422 kN
To account for losses: $F_{p'}$ = 0.9 · 2422 = 2280 kN

The loads in the flange is step by step increased from 0 to F_{Ed} = 10989 · 0.1584 = 1741 kN
0.1584 = length of circle segment of one bolt.

The results are presented in two graphs, the left one sowing the bolt force and gap development, both as a function of the force in the shell. The right-hand figure resents the bending stress in the flange and the membrane and bending in the shell.

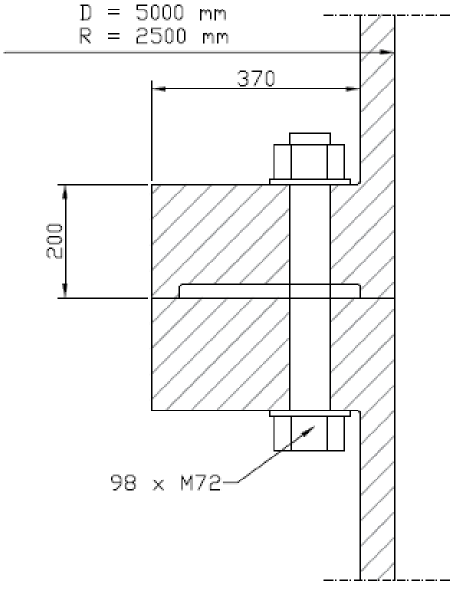




The ratio of increase in bolt force per tension force in the shell is a measure of efficiency and ultimately for fatigue resistance of the connection.
A critical element in this connection is the bending stress in the shell. When choosing this connection, the membrane stress in the shell is always supplemented with approximately 15-30% bending stress, which might require a zone with an increased wall thickness at the foot of the tower and at the top of the monopile. This should be taken in account in the preliminary design stage.

Alternatively, to reduce these bending stresses the flange thickness can increased, or the contact area can be reduced, as indicated in the sketch below.

Vertical stiffeners could be used, but this will reduce the available space and possible number of bolts and will hence reduce the capacity of the connection. This is not recommended.



5 Transport, installation and grid connection

5.1 Transport and installation

The monopiles will consist of hot rolled steel bend plates, jointed by full penetration welding. As the fabrication and welding will largely determine the cost of the monopile, the steel design will be optimized in the next design step. This will likely result in a pile design with segments of different wall thickness and steel grade over the height of the pile.

The monopiles are envisaged to be transported and delivered by onshore methods, in a horizontal position. Due to their large diameter and the circular cross-section is at risk of ovalization. In the preliminary design phase, we will separately analyse this situation and make the necessary recommendations for the suppliers in order to minimize the impact on the structural shape. Any deformations occurred during this stage can influence the drivability of the monopile, the connection to the tower and can introduce secondary effects in the structure.

The installation segments and connection welds will be carefully placed outside the high stress regions. If required, the regions of the on-site welds will be separately checked and evaluated for any fatigue assessment and to avoid weld failure.

Temporary installation platforms will be constructed to accommodate the installation machinery and to allow access for the monopile onsite delivery. The piles will be installed with cranes operating on a platform at +3.5 m NAP, presently foreseen to be constructed as a cofferdam of sheet pile walls with a sand fill.

The maintenance, operation and security considerations dictate the monopile to be equipped with a working platform at interface height, to avoid accessibility of the tower by the general public. The permanent access platform of the tower will be equipped with crane to lift the necessary goods/ machinery at this level. Turbine installation and maintenance personal can access the tower through high vehicles and secured access ladders starting at 4 m above beach level.

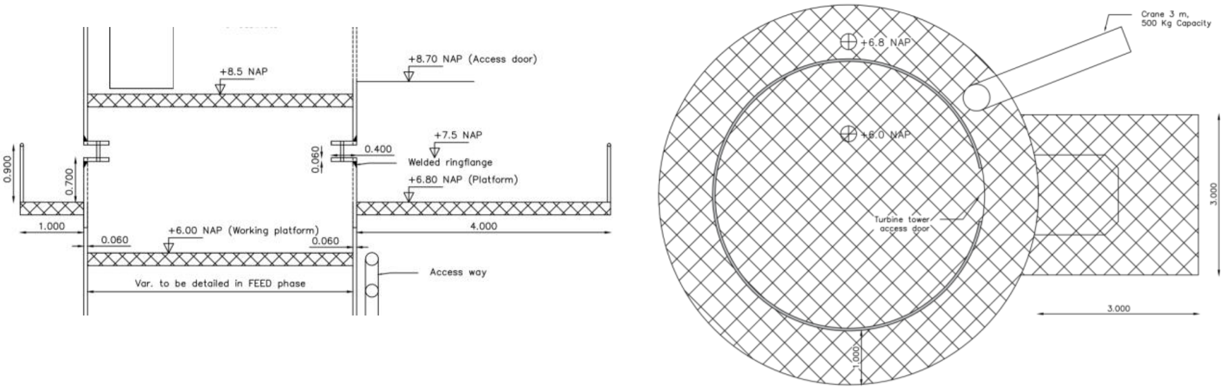


Figure 5-1:Access platform – interface height

After pile driving the monopile will be cut and levelled. A flange will be welded to the top edge of the steel wall of the circular pile, to allow a bolted connection of the first tower segment.

5.2 Pile driving

The estimated weight of the monopile is 300 to 350 ton. The pile will be installed in 45 m soil, partly consisting of dense sand layers. The steel tubular piles are typically open ended, which for large diameter pile will mean that that soil column inside the pile will more or less remain in position during installation.

For pile installation the following methods are considered:

- The common method used to drive monopiles into the seabed is hydraulic impact piling (hammering). The advantage is that it provides a verification of ground conditions related to bearing capacity and that it avoids adverse effects on the bearing capacity which are associated with other techniques as low frequency vibrating, water jetting etc. The hammering technique has two disadvantages: the noise levels during pile driving (which is on issue under-water, not so much in air), and the high stress peaks in the monopiles induced by the hammer. Pile hammering is normally not associated with high vibrations and with risks of damage to adjacent structures or of failure of unstable soil slopes.

- The second method is the use of vibratory hammers. This technique is based on local fluidizing and temporarily weakening of the soil. The feasibility and suitability of this method depends on a variety of parameters. Low frequency vibration for a pile of this size is considered a risk for the stability of adjacent structures, including the temporary work platform.

- The third method to consider is a novel technique, known as gentle pile driving. This technique is based on simultaneous application of low-frequency and high-frequency vibrators exciting two different modes of motion on the monopiles. This method is called “gentle” for its envisaged capability to reduce the driving loads and to reduce the noise levels. This method is being developed and aimed at finding a balance maintaining the penetration speed and the soil bearing capacity normally guaranteed when the classical pile hammering is applied. First field trials (on relatively small sized monopiles) were carried out at the Maasvlakte II in November 2019, and although the results were promising the technique must still prove itself for larger pile diameters in a variety of ground conditions including those at the ZZ-windturbine locations. The feasibility of this method is promising but uncertain at the moment. It is however very well possible that the technique is fully developed and timely available for use in this project.

The pile driving analysis in this concept design step is aimed to assess optimum installation options using vibratory hammers and/or impact hammers able to install the piles to their target depths with acceptable stress levels and with a minimum impact on the environment, the stability of the beach, the hard sea defence, the dunes and the temporary work platform of the project.

We aim to have at least two feasible and permitted methods available. Both methods shall not have a negative and unacceptable impact on the stability of the beach, the hard sea defence, the dunes and the temporary work platform of the project.

The present analysis results can be summarized as follows:

A Dieseko vibro hammer PVE 500M with clamps 350T can install the pile at 18 m penetration depth. CPT 11 was the CPT that is most limiting.

- Total hammer weight in air inclusive clamps, excluding hoses etc: approx. 41 ton
- Maximum width is approx. 5.1 m and max height is approx. 3.4 m

An IHC Hydrohammer S-1200 is considered able to install the pile at full depth

- Total hammer weight in air including ram, excluding anvil and pile sleeve: approx. 140 t
- Length of hammer is approx. 14.3 m, excluding anvil and pile sleeve
- Extra weight for pile sleeve at least approx. 60 t
- Extra length approx. 4 m or more

The installation with the IHC hammer is considered as having the lowest risk profile with regard to disturbance of the soil conditions and adverse effects on the bearing capacity, and for the adjacent structures.

5.3 Grid connection

5.3.1 General concept sketch

The foundation of the turbines on the SSD are offshore type of foundations. The grid connection is in offshore projects normally done at seabed level using J-tubes, which is not an acceptable choice for on on-shore project. Hence, the grid connection will hence have to be done in a unique manner. The location on the beach, in a constant dynamic environment from a morphological point of view, impacts the traditional way in which wind turbines are connected to the grid.

Starting from the recommendations of DNVGL-RP-0360 and in the same time fulfilling the safety requirements on generally accessible areas, it has been decided to elevate the power cables form the grid through the inside of the monopile. Allowance for the cables to pass is made at -5.00m NAP (average insertion centred at that level). Two groups of holes have been envisioned to accommodate incoming and outgoing cable groups. On each side 3 x 250mm diameter for the electrical current cables and two smaller ones for internat. Further details are shown in Figure 5-2: Grid connection

In order to counteract the loss of strength in the cross-section local thickened plates are to be welded with a minimum width of 1m and a height of 3m. These areas will be carefully analysed in the next project phase as these regions become prone to stress concentrations and become thus subject to Fatigue Analysis.

The cable is to be buried in the sand at a safe distance, based on the final erosion and morphological studies. The cables are to be lifted through the inside of the pile by means of a hoisting system situated on the platform from the interface level. The hoisting system and the platform will be designed to support the self-weight of this equipment, cable and tension force in the cable.

The hole cuts in the monopile are considered to be one at 45 degrees inclination and pre-drilled. After cable installation the holes are required to be sealed by means of rubber materials or epoxy resins.

In section 5.3.2 are presented the main steps envisioned for the cable installation.

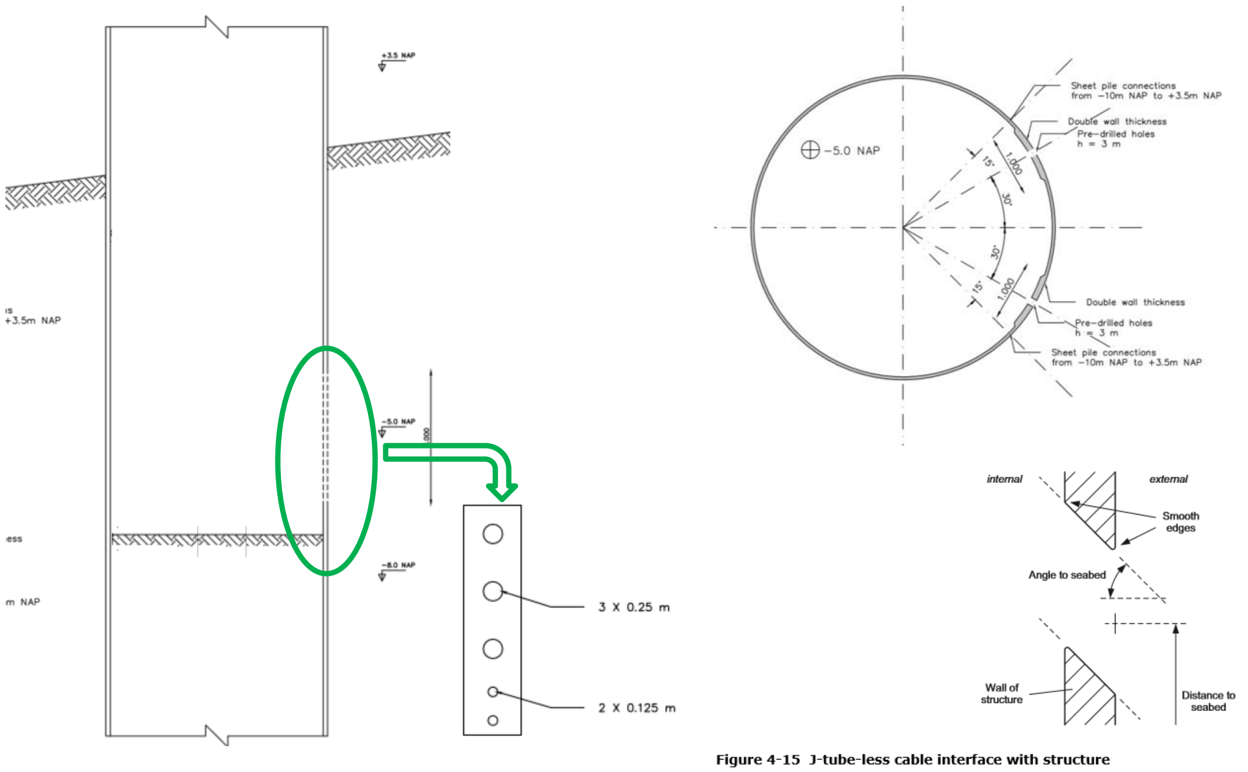


Figure 5-2: Grid connection provisions; DNVGL-RP-0360 fig 4-15 reference

5.3.2 Construction sequencing

For the installation of the electrical cables and connection to the grid several steps are necessary which are also not standard in the wind energy industry.

- Step I: After the monopile installation, temporary sheet piles and props will be installed around the pile to secure the area for excavation around the pile. A layer of underwater concrete will be installed at one side as indicated on drawing BG8375-RHD-ZZ-XX-DR-S-0004-2of2. The underwater concrete is designed a mass concrete, connected to the steel sheet piles. No tension piles and envisaged.

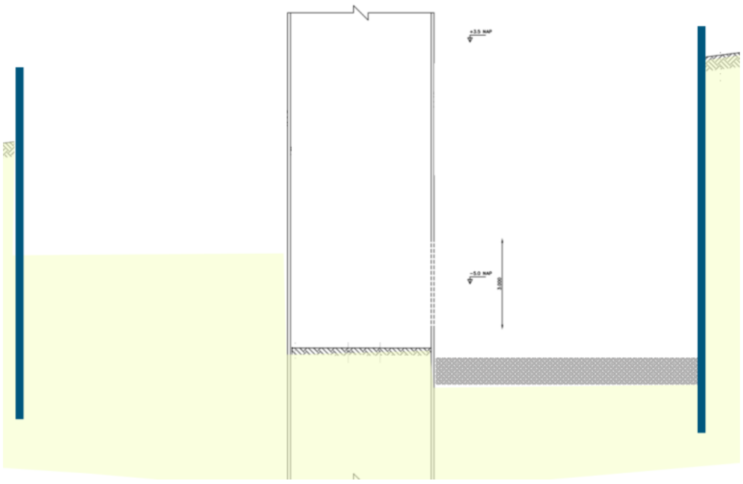


Figure 5-3: Step I Excavation and deep scour protection

- Step II: After the 1st layer of deep scour protection, the cables will be installed and pulled inside the monopile. The bundle will be sequentially split, so as just the cables without the protective tubes will be pulled inside the monopile. The aim is to minimize the section loss in the monopile shell.

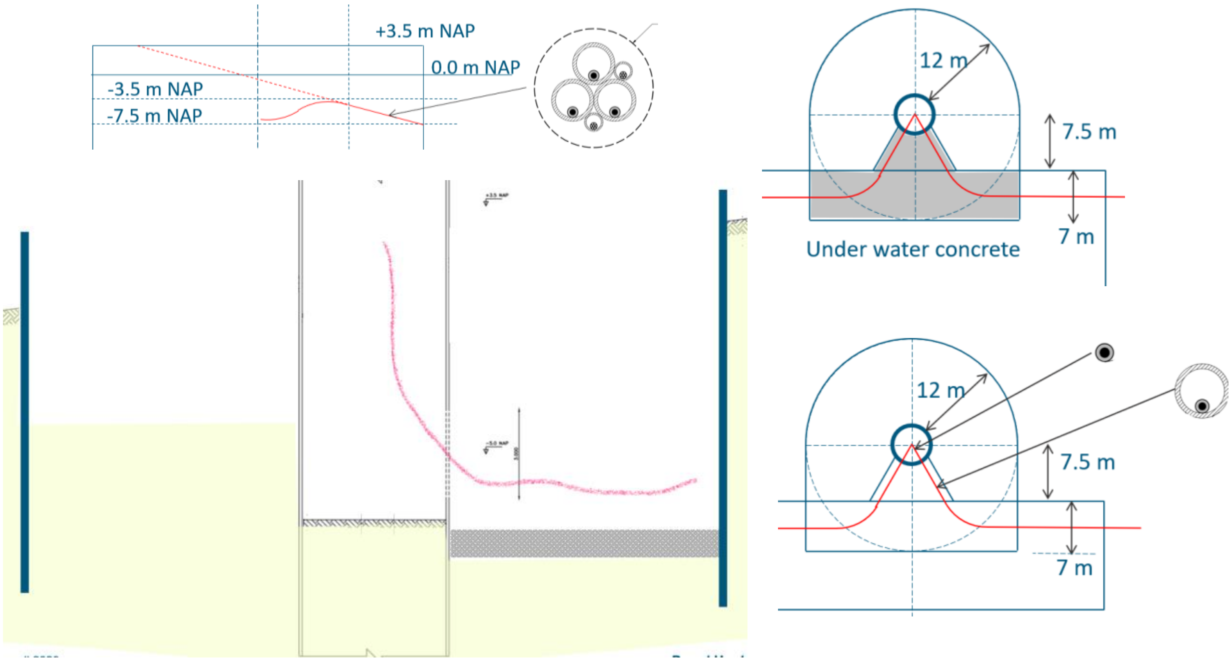


Figure 5-4: Step II Cable installation

- Step III: filling of excavated ground to the general estimated morphologically dynamic zone. It aims to secure the monopile against deep local scour and protect the cable entry zone against the general erosion and scour. The cables at entry level will be buried and protected against general erosion and local scour. For this layer gravel and small rock material will be used. The upper and lower levels will vary in accordance to the cable entry zone.

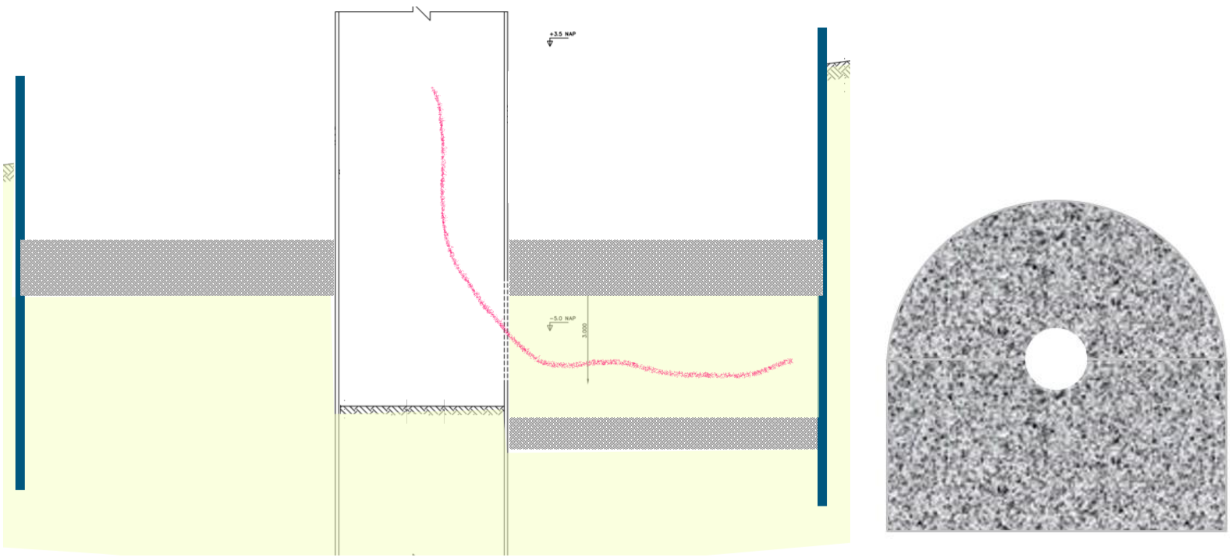


Figure 5-5: Step III Cable installation – Scour protection

- Step IV: filling of excavated ground to the general initial beach level. The inside area of the pile will be equally filled to the initial level.

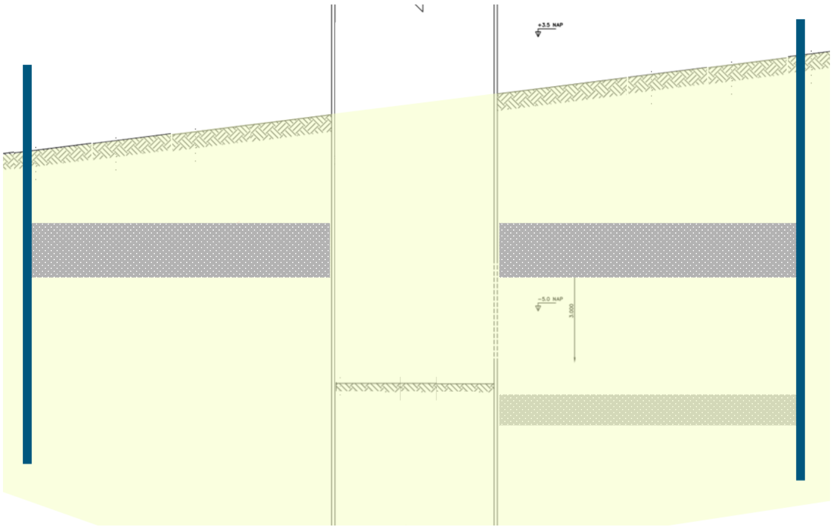


Figure 5-6: Step IV Filling

- Step V: Removal of temporary sheet piles. The final situation has a buried scour protection. The idea is that the beach will have a morphologic dynamic zone with a typical thickness the differs per pile. General erosion can occur in the sandy top layer. When deep local scour holes would form, the scour protection will be encountered, and the formation of a local scour hole will be stopped.

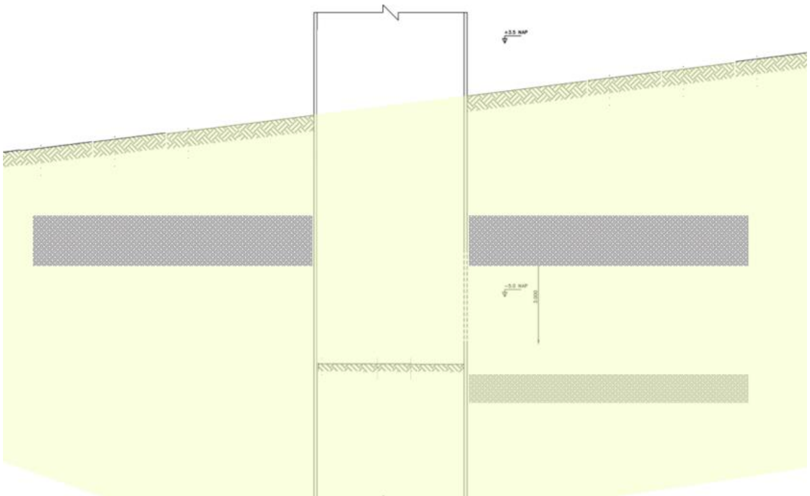


Figure 5-7: Step V – in place new situation

The procedure as described will result in a reduction of local scour but is associated with local disturbance of the original density and compaction of the soil above and including the scour protection layer.

Table 5-1: Electrical cables entry levels

Pile	morphologic dynamic zone z_1 [m]	category [m]	additional margin [m]	scour layer thickness [m]	underside of scour layer [m NAP]	top of under water concrete [m NAP]	thickness of under water concrete [m]	bottom of under water concrete [m NAP]
ZZ-01	-2.4	-3.0	1.0	1.5	-5.5	-8.4	2.00	-10.4
ZZ-02	-1.1	-2.0	1.0	1.5	-4.5	-7.4	1.75	-9.1
ZZ-03	-0.9	-2.0	1.0	1.5	-4.5	-7.4	1.75	-9.1
ZZ-04	-1.3	-2.0	1.0	1.5	-4.5	-7.4	1.75	-9.1
ZZ-05	-2.0	-3.0	1.0	1.5	-5.5	-8.4	2.00	-10.4
ZZ-06	-2.8	-3.5	1.0	1.5	-6.0	-8.9	2.25	-11.1
ZZ-07	-2.0	-3.0	1.0	1.5	-5.5	-8.4	2.00	-10.4
ZZ-08	-1.7	-2.5	1.0	1.5	-5.0	-7.9	2.00	-9.9
ZZ-09	-1.3	-2.0	1.0	1.5	-4.5	-7.4	1.75	-9.1
ZZ-10	-2.1	-2.5	1.0	1.5	-5.0	-7.9	1.75	-9.6
ZZ-11	-1.6	-2.0	1.0	1.5	-4.5	-7.4	1.75	-9.1
ZZ-12	-1.5	-2.0	1.0	1.5	-4.5	-7.4	1.75	-9.1

6 Conclusions and recommendations

6.1 Summary of verification results

In line with the requirements following from the risk management plan ([1], [2]) a monopile foundation concept has been developed for the wind turbines that will be installed along the SSB of MV2. During the concept design loop the performance of the monopile foundation is assessed with a lateral beam-spring model (using p-y curves) and verified on three main items: deformation behaviour, structural capacity and geotechnical stability. The verification results of the concept design loop are summarised in Table 6-1.

Table 6-1: Summary of monopile concept design verification results

Behaviour	Observation	Verification
SLS deformations	Static rotational stiffness is sufficient with D5000/90 monopile foundation.	Rotational stiffness in SLS: • $k_{\varphi,Enercom}$ > 26.666 GNm/rad • $k_{\varphi,GE}$ > 30.000 GNm/rad
	The lateral deflection at mudline level is sufficiently low for the D5000/90 monopile	Normal operational load conditions in SLS: • Lateral deflection at mudline: 3% of pile diameter • Maximum inclination: 1mm/m
	The inclination requirement needs further assessment in the preliminary design phase in relation to lacking load combination data.	Extreme operational ("abnormal") load conditions in SLS: • Lateral deflection at mudline: 6% of pile diameter • Maximum inclination: 3mm/m
ULS structural capacity	Sufficient structural capacity against internal force demand is obtained by applying D5000/90 (S355) with a locally increased wall thickness of 100mm.	Unity check based on CC2 / RC2 Eurocode safety level. Working stress < 40% of yield stress to create sufficient margin for fatigue (which is not assessed in concept design yet) and to accommodate the equivalent soil resistance factor of 1.5.
ULS geotechnical stability (push-over)	The studied soil-pile system with the D5000/90 monopile is sufficiently stable under the ULS turbine load demands. The abnormal scenario with full erosion to NAP -7m and successive lacking maintenance should be prevented. In that specific scenario residual capacity can be insufficient to resist an additional incoming wave load of 2000kN. Minimum pile toe levels are recommended in relation to erosion scenarios: • Mudline at NAP -0.6m → minimum pile toe level at NAP -40m • Mudline at NAP -4.0m → minimum pile toe level at NAP -45m • Mudline at NAP -7.0m → minimum pile toe level at NAP -50m	Margin assessment: load demand and erosion scenarios in relation to push-over curves and minimum pile toe level

6.2 Recommendations for preliminary design phase

During the concept design phase, several recommendations have been identified for the preliminary design phase in relation to further development and optimisation of the monopile foundation design:

- The applicability of the p-y curve methodology for the 5.0m diameter monopiles needs to be validated with other calculation models, including finite element models.
- For the preliminary design phase, a more detailed definition of the turbine load combinations is required for comprehensive foundation verifications and fatigue evaluation. In addition, the turbine load combinations need to be further developed for the exact location and supporting condition.
- Wave loads will be determined in detail in a separate specialist desk study. The loads on the monopiles will also consider the forming of marine growth. The wave study will define oscillating loads, and slamming loads caused by breaking of waves due to limited depth of high wave steepness.
- Once final soil investigation is available the preliminary soil interpretation needs to be updated and the geotechnical safety approach implemented in the design further developed according to Eurocode DA3, in accordance with the DNV-GL standards.
- The design soil profiles that will be selected from the updated soil interpretation can be aligned with the coastal accretion and erosion zones observed along the SSB.
- Realistic scour and erosion scenarios need to be selected for preliminary design phase.
- Although not governing over lateral bearing capacity, axial bearing capacity needs to be verified once final soil investigation is available.
- The SLS inclination requirement needs further assessment in the preliminary design phase in relation to lacking load combination data.
- The structural cross-section utilisation in relation to fatigue needs to be verified. Currently the structural unity check in ULS is reduced to allow for sufficient fatigue capacity.
- Constructability constraints in relation to the required monopile dimensions need to be studied, e.g. in relation to risk mitigation measures M2.1.8.1 and M2.2.6.2 listed in [2].

It is noted that during concept design phase a monopile diameter of 4.3m was also assessed. The results of this assessment are not reported in detail as the D4300/80 monopile capacity appeared insufficient to meet several verification requirements for some turbine types under the assumptions introduced in this concept design step. The 4.3m monopile might proof to be feasible though, once analysed with more detailed input data during the preliminary design phase.

Considering the required size of the monopiles and the MV2 soil conditions it needs to be taken in account that installation of the monopiles with a vibro-hammer might not be feasible. In further consideration of the risk mitigation measures M3 described in [1] and M2.3.3.1 / M2.4.3.1 described in [2], anticipated installation of the monopiles through impact driving and/or using the so-called GPD approach is feasible in terms of driving efficiency, achieved pile bearing capacity, noise levels and collateral damage risk.

7 References

[1] Eneco, Risicobeheersplan Windpark Maasvlakte 2, 2019
[2] Eneco, Verificatiematrix Windpark Maasvlakte 2, 2019
[3] ANSI/API Recommended Practice 2GEO / ISO 19901-4 – Geotechnical and Foundation Design Considerations, 2014
[4] NEN, NEN9997-1+C2 / EN1997-1 Geotechnical design of structures – Part 1: General rules, 2017
[5] DNVGL-ST-0126 – Support Structures for wind turbines, Edition April 2016
[6] NEN, NEN-EN 1993-1-1+C2+A1 Design of steel structures - Part 1-1: General rules and rules for buildings, 2016
[7] NEN, NEN-EN 1993-5 Design of steel structures - Part 5: Piling, 2008
[8] Zur Bemessung geschraubter Ringflanschverbindung von Windenergieanlagen, Marc Seidel, Hannover, 2001
[9] Roark's Formulas for Stress and Strain- Seventh Edition

A1 Appendix 1 – EC3 structural capacity verifications

PROJECT TITLE: ENECO MV2 WINDFARM - MONOPILE FOUNDATION CONCEPT DESIGN

SUBJECT: SOUTH PROFILE - NO EROSION / SCOUR - NO CYCLIC DEGRADATION

PROJECT NO: BG8375 FILE REF: REV:

PREPARED BY: CHA DATE: 1-5-2020 REV DATE:

CHECKED BY: DATE:

(SEE CALC PAGE NO FOR ALTERNATIVE CALCULATIONS)

Royal HaskoningDHV
Enhancing Society Together

REF. | OUTPUT.

A INPUT

A.1 Uncorroded Pile Dimensions

Outer Diameter (d)		=	5000	mm	Class 3
Wall Thickness (t)		=	90.0	mm	

A.2 Corroded Pile Dimensions

Total corrosion loss to outer surface in zone considered	=	0	mm
Total corrosion loss to interior surface	=	0	mm
Outer Diameter (d)	=	5000	mm
Wall Thickness (t)	=	90	mm
Inside Diameter (d_i)		4820	mm

A.3 Material Properties

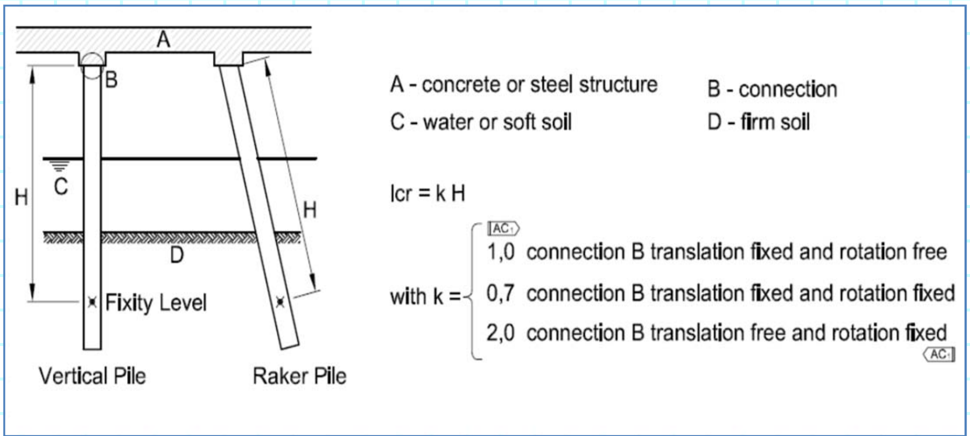
Steel Grade	=	S355	
Design Yield Stress (f_y)	=	315	N/mm^2
Young's Modulus	=	210000	N/mm^2
Cross Section Limits	=	Cold Formed	

A.4 Eurocode Partial Factors

Ref.				
BS EN 1993-5: 2007 Cl 5.1.1(4)	Partial Factor for resistance of cross section	γ_{M0}	=	1.0
	Partial Factor for resistance of members to instability assessed by member checks	γ_{M1}	=	1.0

A.5 Pile Length

BS EN 1993-5: 2007 Figure 5.8



Length of Pile From Soffit to Fixity Level (H)	=	15	m
Length Factor (k)	=	1	

Buckling Length = l_{cr} = k x H

= 1 x 15

l_{cr} = 15.00 m

REF. OUTPUT.

A.6 Maximum Design Forces And Bending Moment

A.6.1 Individual Maximum Forces and Bending Moments

Design Maximum Axial compressive Force (N_{Ed})		=	13000	kN
Design Maximum Tension Force (N_{Ed})		=	0	kN
Design Maximum Shear (V_{Ed})		=	15000	kN
Design Maximum Torsion (T_{Ed})		=	19500	kNm
Design Max Bending Moment ($M_{y,Ed}$)		=	0	kNm
Design Max Bending Moment ($M_{z,Ed}$)		=	275000	kNm
Design Resultant Bending moment (M_{Ed})		=	275000	kNm

A.6.2 Co-Existing Axial Forces and Bending Moment

Design Axial Force (N_{Ed})		=	13000	kN
Design Shear (V_{Ed})		=	0	kN
Design Torsion (T_{Ed})		=	19500	kNm
Design Bending Moment at top in y-y axis ($M_{y,Ed}$)		=	0	kNm
Design Bending Moment at top in z-z axis ($M_{z,Ed}$)		=	275000	kNm
Design Resultant Bending Moment at top (M_{Ed})	Refer section B.11.3 and table B.3 of this Sheet	=	275000	kNm
Design Bending Moment at bottom in y-y axis ($M_{y,Ed}$)		=	0	kNm
Design Bending Moment at bottom in z-z axis ($M_{z,Ed}$)		=	0	kNm
Design Resultant Bending Moment at bottom (M_{Ed})		=	0	kNm

A.6.3 Second Order Moment

(Max p-delta moment that co-exists with forces/moments entered in A.6.1 or A.6.2 above)

pΔ moment co-existing with forces entered in A.6.1	=	0	kNm
pΔ moment co-existing with forces entered in A.6.2	=	0	kNm

A.6.4 Co-Existing Maximum Shear and Torsion

Design Shear (V_{Ed})	=	15000	kN
Design Torsion (T_{Ed})	=	19500	kNm

A.6.5 Local Bending Moment

If there is a local bending moment in the pile due to e.g. wave load insert it here

Design Local Bending Moment (M_s)	=	0	kNm
Type of loading creating local moment	=	Concentrated	

A.6.6 Equivalent Uniform Moment Factor

Equivalent uniform moment factor	C_m	=	1.00

REF. | OUTPUT.

B STRUCTURAL CALCULATIONS

The following calculations will be based on the pile in Corroded condition.

B.1 Parameters for Structural Calculations

Outer Diameter (d)	=	5000	mm
Wall Thickness (t)	=	90	mm
Inner Diameter (d_i)	=	4820	mm
Diameter/wall thickness (d/t)	=	55.56	
Pile Area	=	1388270	mm^2
Moment of Inertia (I)	=	4.18E+12	mm^4
Radius of Gyration (i)	=	1736	mm
Elastic Section Modulus (W_{el})	=	1.67E+09	mm^3
Plastic Section Modulus (W_{pl})	=	2.17E+09	mm^3
Shear Area (A_v)	=	883800	mm^2

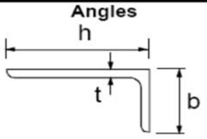
B.2 Classification of Cross Section

BS EN 1993-1-1: 2005 Table 5.2

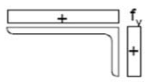
Table 5.2 (sheet 3 of 3): Maximum width-to-thickness ratios for compression parts

Angles

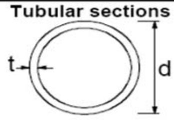
Refer also to "Outstand flanges" (see sheet 2 of 3)



Does not apply to angles in continuous contact with other components

Class	Section in compression
Stress distribution across section (compression positive)	
3	$h/t \le 15\varepsilon : \frac{b+h}{2t} \le 11{,}5\varepsilon$

Tubular sections



Class	Section in bending and/or compression
1	$d/t \le 50\varepsilon^2$
2	$d/t \le 70\varepsilon^2$
3	$d/t \le 90\varepsilon^2$ NOTE For $d/t > 90\varepsilon^2$ see EN 1993-1-6.

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71
	ε^2	1,00	0,85	0,66	0,56	0,51

$$\varepsilon = \sqrt{235/f_y} = 0.86$$

$$50\,\varepsilon^2 = 37.30$$

$$70\,\varepsilon^2 = 52.22$$

$$90\,\varepsilon^2 = 67.14$$

$$d/t = 55.56 \le 90\ \varepsilon^2$$

Type of Class = Class 3

Class 3

REF. | OUTPUT.

B.3 Maximum Axial Compression

N_{Ed} = 13000 kN

BS EN 1993-1-1: 2005 Cl 6.2.4(1) P

The design Value of the compression force N_{Ed} at each cross section shall satisfy:

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0$$

N_{Ed} = Design Normal Force

$N_{c,Rd}$ = Design Resistance of the cross section for uniform compression

N_{Rd} inc. yield stress reduction due to shear effects Refer 6.2.8

N_{Rd} (kN) **437305**

BS EN 1993-1-1: 2005 Eq 6.10

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \mathbf{437305}\ \text{kN}$$ For class 1,2 and 3 cross sections

$$\frac{N_{Ed}}{N_{c,Rd}} = \mathbf{0.03} \leq 1.0$$ OK

N_{Ed}/N_{Rd} **0.03** OK

B.4 Maximum Axial Tension

N_{Ed} = 0 kN

BS EN 1993-1-1: 2005 Cl 6.2.3(1) P

The design Value of the tension force N_{Ed} at each cross section shall satisfy:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0$$

N_{Ed} = Design Normal Force

$N_{t,Rd}$ = Design Resistance of the cross section for tension

N_{Rd} inc. yield stress reduction due to shear effects Refer 6.2.8

N_{Rd} (kN) **437305**

BS EN 1993-1-1: 2005 Eq 6.6

$$N_{t,Rd} = \frac{Af_y}{\gamma_{M0}} = \mathbf{437305}\ \text{kN}$$ (For class 1,2 or 3 cross sections)

$$\frac{N_{Ed}}{N_{t,Rd}} = \mathbf{0.00} \leq 1.0$$ OK

N_{Ed}/N_{Rd} **0.00** OK

B.5 Maximum Bending Moment

M_{Ed} = 275000 kNm

BS EN 1993-1-1: 2005 Cl 6.2.5(1) P

The design value of bending moment M_{Ed} at each cross-section shall satisfy:

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0$$

$M_{c,Rd}$ = Design resistance of the cross-section for bending

BS EN 1993-1-1: 2005 Eq 6.13 & Eq 6.14

$$M_{c,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}}$$ for class 1 or 2 cross sections

$$M_{c,Rd} = \frac{W_{el,min} f_y}{\gamma_{M0}}$$ for class 3 cross sections

Section Modulus to be considered: *Elastic Section Modulus*

$M_{c,Rd}$ = 527307 kNm

$$\frac{M_{Ed}}{M_{c,Rd}} = \mathbf{0.52} \leq 1.0$$ OK

OK

REF. | OUTPUT.

B.6 Maximum Shear

V_{Ed} = 15000 kN

BS EN 1993-1-1: 2005 Cl 6.2.6(1) P

The design value of shear V_{Ed} at each cross-section shall satisfy:

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1.0$$

The design plastic shear resistance of a section may be used in elastic design under EC3

BS EN 1993-1-1: 2005 Cl 6.2.6(4) P

Elastic shear verification is conservative and excludes partial plastic shear distribution which is permitted in elastic design. Therefore elastic shear verification should only be carried out where the verification on the basis of $V_{c,Rd}$ according to equation (6.17) can not be performed.

$$V_{c,Rd} = V_{pl.Rd} = \frac{A_V(f_y/\sqrt{3})}{\gamma_{M0}}$$ (Plastic Shear Resistance)

Refer section B.1 of this Sheet

A_v = Shear Area = $2A/\pi$

= 883800 mm^2

$V_{pl.Rd}$ = 160733 kN

$$\frac{V_{Ed}}{V_{c,Rd}} = \mathbf{0.09} \leq 1.0$$

OK

B.7 Maximum Torsion

BS EN 1993-1-1: 2005 Cl 6.2.7(1)

The design value of the torsion T_{Ed} at each cross-section shall satisfy:

$$\frac{T_{Ed}}{T_{Rd}} \leq 1$$

T_{Rd} = Design torsional resistance of the cross section

T_{Ed} = 19500 kNm

The resistance T_{Rd} of a circular hollow section shall be based on the design shear strength.

Therefore check shear induced by torsion is less than shear strength calculated in B.6 above.

$\tau_{t,Ed}$ = Design shear stress due to St. Venant torsion

$$\tau_{t,Ed} = \frac{T_{t,Ed}}{C}$$ Note: For circular tube effect of torsional warping can be neglected ($T_{w,Ed} = 0$). Therefore $T_{Ed} = T_{t,Ed}$

Where C = Torsional Modulus Constant

C = 2 x W_{el}

C = 2 x 1.67E+09

C = 3.35E+09

$\tau_{t,Ed}$ = 5824 kN/m^2

V_{Ed} due to torsion = $\tau_{t,Ed}$ x Shear Area A_v

= 5824 x 0.8838

= 5148 kN

$$\frac{V_{Ed}}{V_{pl,Rd}} \leq 1.0$$

$$\frac{V_{Ed}}{V_{pl,Rd}} = \mathbf{0.03} \leq 1.0$$

OK

REF. | OUTPUT.

BS EN 1993-1-1: 2005 Cl 6.2.7(9)

B.8 Torsion and Shear

For combined shear force and torsional moment the plastic shear resistance accounting for torsional effects should be reduced from $V_{pl,Rd}$ to $V_{pl,T,Rd}$ and the design shear force should satisfy :

BS EN 1993-1-1: 2005 eq 6.25 & 6.28

$$\frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1.0$$

$$V_{pl,T,Rd} = \left[1 - \frac{\tau_{t,Ed}}{(f_y/\sqrt{3})/\gamma_{M0}}\right] V_{pl,Rd}$$

T_{Ed} = 19500 kNm

$\tau_{t,Ed}$ = $T_{t,Ed}/C$

$\tau_{t,Ed}$ = 5824 kN/m²

$T_{t,Ed}$ = 19500 kNm

A_v = 883800 mm²

$V_{pl,Rd}$ = 160733 kN

$V_{pl,T,Rd}$ = 155585 kN

V_{Ed} = 15000 kN

$$\frac{V_{Ed}}{V_{pl,T,Rd}} = \mathbf{0.10} \leq 1.0$$

OK

BS EN 1993-1-1: 2005 Cl 6.2.8

B.9 Bending and Shear

V_{Ed} = 0 kN

$T_{t,Ed}$ = 19500 kNm

$\tau_{t,Ed}$ = 5824 kN/m²

M_{Ed} = 275000 kNm

$V_{pl,Rd}$ = 160733 kN

$V_{pl,T,Rd}$ = 155585 kN

BS EN 1993-1-1: 2005 Cl 6.2.8(2)

Assuming some torsion is present, $V_{pl,T,Rd}$ will be more onerous than $V_{pl,Rd}$. Hence check effect of $V_{pl,T,Rd}$ on moment resistance first:

If V_{Ed} is less than half $V_{pl,T,Rd}$ then no reduction in moment resistance needs to be considered.

$2V_{Ed} \leq V_{pl,T,Rd}$ *No effect on the Moment Resistance. Ignore boxed section below*

No reduction in moment resistance req'd

BS EN 1993-1-1: 2005 Cl 6.2.8(3)(4)

Reduced Yield Strength = $(1-\rho)\, f_y$

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,T,Rd}} - 1\right)^2$$ (Where torsion is present. See note below)

Note: If there is no torsion present then $V_{pl,Rd} = V_{pl,T,Rd}$ and the above conclusion regarding effect on moment resistance remains the same.

Design Yield Strength for bending in shear = 315 N/mm²

Design Resistance of the cross section for bending in shear $M_{c,Rd}$ = 527307 kNm

$$\frac{M_{Ed}}{M_{c,Rd}} = \mathbf{0.52} \leq 1.0$$

OK

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REF. | OUTPUT.

BS EN 1993-1-1: 2005 Cl 6.2.10

B.10 Bending, Shear and Axial Force

B.10.1 Effect of Shear Force

Type of Class = Class 3

V_{Ed} = 0 kN

$V_{pl,Rd}$ = 160733 kN

Where the shear force is less than half the plastic shear resistance its effect on the bending and axial force resistance may be neglected.

$2\,V_{Ed} \leq V_{pl,Rd}$ *Hence no reduction in yield strength due to shear only*

but consider fy reduction, if applicable, due to torsional shear

Reduced Yield Strength = $(1-\rho)\,f_y$

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2$$

Design Yield Strength for bending in shear = 315 N/mm^2

B.10.2 Effect of Axial Force

Section is *Class 3* therefore go to section *B.10.2.2* of the following calculations.

BS EN 1993-1-1: 2005 Cl 6.2.9.1(2)

B.10.2.1 Class 1 and 2 Cross Sections

For Class 1 and 2 cross sections following criterion shall be satisfied: $M_{Ed} \leq M_{N,Rd}$

$M_{N,Rd}$ = Design plastic moment resistance reduced due to axial force N_{Ed}

M_{Ed} = 275000 kNm

BS EN 1993-1-1: 2005 Cl 6.2.9.1(6)

$M_{N,y,Rd} = M_{N,z,Rd} = M_{pl,Rd} \times (1-n^{1.7})$

$n = N_{Ed} / N_{pl,Rd}$

N_{Ed} = 13000 kN

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} = 437305 \text{ kN}$$

n = 0.030

$M_{pl,Rd}$ = 683541 kNm

$M_{N,Rd} = M_{N,z,Rd} = M_{N,y,Rd}$ = 681807 kNm

$M_{Ed} \leq M_{N,Rd}$ OK

Ratio $M_{Ed} / M_{N,Rd}$ = **0.40** | N/A

Overall interaction

BS EN 1993-1-1: 2005 Cl 6.2.1(7)

$N_{Ed} / N_{Rd} + M_{Ed} / M_{Rd}$ = **0.43** | N/A

(The check below is stated in the code, but can be considered secondary to that done above)

BS EN 1993-1-1: 2005 Cl 6.2.9.1(6)

For biaxial bending in class 1 and 2 cross sections following criterion shall be satisfied:

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leq 1.0$$

$M_{y,Ed}$ = 0 kNm

$M_{z,Ed}$ = 275000 kNm

α = 2

β = 2

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} = \mathbf{0.16} \leq 1.0$$ OK | N/A

REF. | OUTPUT.

BS EN 1993-1-1: 2005 Cl 6.2.9.2(1)

B.10.2.2 Class 3 Cross Sections

For class 3 cross sections, **in the absence of shear force**, the maximum longitudinal stress shall satisfy the criterion:-

$$\sigma_{x,Ed} \leq \frac{f_y}{\gamma_{M0}}$$

$\sigma_{x,Ed}$ = Design value of the local longitudinal stress due to moment and axial force

σ_{axial} = N_{Ed}/A

N_{Ed} = 13000 kN

A = 1388270 mm^2

σ_{axial} = 9 N/mm^2

σ_{moment} = M_{Ed}/W_{el}

M_{Ed} = 275000 kNm M_{Rd} = 527306.7 kNm M_{Ed}/M_{Rd} **0.52** M_{Rd} inc. yield stress reduction due to shear effects Refer 6.2.8

W_{el} = 1.67E+09 mm^3

σ_{moment} = 164 N/mm^2

$\sigma_{x,Ed}$ = 9 + 164

= 174 N/mm^2

Yield Strength, f_y = 315 N/mm^2

Overall interaction

BS EN 1993-1-1: 2005 Cl 6.2.1(7)

$\frac{f_y}{\gamma_{M0}}$ = 315 N/mm^2 $N_{Ed} / N_{Rd} + M_{Ed} / M_{Rd}$ = **0.55**   Ok

$\sigma_{x,Ed} \leq \frac{f_y}{\gamma_{M0}}$ OK = **0.55** Stress Ratio   OK

B.11 Buckling Resistance of Members

B.11.1 Uniform Members in Compression

BS EN 1993-1-1: 2005 Cl 6.3.1.1(1)

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0$$

N_{Ed} = 13000 kN

$N_{b,Rd}$ = Design Buckling Resistance of the Compression Member

BS EN 1993-1-1: 2005 Cl 6.3.1.1(3)

$= \chi \frac{Af_y}{\gamma_{M1}}$ For Class 1, 2 and 3 Sections

χ = Reduction Factor for the Relevant Buckling Mode

BS EN 1993-1-1: 2005 Eq 6.49

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \leq 1.0$$

$$\phi = 0.5\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right]$$

$\bar{\lambda}$ = Non-dimensional slenderness ratio

BS EN 1993-1-1: 2005 Eq 6.50

$$= \sqrt{\frac{Af_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\lambda_1}$$ For Class 1, 2 and 3 cross sections

L_{cr} = Buckling Length = 15 m

i = 1736 mm (Radius of Gyration)

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REF. OUTPUT.

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.9\,\varepsilon$$

$$\lambda_1 = 93.9 \times 0.86 = 81.10$$

$$\bar{\lambda} = 0.11$$

BS EN 1993-5:2007 Cl 5.3.3(3)

The Slenderness criterion for which overall buckling may be assumed to be satisfied is:

$$\frac{N_{Ed}}{N_{cr}} \leq 0.1$$

If this criteria is not met consideration should be given to buckling

$$N_{cr} = \frac{Af_y}{\bar{\lambda}^2}$$

$$N_{cr} = 38539785 \text{ kN}$$

$$\frac{N_{Ed}}{N_{cr}} = \mathbf{0.00} \leq 0.1 \quad \text{OK}$$

OK

For $\bar{\lambda} = 0.11$ χ can be established from table 6.1, 6.2 and figure 6.4

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection Factor α	0.13	0.21	0.34	0.49	0.76

Cross section limits = Cold Formed

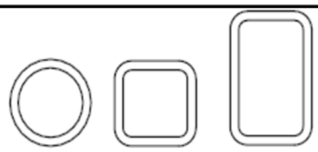
Choose α = 0.49

$$\phi = 0.5\left[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2\right] = 0.483$$

$$\chi = \frac{1}{\phi+\sqrt{\phi^2-\bar{\lambda}^2}} = 1.05 \geq 1.0$$

Use χ = 1

BS EN 1993-1-1:2005 Table 6.2

Cross section		Limits	Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	Buckling curve S 460
Hollow sections		hot finished	any	a	a_0
		cold formed	any	c	c

REF.		OUTPUT.

BS EN 1993-1-1:2005 Figure 6.4

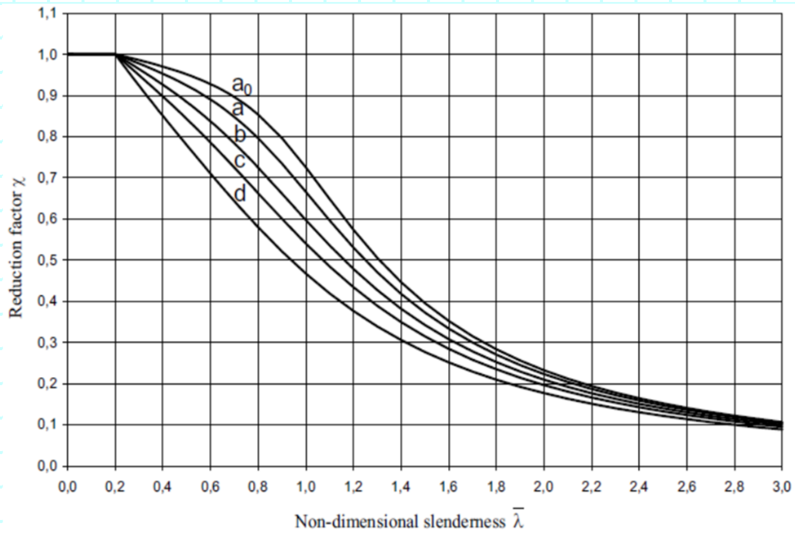


Figure 6.4: Buckling curves

χ = 1

$$N_{b,Rd} = \chi \frac{Af_y}{\gamma_{M1}}$$

= 437305 kN

$$\frac{N_{Ed}}{N_{b,Rd}} = \mathbf{0.03} \leq 1.0$$ OK

OK

B.11.2 Uniform Members in Bending

BS EN 1993-1-1:2005 Cl 6.3.2.1 (2)

Beams with sufficient restraint to the compression flange are not susceptible to lateral-torsional buckling. In addition, beams with certain types of cross sections, such as square or circular hollow sections, fabricated circular tubes or square box sections are not susceptible to lateral-torsional buckling.

Therefore it is not necessary to consider the effects of lateral-torsional buckling for steel tubular piles

B.11.3 Uniform Members in Bending and Axial Compression

BS EN 1993-1-1:2005 Cl 6.3.3 (4) & Eq 6.61/6.62

Members which are subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{(\chi \frac{N_{Rk}}{\gamma_{M1}})} + k \frac{M_{Ed} + \Delta M_{Ed}}{\chi_{LT} (\frac{M_{Rk}}{\gamma_{M1}})} \leq 1.0$$

This equation has been simplified as for a tubular pile only single axis needs to be considered.

REF. OUTPUT.

BS EN 1993-1-1:2005 Table 6.7

Table 6.7: Values for $N_{Rk} = f_y A_i$, $M_{i,Rk} = f_y W_i$ and $\Delta M_{i,Ed}$

Class	1	2	3	4
A_i	A	A	A	A_{eff}
W_y	$W_{pl,y}$	$W_{pl,y}$	$W_{el,y}$	$W_{eff,y}$
W_z	$W_{pl,z}$	$W_{pl,z}$	$W_{el,z}$	$W_{eff,z}$
$\Delta M_{y,Ed}$	0	0	0	$e_{N,y}\,N_{Ed}$
$\Delta M_{z,Ed}$	0	0	0	$e_{N,z}\,N_{Ed}$

NOTE For members not susceptible to torsional deformation χ_{LT} would be $\chi_{LT} = 1{,}0$.

N_{Rk} = $f_y A$ = 315 x 1388270 x 10^{-3}

= 437305 kN

Type of Class = Class 3

Use

W_y = W_z = W = Wel

M_{Rk} = f_y x Wel

= 315 x 1.67E+09

= 5.27E+11 Nmm

= 527307 kNm

BS EN 1993-1-1:2005 Table 6.7

$\Delta M_{y,z,Ed}$ = 0 (For class 1, 2 and 3 sections)

k = Interaction factor

Method 2 shall be used to calculate k factors. In accordance with Steel Construction Institute advice, circular hollow sections can be used in this approach even though reference is only made to I and RHS sections.

BS EN 1993-1-1:2005 Annex B Table B.1

Table B.1: Interaction factors k_{ij} for members not susceptible to torsional deformations

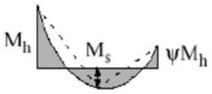
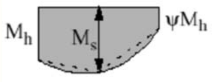
Interaction factors	Type of sections	Design assumptions: elastic cross-sectional properties class 3, class 4	Design assumptions: plastic cross-sectional properties class 1, class 2
k_{yy}	I-sections RHS-sections	$C_{my}\left(1+0{,}6\bar{\lambda}_y \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{my}\left(1+0{,}6 \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$	$C_{my}\left(1+\left(\bar{\lambda}_y-0{,}2\right) \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{my}\left(1+0{,}8 \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$
k_{yz}	I-sections RHS-sections	k_{zz}	$0{,}6\,k_{zz}$
k_{zy}	I-sections RHS-sections	$0{,}8\,k_{yy}$	$0{,}6\,k_{yy}$
k_{zz}	I-sections	$C_{mz}\left(1+0{,}6\bar{\lambda}_z \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+0{,}6 \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$	$C_{mz}\left(1+\left(2\bar{\lambda}_z-0{,}6\right) \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+1{,}4 \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
	RHS-sections		$C_{mz}\left(1+\left(\bar{\lambda}_z-0{,}2\right) \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+0{,}8 \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$

For I- and H-sections and rectangular hollow sections under axial compression and uniaxial bending $M_{y,Ed}$ the coefficient k_{zy} may be $k_{zy} = 0$.

REF. | OUTPUT.

BS EN 1993-1-1:2005 Annex B Table B.3

Table B.3: Equivalent uniform moment factors C_m in Tables B.1 and B.2

Moment diagram	range		C_{my} and C_{mz} and C_{mLT} uniform loading	C_{my} and C_{mz} and C_{mLT} concentrated load
	$-1 \le \psi \le 1$		$0{,}6 + 0{,}4\psi \ge 0{,}4$	
 $\alpha_s = M_s/M_h$	$0 \le \alpha_s \le 1$	$-1 \le \psi \le 1$	$0{,}2 + 0{,}8\alpha_s \ge 0{,}4$	$0{,}2 + 0{,}8\alpha_s \ge 0{,}4$
	$-1 \le \alpha_s < 0$	$0 \le \psi \le 1$	$0{,}1 - 0{,}8\alpha_s \ge 0{,}4$	$-0{,}8\alpha_s \ge 0{,}4$
		$-1 \le \psi < 0$	$0{,}1(1-\psi) - 0{,}8\alpha_s \ge 0{,}4$	$0{,}2(-\psi) - 0{,}8\alpha_s \ge 0{,}4$
 $\alpha_h = M_h/M_s$	$0 \le \alpha_h \le 1$	$-1 \le \psi \le 1$	$0{,}95 + 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$
	$-1 \le \alpha_h < 0$	$0 \le \psi \le 1$	$0{,}95 + 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$
		$-1 \le \psi < 0$	$0{,}95 + 0{,}05\alpha_h(1+2\psi)$	$0{,}90 - 0{,}10\alpha_h(1+2\psi)$
For members with sway buckling mode the equivalent uniform moment factor should be taken $C_{my} = 0{,}9$ or $C_{Mz} = 0{,}9$ respectively.				
C_{my}, C_{mz} and C_{mLT} should be obtained according to the bending moment diagram between the relevant braced points as follows:				
moment factor	bending axis	points braced in direction		
C_{my}	y-y	z-z		
C_{mz}	z-z	y-y		
C_{mLT}	y-y	y-y		

BS EN 1993-1-1:2005 Annex B Table B.3

M_{max} = 275000 kNm

M_{min} = 0 kNm

ψ = 0.00

M_s = 0.0 kNm (local moment)

Equivalent uniform moment factor

C_m = 1.00

Table B.1 gives the following formulas to calculate Interaction factor k for different cross section.

(Tubular members are not susceptible to torsional deformations)

For Class 1 and class 2 cross section:

BS EN 1993-1-1:2005 Annex B Table B.1

$$k = C_m\left[1+(\bar{\lambda}-0.2)\frac{N_{Ed}}{\chi N_{Rk}/\gamma_{M1}}\right] \le C_m\left[1+0.8\frac{N_{Ed}}{\chi N_{Rk}/\gamma_{M1}}\right]$$

These equations have been simplified as for tubular pile only single axis needs to be considered

And

For Class 3 cross section:

$$k = C_m\left[1+0.6\bar{\lambda}\frac{N_{Ed}}{\chi N_{Rk}/\gamma_{M1}}\right] \le C_m\left[1+0.6\frac{N_{Ed}}{\chi N_{Rk}/\gamma_{M1}}\right]$$

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REV:	
PREPARED BY:	CHA
DATE:	1-5-2020
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DATE:	

(SEE CALC PAGE NO FOR ALTERNATIVE CALCULATIONS)

REF.

Refer section B.10.1 of this Sheet

OUTPUT.

Type of class = Class 3

$\bar{\lambda}$ = 0.11

N_{Ed} = 13000 kN

N_{Rk} = 437305 kN

χ = 1

χ_{LT} = 1.0 *(Circular hollow sections not susceptible to lateral torsional buckling)*

γ_{M1} = 1

M_{Ed} = 275000 kNm

ΔM_{Ed} = 0 kNm

M_{Rk} = 527307 kNm

k = 1.002

$$\frac{N_{Ed}}{(\chi \frac{N_{Rk}}{\gamma_{M1}})} + k \frac{M_{Ed} + \Delta M_{Ed}}{\chi_{LT} (\frac{M_{Rk}}{\gamma_{M1}})} = \mathbf{0.55} \leq 1.0$$

OK

OK

PROJECT TITLE: ENECO MV2 WINDFARM - MONOPILE FOUNDATION CONCEPT DESIGN

SUBJECT: SOUTH PROFILE - FULL EROSION (NAP -7m) - CYCLIC DEGRADATION

PROJECT NO: BG8375 FILE REF: REV:

PREPARED BY: CHA DATE: 1-5-2020 REV DATE:

CHECKED BY: DATE:

(SEE CALC PAGE NO FOR ALTERNATIVE CALCULATIONS)

REF. | OUTPUT.

A INPUT

A.1 Uncorroded Pile Dimensions

Outer Diameter (d)	=	5000	mm		Class 3
Wall Thickness (t)	=	90.0	mm		

A.2 Corroded Pile Dimensions

Total corrosion loss to outer surface in zone considered	=	0	mm
Total corrosion loss to interior surface	=	0	mm
Outer Diameter (d)	=	5000	mm
Wall Thickness (t)	=	90	mm
Inside Diameter (d_i)		4820	mm

A.3 Material Properties

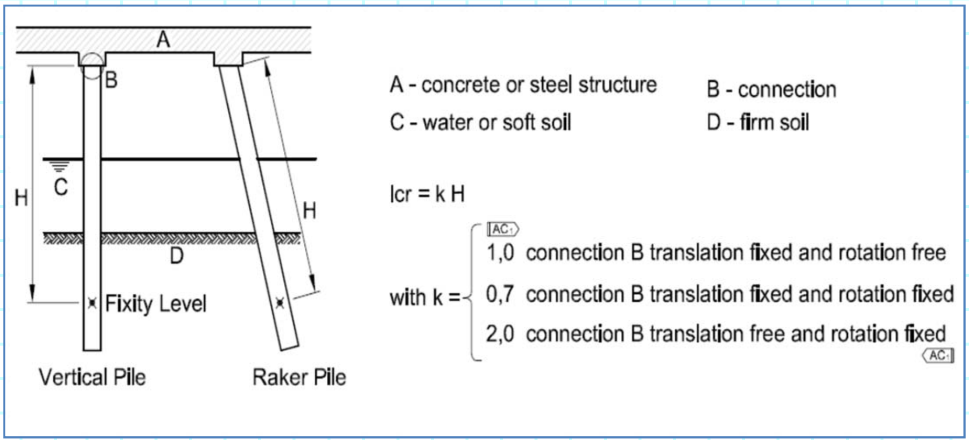
Steel Grade	=	S355	
Design Yield Stress (f_y)	=	315	N/mm^2
Young's Modulus	=	210000	N/mm^2
Cross Section Limits	=	Cold Formed	

A.4 Eurocode Partial Factors

Ref.				
BS EN 1993-5: 2007 Cl 5.1.1(4)	Partial Factor for resistance of cross section	γ_{M0}	=	1.0
	Partial Factor for resistance of members to instability assessed by member checks	γ_{M1}	=	1.0

A.5 Pile Length

BS EN 1993-5: 2007 Figure 5.8



Length of Pile From Soffit to Fixity Level (H)	=	15	m
Length Factor (k)	=	1	

Buckling Length = l_{cr} = k x H

= 1 x 15

l_{cr} = 15.00 m

REF. OUTPUT.

A.6 Maximum Design Forces And Bending Moment

A.6.1 Individual Maximum Forces and Bending Moments

Design Maximum Axial compressive Force (N_{Ed})	=	13000	kN
Design Maximum Tension Force (N_{Ed})	=	19500	kN
Design Maximum Shear (V_{Ed})	=	26000	kN
Design Maximum Torsion (T_{Ed})	=	0	kNm
Design Max Bending Moment ($M_{y,Ed}$)	=	0	kNm
Design Max Bending Moment ($M_{z,Ed}$)	=	310000	kNm
Design Resultant Bending moment (M_{Ed})	=	310000	kNm

A.6.2 Co-Existing Axial Forces and Bending Moment

Refer section B.11.3 and table B.3 of this Sheet

Design Axial Force (N_{Ed})	=	13000	kN
Design Shear (V_{Ed})	=	0	kN
Design Torsion (T_{Ed})	=	19500	kNm
Design Bending Moment at top in y-y axis ($M_{y,Ed}$)	=	0	kNm
Design Bending Moment at top in z-z axis ($M_{z,Ed}$)	=	310000	kNm
Design Resultant Bending Moment at top (M_{Ed})	=	310000	kNm
Design Bending Moment at bottom in y-y axis ($M_{y,Ed}$)	=	0	kNm
Design Bending Moment at bottom in z-z axis ($M_{z,Ed}$)	=	0	kNm
Design Resultant Bending Moment at bottom (M_{Ed})	=	0	kNm

A.6.3 Second Order Moment

(Max p-delta moment that co-exists with forces/moments entered in A.6.1 or A.6.2 above)

pΔ moment co-existing with forces entered in A.6.1	=	0	kNm
pΔ moment co-existing with forces entered in A.6.2	=	0	kNm

A.6.4 Co-Existing Maximum Shear and Torsion

Design Shear (V_{Ed})	=	26000	kN
Design Torsion (T_{Ed})	=	19500	kNm

A.6.5 Local Bending Moment

If there is a local bending moment in the pile due to e.g. wave load insert it here

Design Local Bending Moment (M_s)	=	0	kNm
Type of loading creating local moment	=	Concentrated	

A.6.6 Equivalent Uniform Moment Factor

Equivalent uniform moment factor	C_m =	1.00	

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B STRUCTURAL CALCULATIONS

The following calculations will be based on the pile in Corroded condition.

B.1 Parameters for Structural Calculations

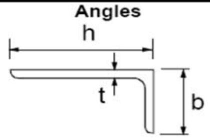
Outer Diameter (d)	=	5000	mm
Wall Thickness (t)	=	90	mm
Inner Diameter (d_i)	=	4820	mm
Diameter/wall thickness (d/t)	=	55.56	
Pile Area	=	1388270	mm^2
Moment of Inertia (I)	=	4.18E+12	mm^4
Radius of Gyration (i)	=	1736	mm
Elastic Section Modulus (W_{el})	=	1.67E+09	mm^3
Plastic Section Modulus (W_{pl})	=	2.17E+09	mm^3
Shear Area (A_v)	=	883800	mm^2

B.2 Classification of Cross Section

BS EN 1993-1-1: 2005 Table 5.2

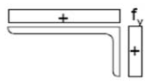
Table 5.2 (sheet 3 of 3): Maximum width-to-thickness ratios for compression parts

Angles

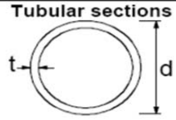


Refer also to "Outstand flanges" (see sheet 2 of 3)

Does not apply to angles in continuous contact with other components

Class	Section in compression
Stress distribution across section (compression positive)	
3	$h/t \le 15\varepsilon : \frac{b+h}{2t} \le 11{,}5\varepsilon$

Tubular sections



Class	Section in bending and/or compression
1	$d/t \le 50\varepsilon^2$
2	$d/t \le 70\varepsilon^2$
3	$d/t \le 90\varepsilon^2$ NOTE For $d/t > 90\varepsilon^2$ see EN 1993-1-6.

$\varepsilon = \sqrt{235/f_y}$						
	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71
	ε^2	1,00	0,85	0,66	0,56	0,51

$$\varepsilon = \sqrt{235/f_y} = 0.86$$

$$50\,\varepsilon^2 = 37.30$$

$$70\,\varepsilon^2 = 52.22$$

$$90\,\varepsilon^2 = 67.14$$

$$d/t = 55.56 \le 90\,\varepsilon^2$$

Type of Class = Class 3

Class 3

REF.		OUTPUT.
	B.3 Maximum Axial Compression	
	N_{Ed} = 13000 kN	
BS EN 1993-1-1: 2005 Cl 6.2.4(1) P	The design Value of the compression force N_{Ed} at each cross section shall satisfy:	
	$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0$	
	N_{Ed} = Design Normal Force	N_{Rd} inc. yield
	$N_{c,Rd}$ = Design Resistance of the cross section for uniform compression	stress reduction
	N_{Rd} (kN)	due to shear effects
BS EN 1993-1-1: 2005 Eq 6.10	$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}}$ = **437305** kN For class 1,2 and 3 cross sections **437305**	Refer 6.2.8
	$\frac{N_{Ed}}{N_{c,Rd}}$ = **0.03** ≤ 1.0 OK N_{Ed}/N_{Rd} **0.03**	OK
	B.4 Maximum Axial Tension	
	N_{Ed} = 19500 kN	
	The design Value of the tension force N_{Ed} at each cross section shall satisfy:	
BS EN 1993-1-1: 2005 Cl 6.2.3(1) P	$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0$	
	N_{Ed} = Design Normal Force	N_{Rd} inc. yield
	$N_{t,Rd}$ = Design Resistance of the cross section for tension	stress reduction
	N_{Rd} (kN)	due to shear effects
BS EN 1993-1-1: 2005 Eq 6.6	$N_{t,Rd} = \frac{Af_y}{\gamma_{M0}}$ = **437305** kN (For class 1,2 or 3 cross sections) **437305**	Refer 6.2.8
	$\frac{N_{Ed}}{N_{t,Rd}}$ = **0.04** ≤ 1.0 OK N_{Ed}/N_{Rd} **0.04**	OK
	B.5 Maximum Bending Moment	
	M_{Ed} = 310000 kNm	
BS EN 1993-1-1: 2005 Cl 6.2.5(1) P	The design value of bending moment M_{Ed} at each cross-section shall satisfy:	
	$\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0$	
	$M_{c,Rd}$ = Design resistance of the cross-section for bending	
BS EN 1993-1-1: 2005 Eq 6.13 & Eq 6.14	$M_{c,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}}$ for class 1 or 2 cross sections	
	$M_{c,Rd} = \frac{W_{el,min} f_y}{\gamma_{M0}}$ for class 3 cross sections	
	Section Modulus to be considered: *Elastic Section Modulus*	
	$M_{c,Rd}$ = 527307 kNm	
	$\frac{M_{Ed}}{M_{c,Rd}}$ = **0.59** ≤ 1.0 OK	OK

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B.6 Maximum Shear

V_{Ed} = 26000 kN

BS EN 1993-1-1: 2005 Cl 6.2.6(1) P

The design value of shear V_{Ed} at each cross-section shall satisfy:

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1.0$$

The design plastic shear resistance of a section may be used in elastic design under EC3

BS EN 1993-1-1: 2005 Cl 6.2.6(4) P

Elastic shear verification is conservative and excludes partial plastic shear distribution which is permitted in elastic design. Therefore elastic shear verification should only be carried out where the verification on the basis of $V_{c,Rd}$ according to equation (6.17) can not be performed.

$$V_{c,Rd} = V_{pl.Rd} = \frac{A_v(f_y/\sqrt{3})}{\gamma_{M0}}$$ (Plastic Shear Resistance)

Refer section B.1 of this Sheet

A_v = Shear Area = $2A/\pi$
= 883800 mm^2
$V_{pl.Rd}$ = 160733 kN

$$\frac{V_{Ed}}{V_{c,Rd}} = \mathbf{0.16} \leq 1.0$$ OK

B.7 Maximum Torsion

BS EN 1993-1-1: 2005 Cl 6.2.7(1)

The design value of the torsion T_{Ed} at each cross-section shall satisfy:

$$\frac{T_{Ed}}{T_{Rd}} \leq 1$$

T_{Rd} = Design torsional resistance of the cross section
T_{Ed} = 0 kNm

The resistance T_{Rd} of a circular hollow section shall be based on the design shear strength.

Therefore check shear induced by torsion is less than shear strength calculated in B.6 above.

$\tau_{t,Ed}$ = Design shear stress due to St. Venant torsion

$$\tau_{t,Ed} = \frac{T_{t,Ed}}{C}$$

Note: For circular tube effect of torsional warping can be neglected ($T_{w,Ed}$ = 0). Therefore $T_{Ed} = T_{t,Ed}$

Where C = Torsional Modulus Constant
C = 2 x W_{el}
C = 2 x 1.67E+09
C = 3.35E+09

$\tau_{t,Ed}$ = 0 kN/m^2
V_{Ed} due to torsion = $\tau_{t,Ed}$ x Shear Area A_v
= 0 x 0.8838
= 0 kN

$$\frac{V_{Ed}}{V_{pl,Rd}} \leq 1.0$$

$$\frac{V_{Ed}}{V_{pl,Rd}} = \mathbf{0.00} \leq 1.0$$ OK

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BS EN 1993-1-1: 2005 Cl 6.2.7(9)

B.8 Torsion and Shear

For combined shear force and torsional moment the plastic shear resistance accounting for torsional effects should be reduced from $V_{pl,Rd}$ to $V_{pl,T.Rd}$ and the design shear force should satisfy :

BS EN 1993-1-1: 2005 eq 6.25 & 6.28

$$\frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1.0$$

$$V_{pl,T.Rd} = \left[1 - \frac{\tau_{t,Ed}}{(f_y/\sqrt{3})/\gamma_{M0}}\right] V_{pl,Rd}$$

T_{Ed} = 19500 kNm

$\tau_{t,Ed} = T_{t,Ed}/C$

$\tau_{t,Ed}$ = 5824 kN/m^2

$T_{t,Ed}$ = 19500 kNm

A_v = 883800 mm^2

$V_{pl.Rd}$ = 160733 kN

$V_{pl,T.Rd}$ = 155585 kN

V_{Ed} = 26000 kN

$$\frac{V_{Ed}}{V_{pl,T,Rd}} = \mathbf{0.17} \leq 1.0$$

OK

BS EN 1993-1-1: 2005 Cl 6.2.8

B.9 Bending and Shear

V_{Ed} = 0 kN

$T_{t,Ed}$ = 19500 kNm

$\tau_{t,Ed}$ = 5824 kN/m^2

M_{Ed} = 310000 kNm

$V_{pl,Rd}$ = 160733 kN

$V_{pl,T.Rd}$ = 155585 kN

Assuming some torsion is present, $V_{pl,T,Rd}$ will be more onerous than $V_{pl,Rd}$. Hence check effect of $V_{pl,T,Rd}$ on moment resistance first:

BS EN 1993-1-1: 2005 Cl 6.2.8(2)

If V_{Ed} is less than half $V_{pl,T,Rd}$ then no reduction in moment resistance needs to be considered.

$2\,V_{Ed} \leq V_{pl,T,Rd}$ *No effect on the Moment Resistance. Ignore boxed section below*

No reduction in moment resistance req'd

BS EN 1993-1-1: 2005 Cl 6.2.8(3)(4)

Reduced Yield Strength = $(1-\rho)\,f_y$

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,T,Rd}} - 1\right)^2$$ (Where torsion is present. See note below)

Note: If there is no torsion present then $V_{pl,Rd} = V_{pl,T,Rd}$ and the above conclusion regarding effect on moment resistance remains the same.

Design Yield Strength for bending in shear = 315 N/mm^2

Design Resistance of the cross section for bending in shear $M_{c,Rd}$ = 527307 kNm

$$\frac{M_{Ed}}{M_{c,Rd}} = \mathbf{0.59} \leq 1.0$$

OK

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BS EN 1993-1-1: 2005 Cl 6.2.10

B.10 Bending, Shear and Axial Force

B.10.1 Effect of Shear Force

Type of Class = Class 3

V_{Ed} = 0 kN

$V_{pl,Rd}$ = 160733 kN

Where the shear force is less than half the plastic shear resistance its effect on the bending and axial force resistance may be neglected.

$2\,V_{Ed} \leq V_{pl,Rd}$ *Hence no reduction in yield strength due to shear only*

but consider fy reduction, if applicable, due to torsional shear

Reduced Yield Strength = $(1-\rho)\,f_y$

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2$$

Design Yield Strength for bending in shear = 315 N/mm^2

B.10.2 Effect of Axial Force

Section is *Class 3* therefore go to section *B.10.2.2* of the following calculations.

BS EN 1993-1-1: 2005 Cl 6.2.9.1(2)

B.10.2.1 Class 1 and 2 Cross Sections

For Class 1 and 2 cross sections following criterion shall be satisfied: $M_{Ed} \leq M_{N,Rd}$

$M_{N,Rd}$ = Design plastic moment resistance reduced due to axial force N_{Ed}

M_{Ed} = 310000 kNm

BS EN 1993-1-1: 2005 Cl 6.2.9.1(6)

$M_{N,y,Rd} = M_{N,z,Rd} = M_{pl,Rd} \times (1-n^{1.7})$

$n = N_{Ed} / N_{pl,Rd}$

N_{Ed} = 13000 kN

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} = 437305 \text{ kN}$$

Overall interaction

BS EN 1993-1-1: 2005 Cl 6.2.1(7)

n = 0.030

$M_{pl,Rd}$ = 683541 kNm

$M_{N,Rd} = M_{N,z,Rd} = M_{N,y,Rd}$ = 681807 kNm $N_{Ed} / N_{Rd} + M_{Ed} / M_{Rd}$ = **0.50** N/A

$M_{Ed} \leq M_{N,Rd}$ OK

Ratio $M_{Ed} / M_{N,Rd}$ = **0.45** N/A

(The check below is stated in the code, but can be considered secondary to that done above)

BS EN 1993-1-1: 2005 Cl 6.2.9.1(6)

For biaxial bending in class 1 and 2 cross sections following criterion shall be satisfied:

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leq 1.0$$

$M_{y,Ed}$ = 0 kNm

$M_{z,Ed}$ = 310000 kNm

α = 2

β = 2

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} = \mathbf{0.21} \leq 1.0$$ OK N/A

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BS EN 1993-1-1: 2005 Cl 6.2.9.2(1)

B.10.2.2 Class 3 Cross Sections

For class 3 cross sections, **in the absence of shear force**, the maximum longitudinal stress shall satisfy the criterion:-

$$\sigma_{x,Ed} \leq \frac{f_y}{\gamma_{M0}}$$

$\sigma_{x,Ed}$ = Design value of the local longitudinal stress due to moment and axial force

$$\sigma_{axial} = N_{Ed}/A$$

N_{Ed} = 13000 kN

A = 1388270 mm^2

σ_{axial} = 9 N/mm^2

$$\sigma_{moment} = M_{Ed}/W_{el}$$

M_{Ed} = 310000 kNm M_{Rd} = 527306.7 kNm M_{Ed}/M_{Rd} **0.59** M_{Rd} inc. yield stress reduction due to shear effects Refer 6.2.8

W_{el} = 1.67E+09 mm^3

σ_{moment} = 185 N/mm^2

$\sigma_{x,Ed}$ = 9 + 185

= 195 N/mm^2

Yield Strength, f_y = 315 N/mm^2

$\frac{f_y}{\gamma_{M0}}$ = 315 N/mm^2

Overall interaction

BS EN 1993-1-1: 2005 Cl 6.2.1(7)

$N_{Ed} / N_{Rd} + M_{Ed} / M_{Rd}$ = **0.63**   Ok

$\sigma_{x,Ed} \leq \frac{f_y}{\gamma_{M0}}$ OK   = **0.62** Stress Ratio   OK

B.11 Buckling Resistance of Members

B.11.1 Uniform Members in Compression

BS EN 1993-1-1: 2005 Cl 6.3.1.1(1)

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0$$

N_{Ed} = 13000 kN

$N_{b,Rd}$ = Design Buckling Resistance of the Compression Member

BS EN 1993-1-1: 2005 Cl 6.3.1.1(3)

$$= \chi \frac{Af_y}{\gamma_{M1}}$$ For Class 1, 2 and 3 Sections

χ = Reduction Factor for the Relevant Buckling Mode

BS EN 1993-1-1: 2005 Eq 6.49

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \leq 1.0$$

$$\phi = 0.5\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right]$$

$\bar{\lambda}$ = Non-dimensional slenderness ratio

BS EN 1993-1-1: 2005 Eq 6.50

$$= \sqrt{\frac{Af_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\lambda_1}$$ For Class 1, 2 and 3 cross sections

L_{cr} = Buckling Length = 15 m

i = 1736 mm (Radius of Gyration)

(SEE CALC PAGE NO FOR ALTERNATIVE CALCULATIONS)

REF. | OUTPUT.

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.9\,\varepsilon$$

$$\lambda_1 = 93.9 \times 0.86 = 81.10$$

$$\bar{\lambda} = 0.11$$

BS EN 1993-5:2007 Cl 5.3.3(3)

The Slenderness criterion for which overall buckling may be assumed to be satisfied is:

$$\frac{N_{Ed}}{N_{cr}} \leq 0.1$$

If this criteria is not met consideration should be given to buckling

$$N_{cr} = \frac{Af_y}{\bar{\lambda}^2}$$

$$N_{cr} = 38539785 \text{ kN}$$

$$\frac{N_{Ed}}{N_{cr}} = \mathbf{0.00} \leq 0.1 \quad \text{OK}$$

OK

For $\bar{\lambda} = 0.11$ $\quad\chi$ can be established from table 6.1, 6.2 and figure 6.4

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection Factor α	0.13	0.21	0.34	0.49	0.76

Cross section limits = Cold Formed

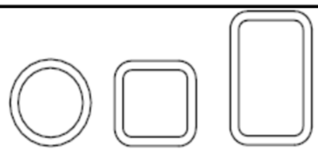
Choose α = 0.49

$$\phi = 0.5\left[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2\right] = 0.483$$

$$\chi = \frac{1}{\phi+\sqrt{\phi^2-\bar{\lambda}^2}} = 1.05 \geq 1.0$$

Use χ = 1

BS EN 1993-1-1:2005 Table 6.2

Cross section		Limits	Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	Buckling curve S 460
Hollow sections		hot finished	any	a	a_0
		cold formed	any	c	c

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BS EN 1993-1-1:2005 Figure 6.4

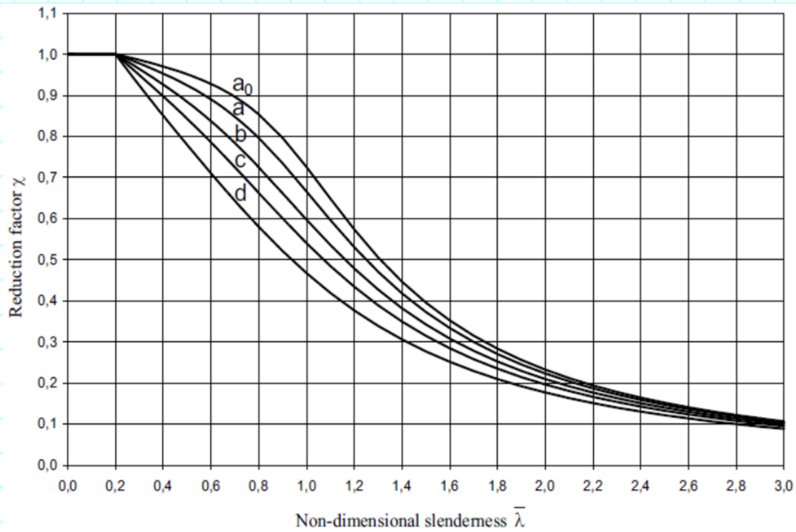

Figure 6.4: Buckling curves

χ = 1

$$N_{b,Rd} = \chi \frac{Af_y}{\gamma_{M1}}$$

= 437305 kN

$$\frac{N_{Ed}}{N_{b,Rd}} = \mathbf{0.03} \leq 1.0 \quad \text{OK}$$

OK

B.11.2 Uniform Members in Bending

BS EN 1993-1-1:2005 Cl 6.3.2.1 (2)

Beams with sufficient restraint to the compression flange are not susceptible to lateral-torsional buckling. In addition, beams with certain types of cross sections, such as square or circular hollow sections, fabricated circular tubes or square box sections are not susceptible to lateral-torsional buckling.

Therefore it is not necessary to consider the effects of lateral-torsional buckling for steel tubular piles

B.11.3 Uniform Members in Bending and Axial Compression

BS EN 1993-1-1:2005 Cl 6.3.3 (4) & Eq 6.61/6.62

Members which are subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{(\chi \frac{N_{Rk}}{\gamma_{M1}})} + k \frac{M_{Ed} + \Delta M_{Ed}}{\chi_{LT} (\frac{M_{Rk}}{\gamma_{M1}})} \leq \; \leq \; 1.0$$

This equation has been simplified as for a tubular pile only single axis needs to be considered.

REF. OUTPUT.

BS EN 1993-1-1:2005 Table 6.7

Table 6.7: Values for $N_{Rk} = f_y A_i$, $M_{i,Rk} = f_y W_i$ and $\Delta M_{i,Ed}$

Class	1	2	3	4
A_i	A	A	A	A_{eff}
W_y	$W_{pl,y}$	$W_{pl,y}$	$W_{el,y}$	$W_{eff,y}$
W_z	$W_{pl,z}$	$W_{pl,z}$	$W_{el,z}$	$W_{eff,z}$
$\Delta M_{y,Ed}$	0	0	0	$e_{N,y}\,N_{Ed}$
$\Delta M_{z,Ed}$	0	0	0	$e_{N,z}\,N_{Ed}$

NOTE For members not susceptible to torsional deformation χ_{LT} would be $\chi_{LT} = 1{,}0$.

$N_{Rk} = f_y A = 315 \times 1388270 \times 10^{-3}$

$= 437305$ kN

Type of Class = Class 3

Use

$W_y = W_z = W = Wel$

$M_{Rk} = f_y \times Wel$

$= 315 \times 1.67E+09$

$= 5.27E+11$ Nmm

$= 527307$ kNm

BS EN 1993-1-1:2005 Table 6.7

$\Delta M_{y,z,Ed} = 0$ (For class 1, 2 and 3 sections)

k = Interaction factor

Method 2 shall be used to calculate k factors. In accordance with Steel Construction Institute advice, circular hollow sections can be used in this approach even though reference is only made to I and RHS sections.

BS EN 1993-1-1:2005 Annex B Table B.1

Table B.1: Interaction factors k_{ij} for members not susceptible to torsional deformations

Interaction factors	Type of sections	Design assumptions: elastic cross-sectional properties class 3, class 4	Design assumptions: plastic cross-sectional properties class 1, class 2
k_{yy}	I-sections RHS-sections	$C_{my}\left(1+0{,}6\bar{\lambda}_y\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\le C_{my}\left(1+0{,}6\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$	$C_{my}\left(1+\left(\bar{\lambda}_y-0{,}2\right)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\le C_{my}\left(1+0{,}8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$
k_{yz}	I-sections RHS-sections	k_{zz}	$0{,}6\,k_{zz}$
k_{zy}	I-sections RHS-sections	$0{,}8\,k_{yy}$	$0{,}6\,k_{yy}$
k_{zz}	I-sections	$C_{mz}\left(1+0{,}6\bar{\lambda}_z\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\le C_{mz}\left(1+0{,}6\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$	$C_{mz}\left(1+\left(2\bar{\lambda}_z-0{,}6\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\le C_{mz}\left(1+1{,}4\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
	RHS-sections		$C_{mz}\left(1+\left(\bar{\lambda}_z-0{,}2\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\le C_{mz}\left(1+0{,}8\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$

For I- and H-sections and rectangular hollow sections under axial compression and uniaxial bending $M_{y,Ed}$ the coefficient k_{zy} may be $k_{zy} = 0$.

PROJECT TITLE: ENECO MV2 WINDFARM - MONOPILE FOUNDATION CONCEPT DESIGN

SUBJECT: SOUTH PROFILE - FULL EROSION (NAP -7m) - CYCLIC DEGRADATION

PROJECT NO: BG8375 FILE REF: REV:

PREPARED BY: CHA DATE: 1-5-2020 REV DATE:

CHECKED BY: DATE:

(SEE CALC PAGE NO FOR ALTERNATIVE CALCULATIONS)

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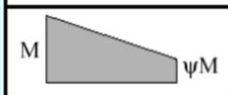
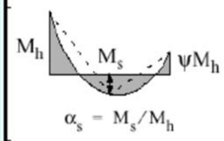
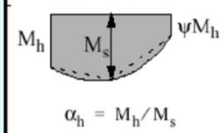
REF. OUTPUT.

BS EN 1993-1-1:2005 Annex B Table B.3

Table B.3: Equivalent uniform moment factors C_m in Tables B.1 and B.2

Moment diagram	range		C_{my} and C_{mz} and C_{mLT} uniform loading	concentrated load
	$-1 \le \psi \le 1$		$0,6 + 0,4\psi \ge 0,4$	
	$0 \le \alpha_s \le 1$	$-1 \le \psi \le 1$	$0,2 + 0,8\alpha_s \ge 0,4$	$0,2 + 0,8\alpha_s \ge 0,4$
	$-1 \le \alpha_s < 0$	$0 \le \psi \le 1$	$0,1 - 0,8\alpha_s \ge 0,4$	$-0,8\alpha_s \ge 0,4$
$\alpha_s = M_s/M_h$		$-1 \le \psi < 0$	$0,1(1-\psi) - 0,8\alpha_s \ge 0,4$	$0,2(-\psi) - 0,8\alpha_s \ge 0,4$
	$0 \le \alpha_h \le 1$	$-1 \le \psi \le 1$	$0,95 + 0,05\alpha_h$	$0,90 + 0,10\alpha_h$
	$-1 \le \alpha_h < 0$	$0 \le \psi \le 1$	$0,95 + 0,05\alpha_h$	$0,90 + 0,10\alpha_h$
$\alpha_h = M_h/M_s$		$-1 \le \psi < 0$	$0,95 + 0,05\alpha_h(1+2\psi)$	$0,90 - 0,10\alpha_h(1+2\psi)$

For members with sway buckling mode the equivalent uniform moment factor should be taken $C_{my} = 0,9$ or $C_{Mz} = 0,9$ respectively.

C_{my}, C_{mz} and C_{mLT} should be obtained according to the bending moment diagram between the relevant braced points as follows:

moment factor	bending axis	points braced in direction
C_{my}	y-y	z-z
C_{mz}	z-z	y-y
C_{mLT}	y-y	y-y

BS EN 1993-1-1:2005 Annex B Table B.3

M_{max} = 310000 kNm

M_{min} = 0 kNm

ψ = 0.00

M_s = 0.0 kNm (local moment)

Equivalent uniform moment factor

C_m = 1.00

Table B.1 gives the following formulas to calculate Interaction factor k for different cross section.

(Tubular members are not susceptible to torsional deformations)

For Class 1 and class 2 cross section:

BS EN 1993-1-1:2005 Annex B Table B.1

$$k = C_m\left[1+(\bar{\lambda}-0.2)\frac{N_{Ed}}{\chi N_{Rk}/\gamma_{M1}}\right] \le C_m\left[1+0.8\frac{N_{Ed}}{\chi N_{Rk}/\gamma_{M1}}\right]$$

These equations have been simplified as for tubular pile only single axis needs to be considered

And

For Class 3 cross section:

$$k = C_m\left[1+0.6\bar{\lambda}\frac{N_{Ed}}{\chi N_{Rk}/\gamma_{M1}}\right] \le C_m\left[1+0.6\frac{N_{Ed}}{\chi N_{Rk}/\gamma_{M1}}\right]$$

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REF.		OUTPUT.
Refer section B.10.1 of this Sheet	Type of class = Class 3	
	$\bar{\lambda}$ = 0.11	
	N_{Ed} = 13000 kN	
	N_{Rk} = 437305 kN	
	χ = 1	
	χ_{LT} = 1.0 *(Circular hollow sections not susceptible to lateral torsional buckling)*	
	γ_{M1} = 1	
	M_{Ed} = 310000 kNm	
	ΔM_{Ed} = 0 kNm	
	M_{Rk} = 527307 kNm	
	k = 1.002	
	$\frac{N_{Ed}}{(\chi \frac{N_{Rk}}{\gamma_{M1}})} + k \frac{M_{Ed} + \Delta M_{Ed}}{\chi_{LT} (\frac{M_{Rk}}{\gamma_{M1}})}$ = **0.62** ≤ 1.0 OK	OK

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Eneco

A2 Input turbine suppliers

A2.1 Vestas V162 – preliminary extreme loads

Mbt1: Resulting bending moment. SQRT(Mxt1^2 + Myt1^2) (also M_{res})
FndFr: Resulting shear force. SQRT(Fxt1^2 + Fyt1^2) (also F_{res})

Characteristic Extreme							
Lead	LC/Family	PLF	Type	Mbt1	Mzt1	FndFr	Fzt1
Sensor	[-]	[-]	[-]	[kNm]	[kNm]	[kN]	[kN]
Mbt1	23NTMVrm00(fam162)	1.49	Abs	**153600**	2513	1366	-7804
Mzt1	23NTMHWO100(fam166)	1.49	Abs	51090	**-17880**	862.9	-7529
FndFr	23NTMVrm00(fam162)	1.49	Abs	144000	3143	**1474**	-7796
Fzt1	42NTMRPVo00(fam249)	1.49	Abs	32600	6132	209.9	**-7891**

Table 2-1 Characteristic Extreme (excl. PLF). Load cases sorted with PLF.

Characteristic Extreme							
Lead	LC/Family	PLF	Type	Mbt1	Mzt1	FndFr	Fzt1
Sensor	[-]	[-]	[-]	[kNm]	[kNm]	[kN]	[kN]
Mbt1	14EcdVrma00(fam43)	1.35	Abs	**154600**	1726	1312	-7777
Mzt1	23NTMHWO100(fam166)	1.49	Abs	51090	**-17880**	862.9	-7529
FndFr	23NTMVrm00(fam162)	1.49	Abs	144000	3143	**1474**	-7796
Fzt1	12IceUHWO200(fam27)	1.35	Abs	61780	2696	509.5	**-7968**

Table 2-2 Characteristic Extreme (excl. PLF). Load cases sorted without PLF.

Characteristic Extreme							
Lead	LC/Family	PLF	Type	Mbt1	Mzt1	FndFr	Fzt1
Sensor	[-]	[-]	[-]	[kNm]	[kNm]	[kN]	[kN]
Mbt1	62E50b06000(fam291)	1.10	Abs	**141100**	3004	1273	-7638
Mzt1	22OSFHWO200(fam103)	1.10	Abs	27150	**-14760**	194.0	-7544
FndFr	62E50b06000(fam291)	1.10	Abs	140100	2658	**1287**	-7629
Fzt1	22VOGVo00(fam112)	1.10	Abs	53730	3775	432.7	**-7889**

Table 2-3 Characteristic Extreme (excl. PLF). Only load cases with PLF = 1.10.

Characteristic Extreme							
Lead	LC/Family	PLF	Type	Mbt1	Mzt1	FndFr	Fzt1
Sensor	[-]	[-]	[-]	[kNm]	[kNm]	[kN]	[kN]
Mbt1	14EcdVrma00(fam43)	1.35	Abs	**154600**	1726	1312	-7777
Mzt1	98NTM2300(fam468)	1.35	Abs	39860	**-15720**	264.3	-7499
FndFr	1310etm00(fam35)	1.35	Abs	129900	-2116	**1377**	-7776
Fzt1	12IceUHWO200(fam27)	1.35	Abs	61780	2696	509.5	**-7968**

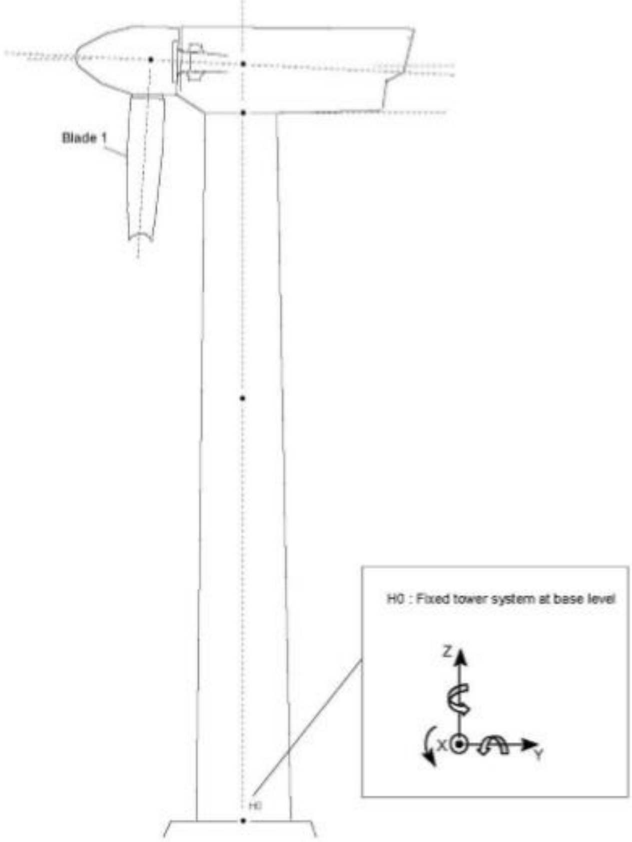
Table 2-4 Characteristic Extreme (excl. PLF). Only load cases with PLF = 1.35.

The nominal spring stiffness used for the load calculations is 500 GNm/rad resulting in a nominal tower frequency of 0.189 Hz. The spring stiffness of the foundation must be at least $C_{\varphi,dyn} \geq 64$ GNm/rad for the loads to be valid. Concurrent values for rotational- and lateral stiffness of the foundation are given in Table 5-1.

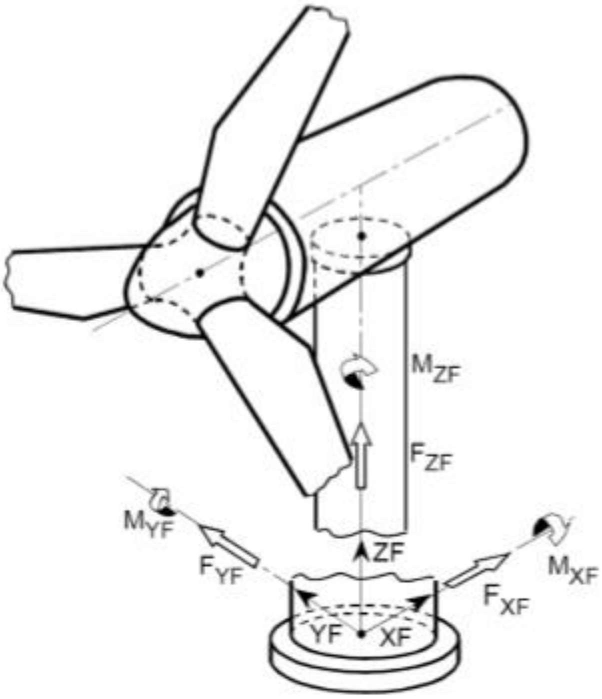
Rotational stiffness	[GNm/rad]	77	105	143	196	268	366	445	500
Lateral stiffness	[MN/m]	32.7	13.8	10.5	8.5	7.6	7.0	6.8	6.8

Table 5-1 Minimum lateral stiffness.

Appendix A. Co-ordinate systems



A2.2 Enercom – E160 – preliminary extreme loads



Title	Foundation Design Loads - E-160 E2 EP5-MST-120-FB-C-01
Document number	M00-C8-30-10861-R0-1
Date	24.03.2020, 16:39:58
Author	Mun Jung
Document revision	R0

NOT RELEASED FOR PRODUCTION

REVISIONS

R0	Initial document
R1	
R2	

Wind Zone

DIBt, October 2012	WZ2 GK II
IEC 61400-1 3rd Edition, 2005-08	WC IIIA (AW7.5 TI16.0)
Design Lifetime	20 years

Technical data

Dead Weight nacelle + rotor	Fz = -2945 kN
Dead Weight tower	Fz = -4288 kN
Total Weight	Fz = -7233 kN
Xcog nacelle + rotor w.r.t tower top axis	Sx,Tower = -4.556 m
Zcog nacelle + rotor w.r.t tower top axis	Sz,Tower = 2.195 m

Ground parameters (miminal value)

Flat Foundation

- Dynamic rotational spring constant	Kφ,dyn,flat ≥ 160000 MNm/rad
- Static rotational spring constant	Kφ,stat,flat ≥ 26666 MNm/rad

Pile Foundation

- Dynamic rotational spring constant	Kφ,dyn,pile ≥ 160000 MNm/rad
- Static rotational spring constant	Kφ,stat,pile ≥ 26666 MNm/rad
- Dynamic translational spring constant	KF,dyn,pile ≥ 500 MN/m

Coordinate System

The coordinate system has its origin at the intersection of the tower axis and the upper edge of the foundation, and does not rotate with the nacelle.
- X: horizontal
- Z: vertically upwards in direction of the tower axis
- Y: horizontally sideways, so that X, Y, Z rotate clockwise

Ultimate loads at top of foundation

- Dynamic properties of the wind turbine (e.g. gust reactions) as well as the effects of imperfections have been considered in the load cases.
- All loads refer to the top of foundation.
- Loss of pressure between soil and foundation, maximally as far as centre of gravity of foundation area for load case Group N/T/DLC8.2.
- No loss of pressure between soil and foundation and no tension in the piles for load case NTM DLC D.3
- Loads for all German earthquake areas are covered according to [1] with stated loads
- Additional saftey factor of 1.05 is applied on the listed load values.

Table 1 Summary of extreme loads at the tower base including consideration of the additional moment

Load case description	Partial safety factors	Fz,min* [kN]	Fz,max* [kN]	Fxy [kN]	Mxy [kNm]	Mz [kNm]
Group A	with	-9576	-6481	1856	229395	-18002
Group N/T	with	-12848	-7759	1878	228911	-19428
Group N/A/T	with	-12848	-6481	1878	229395	-19428
Group N/A/T	without	-8727	-5747	1688	207572	-16365
Group N/T/DLC8.2	without	-8727	-5747	1391	166806	-14391
Group DLC8.1/8.2	without	-8700	-5926	821	96915	-8858
NTM DLC D.3**	without	-8727	-5747	838	94051	4840

- *The value of Fz,min and Fz,max consider partial saftey factor of $\gamma F = 1.1$ and $\gamma F = 0.9$.
- **|Mz| is taken in probability calculation.
- Due to dynamic action from the machine Fz is no longer constant as per calculated from dead weight but fluctuates between the given values of Fz,min and Fz,max.

Load Case Description and Partial Safety Factors acc. to [2] and [3]

Group N	Normal Design Load Case.
Group A	Abnormal Design Load Case.

Group T	Transport and Erection
NTM DLC 8.1 / EWM DLC8.2	Transport, assembly, maintenance and repair turbine states which may persist for longer than one week.
NTM DLC D.3	Operating loads, normal turbulence model with probability exceeding 10^{-2}.
DLC D.5a / 6a	Earthquake wind loads (International) according to [1].

References

[1]	Auslegung von Bauwerken gegen Erdbeben – Teil 1: Grundlagen, Erdbebeneinwirkungen und Regeln für Hoch-bauten; Deutsche Fassung EN 1998-1:2004 + AC:2009 + DIN EN 1998-1/NA:2011-01
[2]	IEC 61400-1:2005-08, Wind Turbines – Part 1: Design requirements, Third Edition, 2005-08
[3]	DIN EN 61400-1:2011-08 Windenergieanlagen – Teil 1: Auslegungsanforderungen (IEC 61400-1:2005 + A1:2010)

A2.3 General Electric GE158 – extreme loads

Load case	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]	Fr [kN]	Mr [kNm]	γ [-]
DLC 2.2	**5950.8**	-338.2	325.1	-9445.5	18136.8	27749.7	469.0	33150.2	1.00
DLC 6.2	5634.9	**-958.4**	-116.6	-2460.4	-20254.9	83768.5	965.5	86182.8	1.00
DLC 2.3	5776.0	-117.9	**1161.5**	-1609.5	115629.4	8409.4	1167.5	115934.8	1.00
DLC 2.2	5811.8	-22.9	247.5	**-10990.8**	24965.4	4746.6	248.5	25413.6	1.00
DLC 2.3	5772.0	-118.7	1160.2	-1407.2	**115794.9**	8444.7	1166.2	116102.4	1.00
DLC 6.2	5604.7	-942.4	-300.0	-2564.3	-43038.3	**86175.1**	989.0	96325.6	1.00
DLC 2.3	5773.6	-160.9	1159.0	-1519.2	115516.5	9288.4	**1170.1**	115888.8	1.00
DLC 2.3	5772.0	-118.7	1160.2	-1407.2	115794.9	8444.7	1166.2	**116102.4**	1.00

Table 1: All IEC 61400 Design Situations and Load Cases; excluding partial safety factor

Load case	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]	Fr [kN]	Mr [kNm]	γ [-]
DLC 1.3	**5885.4**	52.0	568.3	836.5	40671.3	-2526.5	570.7	40749.3	1.00
DLC 1.5	5667.1	**-728.6**	125.8	2502.1	2237.1	27747.3	739.4	27837.5	1.00
DLC 1.3	5793.3	40.7	**1003.6**	-416.6	78285.3	4584.5	1004.4	78419.6	1.00
DLC 1.3	5609.5	-117.5	309.3	**10025.1**	25226.5	17795.0	330.9	30869.8	1.00
DLC 1.4	5778.5	12.4	911.9	-2446.3	**91981.2**	6173.6	912.0	92188.2	1.00
DLC 6.1	5586.8	-579.4	118.4	-3049.9	123.9	**53745.6**	591.3	53746.1	1.00
DLC 1.3	5747.4	102.3	1003.0	-2599.1	82584.9	5595.7	**1008.2**	82774.6	1.00
DLC 1.4	5778.5	12.4	911.9	-2446.3	91981.2	6173.6	912.0	**92188.2**	1.00

Table 1A: IEC 61400 Normal Design Situations and Load Cases Only; excluding partial safety factor

Load case	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]	Fr [kN]	Mr [kNm]	γ [-]
DLC 8.1	**8694.6**	-10.1	110.1	-1669.2	3565.5	4675.4	110.6	5879.8	1.50
DLC 6.2	6198.4	**-1054.2**	-128.2	-2706.5	-22280.4	92145.3	1062.0	94800.7	1.10
DLC 1.3	7821.0	55.0	**1354.9**	-562.4	105685.2	6189.1	1356.0	105866.2	1.35
DLC 1.3	7572.8	-158.6	417.6	**13533.8**	34055.8	24023.3	446.7	41676.3	1.35
DLC 2.3	6349.2	-130.5	1276.2	-1547.9	**127374.4**	9289.1	1282.8	127712.6	1.10
DLC 6.2	6165.1	-1036.7	-329.9	-2820.8	-47342.1	**94792.7**	1087.9	105957.2	1.10
DLC 1.3	7758.9	138.2	1354.1	-3508.8	111489.6	7554.2	**1361.1**	111745.2	1.35
DLC 2.3	6349.2	-130.5	1276.2	-1547.9	127374.4	9289.1	1282.8	**127712.6**	1.10

Table 2: All IEC 61400 Design Situations and Load Cases; including partial safety factor

Load case	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]	Fr [kN]	Mr [kNm]	γ_F [-]
DLC 1.1	5784.7	43.5	732.7	2120.0	72243.5	11257.7	733.9	73115.4	1.00

Table 7: Load cases for check against pile tension loading

The minimum values for the dynamic foundation stiffness that have to be achieved are:

$$k_{\varphi,min} = 1.5 \cdot 10^8 \text{ kNm/rad}; \quad k_{yz,min} = 1.0 \cdot 10^6 \text{ kN/m}$$

The minimum value for the static foundation stiffness that has to be achieved is 1/5 of the dynamic stiffness:

$$k_{\varphi,stat,min} = 3.0 \cdot 10^7 \text{ kNm/rad}$$

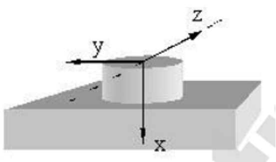
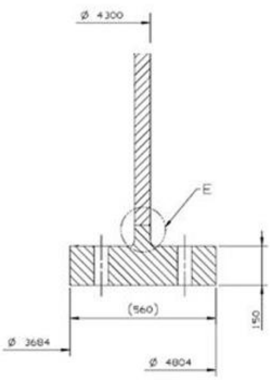
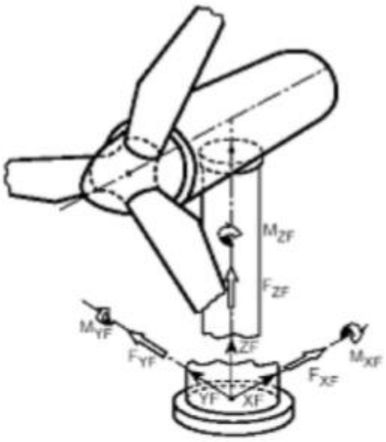


Figure 3: Coordinate System



A2.4 Siemens Gamesa



XF horizontal
ZF vertically upwards in direction of the tower axis
YF horizontally sideways, so that XF, YF, ZF rotate clockwise

Figure 1 Coordinate system

Extreme load

The extreme loads for the design of the SG 6.0-155 T102.5-50A foundations are shown in *Table 3*.

Load case	Load facto r	F_x (kN)	F_y (kN)	F_z (kN)	F_{xy} (kN)	M_x (kNm)	M_y (kNm)	M_z (kNm)	**M_{xy} (kNm)**
dlc62_V42.5_030_s6	1,1	-28,18	-1509,07	-6261,58	1509,33	152409,4	-13619,72	1657,48	**153016,71**

Table 3 SG 6.0-155 T102.5-50A Factored/Unfactored Extreme loads at tower bottom

Characteristic load

Characteristics loads (maximum M_{xy} bending moment load combination of groups N, E and T according to GL2012 Sec. 5.4.3.1.3, or equivalent groups N-T according to IEC 61400-1 2006) have been estimated as shown in *Table 4*:

Load case	F_x (kN)	F_y (kN)	F_z (kN)	F_{xy} (kN)	M_x (kNm)	M_y (kNm)	M_z (kNm)	**M_{xy} (kNm)**
dlc62_V42.5_030_s6	-25,62	-1371,88	-5692,34	1372,12	138554	-12381,56	1506,8	**139106,1**

Table 4 SG 6.0-155 T102.5-50A Characteristics Loads at the base of the tower

Quasi-permanent load

Loads according to GL2010, considering DLC 1.1 and 6.4 with a probability of exceedance of pf = 10^{-2} (equivalent to 1750 h in 20 years) with $\gamma F = 1.0$ have been estimated as shown in Table 5:

pf=0.01000	Tower loads at section							
Section Height from bottom (m)	Fx (KN)	Fy (KN)	Fxy (KN)	Fz (KN)	Mx (KNm)	My (KNm)	Mxy (KNm)	Mz (KNm)
0	872,32	98,51	872,67	-5726,9	16276,45	91477	**91849,96**	3965,41

Table 5 SG 6.0-155 T102.5-50A Quasi Permanent Loads at tower bottom

WTG	SG 6.0-155 T102.5-50A
Minimum rotational stiffness of the foundation	1.5E+11 Nm/rad

Table 2 SG 6.0-155 T102.5-50A Minimum rotational stiffness