EFFECTS OF DEEP WATER ON MONOPILE SUPPORT STRUCTURES FOR OFFSHORE WIND TURBINES

W.E. de Vries and V.D. Krolis
Duwind, Faculty of Civil Engineering and Geosciences, Delft University of Technology
Stevinweg 1, 2628 CN Delft, The Netherlands
Tel.: +31 15 27 87568, E-mail: w.e.devries@tudelft.nl

SUMMARY

As offshore wind farms will be required to move further offshore in the future, turbines will likely be installed in deeper water. This paper investigates the effect of placing a turbine in deep water has on the mass of a monopile support structure. To this end the buckling of a section at the mudline is assessed. A simplified design is made for 7 different water depths ranging from 20 to 50 m. The wall thickness is optimized based on the buckling checks. The results show that the mass of the support structure increases dramatically for increasing water depths and that for the larger water depths the overturning moment is dominated by the hydrodynamic loads.

Keywords: Deep water, monopile, support structure, mass, design

1 INTRODUCTION

More offshore wind farms are being built every year. The most favourable location for an offshore wind farm is generally near shore, as costs for the support structure are lower for a shallow water location and the costs for grid connections are lower if the distance to shore is not so large. However, such favourable locations are rare as offshore wind farms require a lot of space and the seafloor in near shore locations is in use for many other purposes, i.e. shipping lanes, military zones, cables and pipelines. Therefore it will eventually be necessary to move to locations further away from the shore. This implies a move to deeper waters.

Currently the most common support structure configuration is the monopile. It has been applied in waters up to 25 m deep. If a monopile support structure is placed in deeper water, hydrodynamic loads and bending moments at the seabed increase, and the dynamic behaviour of the structure changes. This paper investigates the effects of water depth on the structure with respect to the response to extreme loads. To this end a monopile support structure will be subjected to an extreme load analysis for water depths ranging from 20 to 50 m.

Approach

To obtain results which are relevant to current design practice for offshore wind turbines, this study will be based on actual environmental data. The steps in the normal preliminary design procedure will be followed, although the level of detail may be lower in order to keep the amount of work in hand, as multiple design are required. For the same reasons simplifications will be adopted when acceptable. A support structure design is made for each water depth starting at 20 m and increasing with a step size of 5 m. This results in 7 monopile support structure designs which are established with the same set of design rules. A target natural frequency is determined, to which the 7 support structures will be designed. After determining the preliminary geometry, the penetration depth is established. The designs will subsequently be subjected to buckling checks for a single location at the mudline. If the buckling check is satisfied, the wall thickness may be optimized until the final wall thickness is found. If not, the wall thickness is increased and the buckling check repeated. The total mass of each of the final support structure designs is recorded. The key parameters of the designs are presented, as well as the monopile diameter and overturning moment for the Ultimate Limit State as functions of the water depth.

2 SITE AND ENVIRONMENTAL DATA

Location

A reference location has been selected for which the environmental data has been gathered. This location is situated in the Dutch part of the North Sea at approximately 53º39’N, 3º57’E. Wind, wave and current data were obtained from the NEXTRA database. Access to this database was provided by Shell within the framework of the Upwind project. Figure 1 gives a general indication of this selected location, while the origin of the different environmental data is depicted in more detail in figure 2.

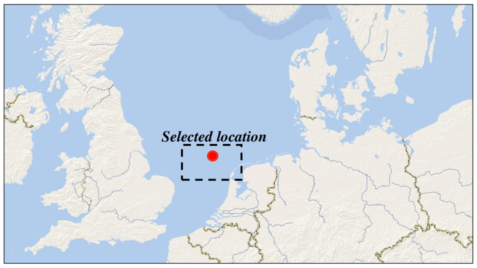


Figure 1: *Selected location for this study*

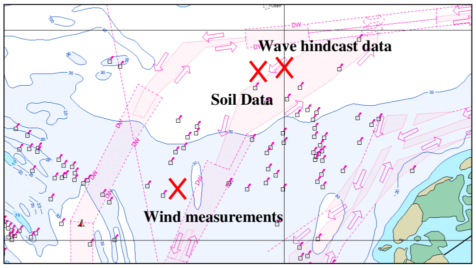


Figure 2: *Origin of different environmental data*

Water depth

The actual water depth at the selected location is approximately 40 m. However, as the purpose of this investigation is to study the effects of water depth on monopile support structures, the water depth is taken as a variable. A total of 7 different water depths ranging from 20 to 50 m with a step size of 5 m will be used.

Soil Data

The soil conditions for the study area have been obtained from another current project at Delft University of Techology. In this project, soil data was provided by Fugro, a leading company in the field of geotechnical data collection. The soil data comprises the effective unit weight of the soil γ', the angle of internal friction δ in the case of sand and the undrained shear strength c_u for clay. The profile consits mainly of sand with a few thin layers of clay and silt. The soil characteristics are given in table 1.

Table 1: *Soil profile for selected location.*

Layer Number	Soil Type	γ' [kN/m³]	δ [°]	$c_{u,top}$ [kPa]	$c_{u,bottom}$ [kPa]
1	Sand	8.5	15	-	-
2	Clay	8.0	-	20	20
3	Sand	9.0	20	-	-
4	Clay	8.0	-	30	30
5	Sand	11.0	35	-	-
6	Sand	9.5	25	-	-
7	Sand	11.5	35	-	-
8	Silt	9.5	15	-	-
9	Sand	10.5	25	-	-
10	Sand	11.0	30	-	-
11	Silt	9.5	20	-	-
12	Sand	10.5	25	-	-
13	Clay	9.5	-	250	250
14	Sand	10.5	25	-	-
15	Sand	11.0	30	-	-
16	Sand	11.0	30	-	-
17	Sand	10.5	25	-	-
18	Sand	11.5	35	-	-

Water Levels and Scour

All levels and elevations will be referred to Low Astronomical Tide (LAT). The tidal range has been obtained for a location at 53°37'00"N, 4°12'00"E. This is approximately 30 km from the selected location. However, as the circumstances are similar for both locations, the tidal range can be assumed to apply to the proposed location as well. The tidal range is 1.6m. Based on experience that Delft University of Technology has gained during previous studies in the field of offshore engineering for locations on the North Sea a storm surge value of 2.0 m may be adopted for the given location. For this study it is assumed that scour protection will prevent the occurrence of scour. Therefore scour will not be considered.

Extreme Wind and Waves and Current

The wave height as a function of return period is given in table 2. The design wave height H_D is equal to the maximum wave height with a return period of 50 years, $H_{max,50}$. Thus the design wave height is 20.29 m. The maximum current velocity is 0.82 m/s. The current data as a function of return period is given in table 2. The extreme wind speeds as a function of the return period are also listed in table 2. The data in the last column of this table is valid for 100 m above sea level.

Table 2: *Extreme wind velocity, current velocity and wave heights as a function of return period.*

T_{return} [yr]	H_s [m]	H_{max} [m]	U_c [m/s]	$V_{w,}$ [m/s]
1	7.72	14.35	0.70	33.85
5	9.03	16.80	0.80	37.99
10	9.60	17.85	0.84	39.78
50	10.91	20.29	0.94	43.92
100	11.48	21.34	0.98	45.70

Turbine

The turbine used in this study is the Vestas V90. It is representative for the offshore wind turbines currently being installed in the North Sea. The most important parameters are listed in table 3.

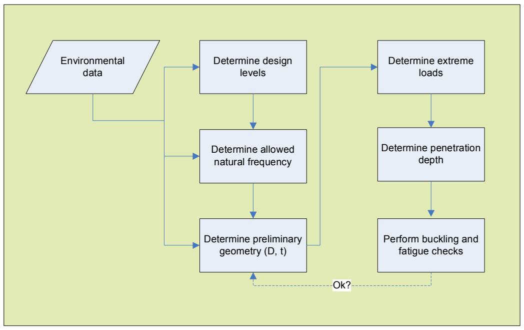
Table 3: *Key parameters for the Vestas V90 turbine.*

Power rating	3.0	MW
Turbine mass	108	ton
Rotor diameter	90	m
Nominal rotor speed	16.1	rpm
Rotational interval	8.6 – 18.4	rpm
Cut-in windspeed	4	m/s
Nominal windspeed	15	m/s
Cut-out windspeed	25	m/s

3 DESIGN

The design procedure is depicted in figure 3. Based on the environmental data and the rotor diameter the design levels are determined. Using the turbine properties and a wave spectrum that is representative for fatigue, the allowable natural frequency band can be determined. Based on this allowable frequency band a target natural frequency is set. Subsequently, the diameter and the wall thickness of the support structure are chosen such that the target frequency is attained. This is an iterative process in which a set ratio between the diameter and the wall thickness is maintained. The diameter, having the largest effect on the natural frequency, is varied until the desired natural frequency is obtained. With the geometry known, the extreme loads can be determined. The extreme loads are due to wind, wave and current loads. Usually a combination of an extreme wind speed and a reduced maximum wave height or a reduced wind speed and an extreme wave height is applied. Conservatively, the maximum wind speed, current and wave height can be combined. The appropriate design standards should be consulted. Using the thus determined loads, the penetration depth can be determined. To this end the lateral and axial stability of the foundation is considered. For a monopile foundation the lateral stability is generally governing. Subsequently, buckling checks are performed. If the buckling check is not satisfied, the wall thickness of the support structure must be increased.

Figure 3: *General design process for the preliminary design of monopile support structures*



Determining the design levels

The first step in the preliminary design process is the determination of design levels for the platform and the hub height. The platform level is of importance as it is located at the top of the transition piece and it is the location of the flange connection between the transition piece and the turbine tower. The hub height should be known as the wind loads are calculated at that level. Furthermore, the location of the centre of gravity of the nacelle mass is dependent on the hub height. This parameter has a large impact on the natural frequency. Therefore this level should be determined at the earliest stage. Figure 4 indicates the various design levels for a monopile support structure.

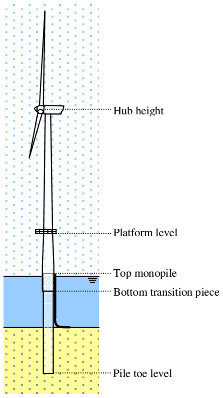


Figure 4: *Design levels for a monopile offshore wind turbine*

The reference level used for the preliminary design of the monopile is Lowest Astronomical Tide (LAT). The top of the monopile is set at LAT + 0.5 m. The required platform level can be found by adhering to:

$$z_{platform} = LAT + \Delta z_{tide} + \Delta z_{surge} + \Delta z_{air} + \xi *$$

With:

$z_{platform}$ = Platform level
Δz_{tide} = Tidal range
Δz_{surge} = Storm surge
Δz_{air} = Air gap
$\xi *$ = Highest wave elevation above still water level

The highest wave elevation can be found with $\zeta^* = H_D$ in which H_D is the design wave height and δ is the wave elevation coefficient. The design wave height is equal to the maximum wave height with a 50-year return period $H_{max,50}$. However, H_D cannot exceed the wave breaking limit H_B, which has been determined empirically at 0.78 times the local water depth. For δ a value of 0.65 will be maintained [1]. To determine the design period T_D the following equation is used, in which $H_{s,50}$ is the significant wave height associated with a return period of 50 years [2]:

$$11.1\sqrt{H_{s,50}/g} \le T_D \le 14.1\sqrt{H_{s,50}/g}$$

The hub height can easily be determined using the previously defined platform level as a starting-point. The hub height is determined by the platform level, a blade clearance $\Delta z_{clearance}$ and the rotor diameter D_{rotor}. The blade clearance is the distance between the blade tip in its lowest position and the platform. This distance should be sufficient to allow safe access to the platform for personnel and equipment. A blade clearance of 5 m is deemed to be sufficient. Table 4 lists the hub heights that were calculated according to:

$$z_{hub} = z_{platform} + \Delta z_{clearance} + 0.5 \cdot D_{rotor}$$

Table 4: *Design levels per water depth*

Water depth [m]	*Platform level* [m + LAT]	*Hub height* [m + LAT]
20	15.25	65.25
25	18.00	68.00
30 - 50	18.50	68.50

Determining the required natural frequency

The first natural frequency of the support structure is a very important parameter as it determines the dynamic behaviour of the offshore wind turbine. If the frequency of excitation is near the natural frequency, resonance occurs and the resulting response will be larger than in the quasi-static case. This leads to higher stresses in the support structure and, more importantly to higher stress ranges, an unfavourable situation with respect to the fatigue life of the offshore wind turbine. Therefore it is important to ensure that the excitation frequencies with high energy levels do not coincide with the natural frequency of the support structure. In the case of an offshore wind turbine excitation is due to both wind and waves. For fatigue considerations sea states with a high frequency of occurrence have the largest effect. These are generally relatively short waves with a significant wave height H_s of around 1 m to 1.5 m and a zero-crossing period T_z of around 4 s to 5 s. The wind excitation frequencies that should be avoided are those that coincide with the range of rotational frequencies of the rotor. With a minimum rotational speed at the cut-in wind speed of 8.6 rpm and a maximum rotational speed of 18.4 rpm, the rotational frequency interval to stay clear of ranges from 0.15 Hz to 0.31 Hz. This interval is indicated with 1P. Furthermore, the blade-passing frequency interval should also be avoided. This interval, indicated with 3P for a triple bladed turbine is equal to the rotational frequency interval times the number of blades. Taking the above into account, the natural frequency is chosen at 0.32 Hz, as indicated in Figure 5.

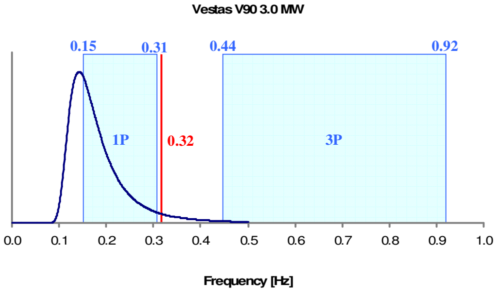


Figure 5: *Diagram showing natural frequency and excitation frequencies*

Determining preliminary geometry

Using the selected natural frequency of 0.32 Hz as a target, the dimensions of the tubular elements of the support structure are adjusted. In the case of the monopile the diameter of the pile is varied. The diameter of the transition piece subsequently depends on the diameter of the monopile following:

$$D_{TP} = D_{MP} + 2(t_{TP} + t_{grout})$$

Where

D_{TP} = Diameter of transition piece
D_{MP} = Diameter of monopile
t_{TP} = Wall thickness of transition piece
t_{grout} = Thickness of grout connection

The grout thickness is taken as 50 mm. A generally accepted rule for grout length is $L_{grout}/D_{grout} \approx 1.5$, where L_{grout} is the grout length and D_{grout} is diameter of the grout connection [1] [2]. D_{grout} is taken equal to the diameter of the monopile, D_{MP}.

The wall thickness of the monopile and transition piece is initially determined by taking a fixed ratio of 1:80 to the diameter of the monopile. The wall thickness is taken constant over the entire length of monopile and transition piece. The wall thickness will be optimized with respect to buckling in a later stage.

To incorporate the tower into the design of the support structure a scaling method has been adopted to which the original geometry of the Vestas V90 tower is subjected. The diameter of the tower is scaled such that the diameter at the tower top is equal to 2.5 m, while the diameter at the tower base is set equal to the diameter of the transition piece. The wall thickness is scaled by the same factor. Furthermore the length of the tower is scaled to fit the previously determined hub height.

Determining extreme loads

The maximum wind load on the turbine is approximated by determining the thrust on the rotor at the rated wind speed and taking increased load due to occurrence of a gust into account by multiplying the thrust by 1.5. At the rated wind speed the turbine reaches rated power. If the wind speed increases, the blade pitch will be adjusted to maintain constant rotor speed and thereby constant power output. Although the power output remains constant the thrust on the rotor drops. However, if a gust occurs at the rated wind speed, the control system will not be able to react instantaneously and the blades will not pitch immediately. This causes a temporary increase of the thrust by 50%. This generally gives a reliable estimate of the maximum wind load that can be expected to act on the turbine [3]. The thrust on the rotor as a function of wind speed is calculated using the matlab Based simulation tool RECAL [4]. This graph is shown in Figure 6. The maximum load on the turbine at the rated wind speed including gust is 584 kN.

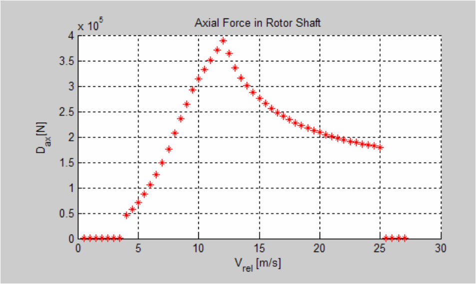


Figure 6: *Axial force in rotor shaft as a function of wind speed.*

Linear airy wave theory with Wheeler stretching is used. Although non-linear wave theories describe the waves better for extreme waves, Airy waves give a reasonable approximation. Its simplicity allows for quick evaluation of the wave loads on a monopile support structure. No marine growth has been taken into account. The hydrodynamic coefficient for drag C_D is equal to 1 while the inertia coefficient C_M is equal to 2.

Table 5 lists the basic load cases which are to be assessed at least [1] [2].

Table 5: *Basic load cases*

Load Combination	Water Level	Wind	Ice	Waves	Current
1	50 years	50 years		5 years	5 years
2	50 years	5 years		50 years	5 years
3	50 years	5 years		5 years	50 years
4	MWL	5 years	50 years	5 years	5 years
5	MWL	50 years	50 years		5 years

However, as surface ice is not expected to occur at the selected location, the 5th load case can be dropped and the ice load is disregarded in the 4th load case. A load factor of 1.35 is to be adopted for environmental loads [1] [2]. Furthermore wind and waves will be assumed to come from the same direction.

Determining penetration depth

The penetration depth must be sufficient to provide both axial and lateral stability. For a monopile support structure, the lateral stability is generally governing. Standards for offshore wind turbine design give no specific instructions for determining the penetration depth. The pile-soil interaction is modelled in Ansys using non-linear soil springs in the form of *p-y* curves. The penetration depth is determined by assuming an initial pile toe level of $9*D_{MP}$. The extreme loads are applied to the structure and the horizontal displacement at the mudline is determined. Subsequently, the pile is shortened by 2 m and the horizontal displacement is determined again. This process is repeated until the mudline deflection for the subsequent step exceeds the deflection of the current step by more than 5%. An additional requirement is that there should be a near vertical tangent and that the horizontal displacement at the mudline does not exceed 0.2 m. This procedure is illustrated for a water depth of 30 m in Figure 7. The final penetration depth is indicated by the red circle. The results for all water depths can be viewed in table 6.

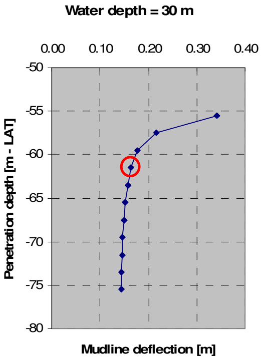


Figure 7: *Determination of penetration depth for a design for 30 m water depth.*

Table 6: *Penetration depth as a function of water depth*

Water depth [m]	*Overturning moment* [MNm]	*Base shear* [MN]	*Penetration depth* [m + LAT]
20	126.8	4.2	-48.00
25	182.6	6.0	-56.00
30	220.9	6.6	-61.50
35	255.7	7.0	-67.00
40	302.7	7.8	-71.50
45	354.3	8.5	-80.00
50	432.0	9.8	-88.00

Normally, the axial stability of the pile foundation should be verified. However, as the lateral stability is generally governing for monopile support structures for offshore wind turbines, this verification will not be given here.

Performing buckling checks

The support structure is to be checked for overall and local buckling [2]. To this end the extreme loads are applied to the Ansys model. The mudline bending moment and normal forces at the mudline are recorded. These loads serve as input for the buckling check. The check will be performed for the section at the mudline only. If the unity check exceeds 1.0 the wall thickness must be increased and the check must be repeated. If the unity check is below 1.0 the wall thickness is reduced until a value as close as possible to 1.0 is reached.

Bar buckling

When a structural element is subjected to an axial compression force and a bending moment, the proof of buckling strength is to be carried out with using the following formula:

$$\frac{N_d}{\kappa N_p} + \frac{\beta_m M_d}{M_p} + \Delta n \leq 1.0$$

In this equation:

N_d = design axial compression force [N]
M_d = design bending moment [Nm]
κ = reduction factor for flexural buckling [-]
β_m = moment coefficient [-]
N_p = plastic compression resistance [N]
M_p = plastic resistance moment [Nm]

N_d and M_d are the factored loads for the element under consideration. N_p, M_p, κ and β_m have been determined following the Germanischer Lloyd guidelines [5]

Table 7: *Results of overall buckling check*

Water depth [m]	Wall thickness [mm]	Unity Check N [-]	Unity Check M [-]	delta n [-]	Unity Check Total [-]	Failure [-]
20	50	0.070	0.666	0.1	0.836	No
25	54	0.073	0.716	0.1	0.888	No
30	57	0.070	0.642	0.1	0.812	No
35	60	0.068	0.676	0.1	0.843	No
40	64	0.063	0.686	0.1	0.849	No
45	69	0.059	0.609	0.1	0.768	No
50	75	0.053	0.599	0.1	0.752	No

Table 7 shows that overall buckling will not occur at the mudline for any of the support structure designs for the given wall thickness.

Shell buckling

For the buckling check of long unstiffened cylindrical shells under combined axial compressive loading and circumferential stresses due to external pressure, the following check must be satisfied:

$$\left(\frac{\sigma_x}{\sigma_{xu}}\right)^{1.25} + \left(\frac{\sigma_\varphi}{\sigma_{\varphi u}}\right)^{1.25} \leq 1.0$$

In this equation:

σ_x = axial compressive stress [N/m^2]
σ_{xu} = ultimate buckling stress for axial compressive stress [N/m^2]
σ_φ = circumferential stress due to external pressure [N/m^2]
$\sigma_{\varphi u}$ = ultimate buckling stress for circumferential stress [N/m^2]

σ_φ and $\sigma_{\varphi u}$ have been determined following the Germanischer Lloyd guidelines [5]. It is assumed that the water level within the pile is equal to Mean Sea Level (0.8 m + LAT) and the sea level is equal to the still water level during storm conditions at High Astronomical Tide + storm surge (3.6 m + LAT).

Table 8: *Results of shell buckling check*

Water depth [m]	Wall thickness [mm]	Unity Check σ_x [-]	Unity Check σ_φ [-]	Unity Check Total [-]	Failure [-]
20	50	0.939	0.0470	0.946	No
25	54	**1.006**	0.0502	**1.031**	**Yes**
30	57	0.905	0.0531	0.909	No
35	60	0.950	0.0557	0.966	No
40	64	0.964	0.0587	0.985	No
45	69	0.857	0.0593	0.853	No
50	75	0.843	0.0616	0.839	No

Failure will occur at the section at the mudline for the design made for 25 m water depth. The unity checks for shell buckling at the mudline are passed for all other water depths. The wall thickness can subsequently be optimized in order to attain a value that is as close to 1.0 as possible. The resulting wall thickness and unity checks for overall and shell buckling are given in table 9. It can be seen that the final values are reasonably close to the initial values for the wall thickness, in particular for the shallower

Table 9: *Results of buckling checks after optimizing the wall thickness*

Water depth [m]	Wall thickness [mm]	Unity Check overall buckling [-]	Unity Check shell buckling [-]	Failure [-]
20	49	0.850	0.977	No
25	56	0.861	0.974	No
30	54	0.851	0.990	No
35	59	0.856	0.992	No
40	64	0.849	0.985	No
45	64	0.819	0.963	No
50	68	0.818	0.983	No

4 RESULTS

In this section the results will be presented graphically and are briefly discussed. The data to be presented are the diameter and the wall thickness of the monopile for each of the designs for the different water depths as well as the overturning moment and total mass of the support structure as a function of the water depth.

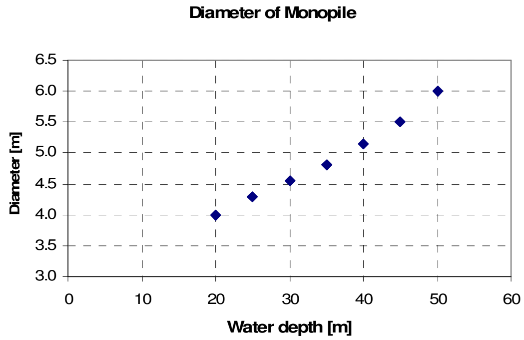


Figure 8: *Diameter of monopile as a function of water depth*

Figure 8 shows the diameter of the support structure as a function of the water depth. The data points show a quadratic trend. However, the first two points do not agree with the trend. This is due to the difference in hub height of the first two designs with respect to the other designs. This difference is due to the fact that the hub height is dependent on the platform level which is in turn dependent on the depth delimited wave height. For water depths of 20 and 25 m the breaking wave height H_B is smaller than the 50 year maximum wave height $H_{max,50}$. Therefore the hub height is lower for the first two designs than for the other designs.

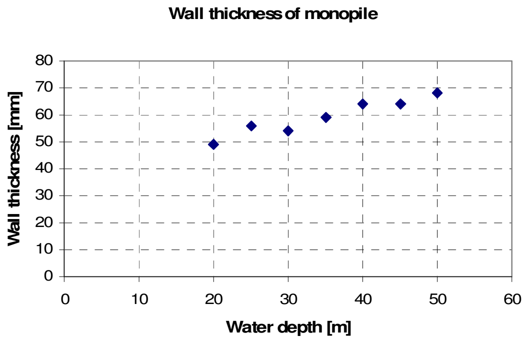


Figure 9: *Wall thickness of monopile per water depth*

The graph in Figure 9 shows the wall thickness per water depth. The data points show a roughly linear trend. Again it should be kept in mind that the first to data points will deviate from this trend due to the different hub height.

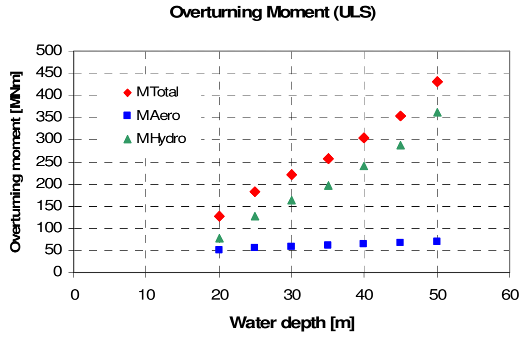


Figure 10: *Overturning moment as a function of water depth*

Figure 10 shows the total overturning moment at the mudline as a function of the water depth and the contributions to the overturning moment of the wind loads and wave loads. The overturning moment due to wind loads, represented by the blue squares, show only a small increase with increasing water depth, whereas the overturning moment due to wave load, indicated by the green triangles, increases severely for larger water depths. This is due to the fact that with increasing water depth the length of the structure exposed to hydrodynamic loads becomes larger. Furthermore, the hydrodynamic loads are related to the diameter of the support structure, which increases rapidly for increasing depths.

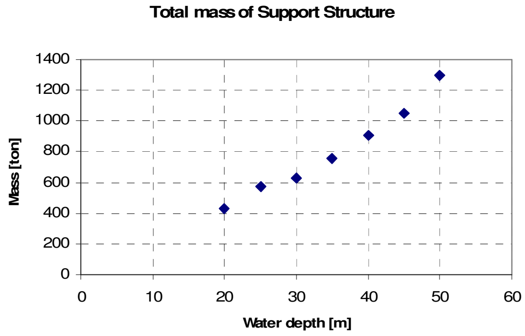


Figure 11: *Total mass of support structure as a function of water depth.*

Finally, the total mass of the structure is presented in figure 11. This figure includes the monopile, the transition piece and the tower masses, but excludes the mass of the turbine itself. A severe increase in the total mass of the support structure is evident as the water depth increases.

CONCLUSIONS & OUTLOOK

After preparing simplified designs for seven different water depths and performing a buckling check of the section at the mudline for each of the designs, the following conclusions can be drawn.

- The mass of the support structure increases dramatically with increasing water depth.
- The overturning moment for deeper water in this study is dominated by the hydrodynamic loads
- A fixed ratio of the wall thickness and the diameter of the monopile of 1:80 is a good initial estimate with respect to buckling at the sections near the mudline.

In retrospect, not choosing the same hub height for all water depths made it difficult to interpret the data for the designs for the first two water depths. In similar future studies it will be beneficial to maintain the same hub height for all water depths. In this study the buckling checks have only been performed for a single section at the mudline, in a future study all sections should be considered. Furthermore, this study has not taken fatigue into account other than choosing a suitable natural frequency. As fatigue may be the dominant phenomenon with regard to the determination of the wall thickness it will be addressed in a future study. Another aspect that should be taken into account in a future study is the fact that for deeper waters generally larger turbines are used.

REFERENCES

1. Det Norske Veritas. Offshore Standard DNV-OS-J101, Design of Offshore Wind Turbine Structures. 2004
2. Germanischer Lloyd. Rules and Regulations; IV Industrial Servces; 2 Guidelines for the Certification of Offshore Wind Turbines, Final draft. 2004
3. Germanischer Lloyd. Rules and Regulations; IV Non-Marine Technology; 2 Offshore Wind Energy Convertors. 1999
4. Storm R. Re-Use of Obsolete Jacket Structures for Carrying an Offshore Wind Turbine. MSc thesis report. Delft University of Technology. 2004.
5. Cerda Salzmann, DJ. Dynamic Response Calculations of Offshore Wind Turbine Support Structures. MSc thesis report. Delft University of Technology. 2004