This paper is accepted for publication in the Journal of Ocean Engineering

Simplified load estimation and sizing of suction anchors for spar buoy type floating offshore wind turbines

Laszlo Arany¹, S. Bhattacharya²

¹ Design Engineer, Atkins (UK)

² Chair Professor in Geomechanics, University of Surrey (UK)

Abstract:

Floating offshore wind turbines are complex dynamic structures, and detailed analysis of their loads require coupled aero-servo-hydro-elasto-dynamic simulations. However, time domain approach used for such analysis is slow, computationally expensive and requires detailed data about the wind turbine. Therefore, simplified approaches are necessary for feasibility studies, front-end engineering design (FEED) and the early phases of detailed design. This paper aims to provide a methodology with which the designer of the anchors can easily and quickly assess the expected ultimate loads on the foundations. For this purpose, a combination of a quasi-static wind load analysis and Morison's equation for wave load estimation using Airy waves is employed. Dynamic amplification is also considered and design load cases are established for ultimate limit state (ULS) design. A simple procedure is also presented for sizing suction caisson anchors. All steps are demonstrated through an example problem and the Hywind case study is considered for such purpose.

Keywords: Floating Wind Turbines, Anchor lines, ULS Loads, Hywind wind farm

1.0 Introduction:

Offshore wind turbines are becoming significant contributors to the energy mix of many European countries, including the UK, the Netherlands, Germany and Belgium. However, the vast majority of the currently installed capacity worldwide is in shallow waters, particularly the North Sea, Irish Sea and Baltic Sea. The water depth for most currently operational wind farms is below 30-35m. Commercial wind turbines are almost exclusively bottom fixed structures, with the majority of them installed on monopile foundations.

Most of the wind resource worldwide, however, is found in deeper waters, including significant portions of the coasts of the US, Japan, China, Norway and the Mediterranean (Henderson et al 2002; European Wind Energy Association 2013; Ho et al 2016). Floating offshore wind turbines (FOWTs) are considered the best solution for harvesting wind energy from deep water sites where bottom fixed turbines are uneconomical (Myhr et al 2014). Analysis of loads and motions of FOWTs is a challenging task, and typically requires a coupled aero-servo-hydro-dynamic analysis. Furthermore, anchor design requires incorporating soil-structure interaction (SSI) in the analysis.

It is important to have a simplified methodology for estimating the loads on the anchors in order to generate conceptual anchor designs for feasibility studies and the early phases of design. This paper aims to provide a simplified approach for finding an upper bound limit for the expected loads on the floating offshore wind turbine structure. These loads may be transferred to the anchor through different load paths for different mooring and anchor types (Randolph and Gourvenec 2011). The load estimation methodology presented in the paper is applicable for most combinations of mooring systems and anchors, however, the anchor sizing example presented considers catenary moorings and suction caisson anchors.

As opposed to offshore oil and gas structures where vertical and horizontal loads dominate the loading, the dominant load for bottom fixed offshore wind turbines is the overturning moment. These moments usually form the design basis for both Ultimate Limit State (ULS) and Serviceability Limit State (SLS) requirements. However, in the case of floating wind turbines, the righting moment which acts against the overturning moment is provided by the floating platform (e.g. a ballast system for a spar supported wind turbine). Therefore, designing against the overturning moment is the task of the naval architect (the designer of the floating platform) and is of little concern to the designer of the foundation (anchor).

The main loads transferred to the anchoring system are the horizontal and vertical forces, see Figure 1. The horizontal force is caused by the combination of

- thrust force on the rotor due to wind - F_u ,
- drag force on the wind turbine tower and the platform sections above mean sea level - F_{DT} ,
- wave load on the spar buoy - F_w ,
- current load on the spar buoy - F_C ,
- rotational frequency loads such as mass and aerodynamic imbalance loads (1P loads) - F_{1P} ,
- blade passage frequency loads (3P loads) - F_{3P} .

It has been shown by Arany et al. (2015, 2017) that the wind load on the rotor and the wave load on the support structure dominates the magnitude of loading on the foundation. Furthermore, the 1P, 3P, current and tower drag loads are less significant. It should be noted, however, that current loads are

significant for estimating the motions of the platform. Simplified calculation methods are derived below to obtain the ultimate load on the anchor of a spar supported floating offshore wind turbine. An example of an offshore wind turbine supported on a floating spar is then considered based on the Hywind floating platform, currently being built in Scotland (Statoil 2015). Finally, a simplified anchor sizing procedure is presented, which is demonstrated to produce conservative upper bound estimates for the required suction caisson with similar values to those found in the environmental statement of the Hywind floating wind park (Statoil 2015).

2.0 Methodology

In order to analyse the Ultimate Limit State (ULS) load on the Floating Offshore Wind Turbine (FOWT) anchor, the following ultimate load scenarios are defined, using the terminology of DNVGL-ST-0437 (DNVGL 2017):

- (E-1) the combination of the 50-year extreme wind speed (with the turbine shut down) and the maximum wave load due to the 50-year extreme wave height, or
- (E-2) the combination of the maximum wind load due to Extreme Operating Gust (EOG) at rated wind speed and the 1-year extreme wave height.

It is not necessary to consider the scenario with the maximum wave height (due to the 50-year extreme wave height) and the maximum wave load (due to EOG at rated wind speed) together as the probability of both occurring together is negligible for the intended design life of 25 years. This is because the maximum wind load occurs when the wind speed is around the rated wind speed and the turbine is operational, while the maximum wave load occurs in a 50-year storm when the turbine is shut down due to the high wind speed. The thrust load on the shutdown turbine is significantly reduced compared to the peak thrust force around the rated wind speed and an example is shown in Figure 2.

A further complication in the load calculation of FOWTs compared to bottom-fixed structures is the range of allowed motions of the floater itself. Motions in six degrees of freedom (surge (x), sway (y), heave (z) displacements and the pitch (y), roll (x) and yaw (z) rotations) have to be considered for floating structures. An important difference between bottom fixed and floating structures is the allowed roll or pitch angle (typically called tilt for bottom fixed structures). DNV-JS-101 (DNV 2014) suggests 0.5° total allowed tilt for bottom fixed structures including accumulated rotation, while DNV-JS-103 (DNV 2013) permits 7° of pitch motion for FOWTs. The pitch motion of the structure introduces a relative velocity component in the wind speed experienced by the rotor, and therefore special control algorithms are required to avoid positive feedback of the motion (Nielsen et al 2006; Jonkman 2007).

The maximum load is assumed to be the sum of the wind load F_u , the drag F_D and inertia F_I components of the wave load F_w , the wind drag on the superstructure (structural components above still water level) F_{DT} , and the current load on the floating platform F_C .

$$F_{total} = F_u + F_I + F_D + F_{DT} + F_C \quad (1)$$

The loads shown here are calculated as loads at the floater padeye where the mooring lines are connected to the floater. This load is transferred through the mooring line to the anchor. Based on the mooring and anchor type, various load paths are possible. The main mooring types are

- (a) catenary mooring (typical for spar platforms and semi-submersible designs),
- (b) taut line mooring (currently mainly used in very deep water in offshore oil and gas applications),
- (c) vertical mooring (typical for tension leg platforms).

The most important anchor types are

- (1) gravity anchors (such as box anchors and grillage and berm anchors),
- (2) embedded anchors (such as suction caisson, pile and drag anchors).

Both embedded and gravity anchors have been used in different floating offshore wind turbine concepts (Principle Power 2010; edp 2012; George 2014; Myhr et al 2014). For different mooring line and anchor combinations, the spar padeye load may be reduced by:

- the weight of the suspended section of the mooring line - W ,
- soil friction on the horizontal mooring line sections on the seabed (typical for catenary mooring lines) - F_f , and
- soil reaction forces acting on the inverse catenary forming at an embedded anchor between the anchor padeye and the seabed - F_r .

This paper focuses on spar supported floating offshore wind turbines with catenary mooring and suction caisson anchors (using the example of the Hywind concept). Therefore, all three effects act to reduce the anchor padeye loading in normal operating conditions. For this structure type, the restoring (or righting) moment M_r necessary to resist the overturning moment is provided by the ballast system. These are shown in Figure 1.

An upper bound estimate for the ultimate load on the anchor of a spar supported FOWT can be obtained by taking the configuration where the mooring line is completely stretched and there is no part of it lying on the seabed. This is very similar to the configuration of a single taut mooring line. In this case the load is transferred directly to the anchor without the effect of soil friction on a horizontal section of the mooring line. Furthermore, in this configuration the angle of the mooring line at the seabed is also maximal, which impacts the inverse catenary shape at the anchor. This configuration is also shown in Figure 1.

Figure 1. Normal operating conditions and stretched mooring line configurations with loads and reaction forces. Also shown are anchor loads (vertical and horizontal), mooring segments on the seabed, and inverse catenary at anchor.

In this paper, the following methodology will be followed:

- Step 1:** Calculate wind load at spar padeye for both environmental load cases E-1 and E-2.
- Step 2:** Calculate wave load at spar padeye for both environmental load cases E-1 and E-2.
- Step 3:** Determine ULS load scenario as the maximum of load cases E-1 and E-2.
- Step 4:** Calculate the load transferred to the anchor.

The tasks are detailed in the following sections.

2.1 Horizontal load estimation

Current design procedure

Floating offshore wind turbine design is currently carried out following the standard DNV-OS-J103 (DNV, 2013). The standard allows for

- design by partial safety factor method,
- design assisted by testing, and
- probability-based design.

Most commonly the partial safety factor approach is chosen, which uses appropriate material factors and load factors coupling the motions of the floating platform and the wind turbine generator (WTG), that is, the rotor-nacelle-tower assembly. In this approach, the response of the platform is often determined using hydrodynamic analyses with the assumption that the WTG is a rigid a body. The obtained responses are then typically fed into a multi-body dynamics software package (e.g. DNVGL Bladed) and are used as boundary conditions at the bottom of the tower in aero-servo-elastic simulations.

The approach presented here provides an upper bound estimate for the anchor loads for preliminary and FEED design stages, using only basic data which are available at the early design phases and circumventing the need for time consuming and computationally expensive numerical analyses.

2.1.1 Wind load on the rotor (thrust)

From the point of view of wind load analysis, it is possible to employ a quasi-static or a coupled dynamic analysis approach. In practice, this means that the loads can be analysed such that the platform motion is neglected while calculating the wind load or by incorporating the platform motion in the load calculation. In this respect, it is important to take into account the typical natural periods of pitch and surge (or roll and sway) vibrations. Generally, FOWTs are designed such that the natural periods are above the typical wave periods. Most of the energy in a wave spectrum is in the wave period range between 3 and 25 seconds (or equivalently between 0.04 and 0.333Hz). The natural vibration periods of floating platforms are typically close to the highest natural periods in this range. These values are also well above the turbine's rotational frequency (1P), which is typically taken as the time constant of the blade pitch control. This means that if the changes in wind speed occur slowly, then the pitch control follows this change and therefore it does not register as a "sudden" change which would cause high dynamic wind load. Using this assumption, a quasi-static analysis of the wind load is possible. The thrust curve of a typical wind turbine is shown in Figure 2, which can be used to calculate the mean thrust force on the turbine.

Figure 2. Thrust coefficient as a function of mean wind speed for a typical wind turbine.

A simplified way to calculate the quasi-static approximation of the wind load is assuming that the wind speed is the sum of a mean wind speed component and a turbulent wind component. The maximum wind load acts when the wind turbine is operating at the rated wind speed U_R (or more precisely, slightly below U_R) where the thrust curve reaches its maximum (see Figure 2, where the rated wind speed is 14m/s). The maximum wind load is then given by the scenario when the wind turbine is operating at the rated wind speed and the 50-year extreme operating gust (EOG) with wind speed magnitude u_{EOG} hits the rotor, that is

$$F_{u,EOG} = \frac{1}{2} \rho_a A_R C_T (U_R + u_{EOG})^2 \quad (1)$$

where ρ_a is the density of air, $A_R = D^2 \pi / 4$ is the rotor swept area, D is the rotor diameter, and C_T is the thrust coefficient obtained from

$$C_T = \frac{3.5(2U_R + 3.5)}{U_R^2} \quad (2)$$

following Frohboese & Schmuck (2010). The extreme operating gust speed u_{EOG} is obtained from the formulation in DNV (2014) based on the long term distribution of 10-minutes mean wind speeds. This is expressed with the Weibull distribution, written in the cumulative distribution function (CDF) form as

$$\Phi(\lambda, \kappa) = 1 - e^{-\left(\frac{U}{\lambda}\right)^\kappa} \quad (3)$$

from which the distribution of 1-year wind speeds is given using the fact that

$$1 [\text{year}] = 365.25 \cdot 24 \cdot 6 = 52596 [10 - \text{minutes intervals}] \quad (4)$$

as

$$\Phi_{1yr}(\lambda, s) = U(\lambda, s)^{52596} \quad (5)$$

The 50-year extreme wind speed is then expressed as

$$U_{10,50yr} = \lambda \left[-\ln \left(1 - 0.98^{\frac{1}{52596}} \right) \right]^{\frac{1}{\kappa}} \quad (6)$$

With these the 50-year extreme operating gust speed at the rated wind is expressed as

$$u_{EOG} = \min \left\{ 1.35(U_{10,1-year} - U_R); \frac{3.3\sigma_{U,c}}{1 + \frac{0.1D}{\lambda_1}} \right\} \quad (7)$$

where $\Lambda_1 = L_k/8.1$ is the turbulence scale parameter, $\sigma_{U,c} = 0.11U_{10,1-year}$ is the characteristic standard deviation of wind speed, $U_{10,1-year} = 0.8U_{10,50-year}$ is the 1-year return period 10-minutes mean wind speed.

During the 50-year extreme wind speed, the turbine is shut down. Therefore, the thrust load reduces to the wind drag force on the tower, blades and hub. If the wind speed is assumed to be constant with height (no wind shear) then the wind drag load in the 50-year extreme wind may be written as

$$F_{u,U50} = \frac{1}{2}\rho_a(3A_B C_{DB} + A_H C_{DH})U_{10,50yr}^2 + F_{DT}(U_{10,50yr}) \quad (8)$$

where A_B, A_H are the face area of a blade and the hub, respectively; C_{DB}, C_{DH} are the drag coefficient of the blade and hub, respectively; F_{DT} is the drag force on the tower (given below); ρ_a and $U_{10,50yr}$ have been defined above.

2.1.2 Tower drag

The tower drag load is caused by the wind exerting a drag force on the wind turbine tower. This load is typically low in magnitude as compared to rotor thrust due to wind and compared to wave loads on the spar buoy. The vertical distribution of wind speed is given by a power law profile as

$$U(z) = \bar{U} \left(\frac{z}{z_{hub}} \right)^\gamma \quad (9)$$

where γ is the wind shear exponent, z is the vertical coordinate measured from water level, z_{hub} is the hub height and $\bar{U}$ is the wind speed at hub height. The drag force on the tower can be written as

$$F_D = \int_0^{z_{hub}} \frac{1}{2}\rho_a D(z)C_{DT}U^2(z)dz \quad (10)$$

where $D(z)$ is the vertical distribution of the tower diameter, C_{DT} is the drag coefficient of the tower circular cross section, ρ_a is the density of air. Carrying out the integration one obtains

$$F_D = \frac{\rho_a C_{DT} \bar{U}^2}{2z_{hub}^{2\gamma}} \int_0^{z_{hub}} \left(D_b - \frac{D_b - D_t}{z_{hub}} z \right) z^{2\gamma} dz = \frac{1}{2}\rho_a C_{DT} \bar{U}^2 z_{hub} \frac{D_b + (2\gamma+1)D_t}{(2\gamma+1)(2\gamma+2)} \quad (11)$$

where D_b is the tower bottom diameter, D_t is the tower diameter at the top. Derivation of the terms given in the equations can be found in Arany et al (2017).

2.1.3 Wave load on the spar buoy

The wave load calculation is typically solved in four steps

- (1) Obtain wave data: in this case maximum wave height H_m and peak wave period T_m are required. The maximum wave height and peak wave period are calculated separately for the 1-year and 50-year extreme wave heights. Typically, the 50-year significant wave height H_{S50} and wave period T_{S50} are known for the site. Then the 1-year significant wave height can be obtained as

$$H_{S1} = 0.8H_{S50} \quad (12)$$

The maximum wave heights can be obtained from the significant wave heights as

$$H_m = H_S \sqrt{\frac{1}{2} \ln N} \quad \text{where} \quad N = \frac{3 \text{ hours}}{T_s} = \frac{10800[s]}{T_s} \quad (13)$$

and N is the number of waves in a 3-hour sea state. The smallest wave period for a given wave height can be obtained following DNV-RP-205 as

$$T = 11.1 \sqrt{\frac{H}{g}} \quad (14)$$

where T is the wave period, H is the wave height and g is the gravitational constant. Equations 10 and 11 can be used both for the 1-year and 50-year extreme wave heights and periods.

- (2) Choose kinematic model: here linear (Airy) wave theory is chosen as floating turbines are expected to be installed in deep water where the linear approximation is more appropriate.

$$\eta = \frac{H_m}{2} \cos\left(-\frac{2\pi t}{T_m}\right) \quad (15)$$

$$u(z, t) = \frac{\pi H_m \cosh(k(S+z))}{T_m \sinh(kS)} \cos\left(-\frac{2\pi t}{T_m}\right) \quad (16)$$

$$\dot{u}(z, t) = \frac{2\pi^2 H_m \cosh(k(S+z))}{T_m^2 \sinh(kS)} \sin\left(-\frac{2\pi t}{T_m}\right) \quad (17)$$

where S is the water depth, k is the wave number, z is the vertical coordinate with $z = 0$ at the still water level, T_m is the peak wave period, H_m is the maximum wave height, η is the surface elevation, $u(z, t)$ and $\dot{u}(z, t)$ are the vertical distribution of the horizontal wave particle velocity and acceleration, respectively.

- (3) Choose a wave load calculation method: in this case Morison's equation is used. The force on an infinitesimal section of the spar is calculated as

$$dF_w(z, t) = dF_D(z, t) + dF_I(z, t) = \frac{1}{2} \rho_w D_P C_D u(z, t) |u(z, t)| + C_m \rho_w A_P \dot{u}(z, t) \quad (18)$$

where ρ_w is the density of water, C_D is the drag coefficient of the spar, C_m is the inertia coefficient of the spar, D_P is the diameter of the spar and $A_P = D_P^2 \pi / 4$.

The total force is obtained by integrating dF_T along the submerged length (draught) of the spar B

$$F_w(t) = \int_{-B}^{\eta} dF_D dz + \int_{-B}^{\eta} dF_I dz = \int_{-B}^{\eta} \frac{1}{2} \rho_w D_P C_D u(z, t) |u(z, t)| dz + \int_{-B}^{\eta} C_m \rho_w A_P \dot{u}(z, t) dz \quad (19)$$

The drag load is highest when the surface elevation is maximal $\eta = H_m/2$, the inertia load is highest when the surface elevation is zero $\eta = 0$. Therefore, the maximum drag and inertia load occur at different time instants, although calculating the maxima separately and summing them to obtain the total wave force is a conservative approach.

$$\begin{aligned} F_{D,max} &= \int_{-\frac{B}{2}}^{\frac{H_m}{2}} \frac{1}{2} \rho_w D_P C_D u(z, 0) |u(z, 0)| dz = \frac{1}{2} \rho_w D_P C_D \frac{\pi^2 H_m^2}{T_m^2 \sinh(kS)^2} \int_{-\frac{B}{2}}^{\frac{H_m}{2}} \cosh(k(S+z))^2 dz = \\ &= \frac{1}{2} \rho_w D_P C_D \frac{\pi^2 H_m^2}{T_m^2 \sinh^2(kS)} P_D \\ P_D &= \frac{1}{8k} \left[e^{2k(S+\frac{H_m}{2})} - e^{-2k(S+\frac{H_m}{2})} - e^{2k(S-B)} + e^{-2k(S-B)} \right] + \frac{H_m}{4} + \frac{B}{2} \end{aligned} \quad (20)$$

$$\begin{aligned} F_{I,max} &= \int_{-B}^0 C_m \rho_w A_P \dot{u}\left(z, \frac{T_m}{4}\right) dz = -2\pi^2 C_m \rho_w A_P \frac{H_m}{T_m^2 \sinh(kS)} \int_{-B}^0 \cosh(k(S+z)) dz = \\ &= 2\pi^2 C_m \rho_w A_P \frac{H_m}{T_m^2 \sinh(kS)} P_I \\ P_I &= \frac{\sinh(kS) - \sinh(k(S-B))}{k} \end{aligned} \quad (21)$$

The wave number k can be determined from the dispersion relation

$$\omega^2 = gk \tanh kS \quad (22)$$

- (4) Apply dynamic amplification factor based on the natural frequency of the structure f_{surge} and damping ratio ξ_{surge} for the surge mode of vibration, as well as the peak wave frequency.

$$DAF_{surge} = \frac{1}{\sqrt{(1-\beta_{surge}^2)^2 + (2\xi_{surge}\beta_{surge})^2}} \quad \beta_{surge} = \frac{f}{f_{surge}} \left(= \frac{\text{excitation frequency}}{\text{natural frequency}} \right) \quad (23)$$

It should be noted here that Response Amplitude Operators (RAOs) are often used in naval architecture instead of dynamic amplification factors. In linear analysis, the RAO expresses the structural response magnitude to a wave of unit height, for various time periods of excitation. The unit of the RAO in e.g. surge is [m/m], that is, motion response in metres per unit wave height (1 metre).

2.1.4 Current load

The simplest way to estimate the load due to currents is to assume a constant velocity v_c profile along the spar length L_p . The force per unit length is then given by

$$dF_c(z, t) = dF_{D,C}(z, t) = \frac{1}{2} \rho_w D_P C_{DP} v_c |v_c| \quad (24)$$

where C_D is the drag coefficient of the spar. The total force is obtained as

$$F_c = \int_{-B}^0 dF_D dz = \frac{1}{2} \rho_w D_P C_{DP} v_c^2 B \quad (25)$$

The constant velocity of the current is the sum of the tidal current velocity, the storm surge and the wind induced current. The wind induced component may be taken as 1.6-3.3% of the mean wind speed following DNVGL-ST-0437 (DNV GL 2016). Typically, maximum 50-year current speeds are in the order of 1-5m/s and in this paper $v_c \approx 0.05\bar{U} \approx 2[m/s]$ is used as a conservative estimate.

3.0 Worked example

A worked example presented here basically emulates the Hywind Pilot Park close to Peterhead in Scotland. Five 6MW turbines are planned on a spar buoy platform, utilizing suction caisson anchors. Provisional data (Statoil 2015) is presented in Table 1. The wind and wave loads are calculated following Section 2.

3.1 Wind Load

Weibull distribution for long term is given in Equation 3 with parameters in Table 1, which gives the 50-year and 1 year return period 10-minutes mean wind speeds as

$$U_{10,50yr} = 35.7 \left[\frac{m}{s} \right] \quad U_{10,1yr} = 28.6 \left[\frac{m}{s} \right] \quad (30)$$

The Extreme Operating Gust (EOG) wind speed is calculated as

$$u_{EOG} = 7.6 \left[\frac{m}{s} \right] \quad (31)$$

and wind load due to the EOG at the rated wind speed is

$$F_{u,EOG} = 2.9[MN] \quad (32)$$

The wind load on the shut down structure in the 50-year extreme wind speed is

$$F_{u,U50} = 0.72[MN] \quad (33)$$

of which the the tower drag load and rotor drag load components are

$$F_{DT} = 0.18[MN] \quad F_{DT,50yr} = 0.54[MN] \quad (34)$$

The drag force on the tower at the rated wind speed is $F_{DT,U_R} = 0.02[MN]$.

Table 1. Parameters of the floating offshore wind turbine and the site.

Parameter	Symbol	Value	Unit
Turbine parameters			
Rotor diameter	D	154	m
Rated wind speed	U_R	12	m/s
Mass of the rotor-nacelle assembly	m_{RNA}	403	tons
Mass of the tower	m_T	626	tons
Drag coefficient of tower	C_{DT}	0.5	[-]
Tower bottom diameter	D_b	6.5	m
Tower top diameter	D_t	4.1	m
Hub height above sea level	z_{hub}	100	m
Spar and mooring parameters			
Spar diameter	D_S	14.4/9.5*	m
Spar draft (depth below sea level)	B	85	m
Mass of the ballast	m_B	8000	tons
Mass of the spar buoy	m_S	1700-2500	tons
Centre of buoyancy below sea level	z_B	50	m
Mooring radius	r_m	600-1200	m
Unit weight of mooring chains	μ_C	200-550	kg/m
Mass of the mooring cables	m_C	120-660	tons
Wind parameters			
Mean wind speed at the site	κ	1.8	[-]
Weibull distribution shape parameter	λ	8	m/s
Wind profile exponent	γ	1/7	[-]
Integral length scale	L_k	340.2	m
Turbulence Intensity	I_{15}	20	%
Wave parameters			
Water depth	S	95-120	m
Significant wave height	H_S	10	m
Peak wave period	T_S	11.2	s
Density of sea water	ρ_w	1030	kg/m ³
Soil parameters			
Soil type	Medium sand		
Mooring chain friction on sand	μ	0.25	[-]
Internal angle of friction	ϕ'	30	°
Submerged unit weight	γ'	9	kN/m ³
* The diameter of the lower 58m of the spar is 14.4m, then there is a coned section 15m long, and the upper section has a diameter of 9.5m.			

3.2 Wave and Current Loads

These loads are calculated by breaking up the spar into three sections as specified in Table 1. The bottom 58m is modelled with a diameter of 14.4m, the 15m long coned sections is modelled with the average diameter of 11.95m, and the top section with a diameter of 9.5m. This section can be modelled using an equivalent diameters of $D_D = 11.33m$ for drag load calculations, and $D_I = 12.89m$ for inertia load calculations.

The maximum of the drag load for the 50-year Extreme Wave Height is

$$F_{D,max,50yr} = 3.56[MN] \quad (35)$$

and the maximum of the 50-year inertia load is

$$F_{I,max,50yr} = 20.04[MN] \quad (36)$$

The peak loads occur at different time instants, therefore the 50-year extreme wave load is taken as

$$F_{w,50yr} \approx 20.1 [MN] \quad (37)$$

For the 1-year Extreme Wave Height scenario the maximum of the drag load and inertia load are calculated as

$$F_{D,max,1yr} = 2.2[MN] \quad F_{I,max,50yr} = 16.67[MN] \quad (35)$$

The total wave load is then

$$F_{w,1yr} \approx 16.7 [MN] \quad (36)$$

The current load is calculated as

$$F_C = \frac{1}{2} \rho_w D_P C_{DP} v_C^2 B \approx 2.32[MN] \quad (37)$$

3.3 Anchor load combinations

The loads under the combined actions of wind and waves have to be considered. The two combinations of loads (E-1) and (E-2) are calculated as

$$F_{E-1} = 20.1 + 0.72 + 2.32 \approx 23.1 [MN] \quad (38)$$

$$F_{E-2} = 16.7 + 2.9 + 2.32 \approx 21.9[MN] \quad (39)$$

As expected, the wave load dominates, and the scenario with the combination of the 50-year Extreme Wave Height and the 50-year extreme mean wind speed combination produces the ULS load. It should be noted here that this load is conservative for anchor design, as the load that acts on the anchor is reduced by the weight of the suspended section of the mooring line, the friction on the horizontal section (Touch Down Zone) of the mooring line, the soil reaction on the inverse catenary shaped forerunner in the soil and the weight of the forerunner. The vertical load acts on the spar at the instant when the surface elevation at the spar is at its highest point (wave crest), while the horizontal load is dominated by the inertia load, which is highest when the surface elevation is at the mean water level. Therefore, the ultimate load is taken as the horizontal load as calculated above.

3.4 Sizing the anchor

In this section, a simple anchor sizing exercise is carried out assuming a suction caisson anchor. The diameter of the caisson D and the embedment depth L are the two main independent parameters that govern the holding capacity of the caisson for a given soil profile. Formulations for both clayey and sandy soils are given in this section. At the Hywind site, the top layer of the seabed soil is dominated by loose to medium sand. The sub-seabed soil within the embedment range of the anchor is dominantly soft clay with intermittent sand layers. In the worked example, three soil types are considered:

- (1) Clay with constant undrained shear strength with depth, using an average value of $s_u = 30kPa$
- (2) Clay with linearly increasing undrained shear strength with depth, using $s_{u0} = 15kPa$ and $\frac{ds_u}{dz} = 2kPa/m$.
- (3) Soft/medium sand with angle of internal friction of $\phi = 30^\circ$ and effective unit weight of $\gamma' = 9 \left[\frac{kN}{m^3} \right]$

The holding capacity of suction caissons is typically determined in terms of an envelope based on the horizontal and vertical load components at the anchor. Following Randolph and Gourvenec (2011) and Supachawarote et al (2004), the envelope is given as:

$$FP = \left(\frac{H_u}{H_m}\right)^a + \left(\frac{V_u}{V_m}\right)^b < 1 \quad (40)$$

where

$$a = \frac{L}{D} + 0.5 \quad b = \frac{L}{3D} + 4.5 \quad (41)$$

An alternative formulation by Senders and Kay (2002) replaces a and b with $k = 3$. In equation 40, H_m is the horizontal capacity and V_m is the vertical capacity. On the other hand, H_u and V_u are the applied load. FP is the failure criterion and the maximum value can be 1 (limiting condition).

3.4.1 Suction caisson bearing capacity in clay

The horizontal capacity H_m in clay is given following Randolph and Gourvenec (2011) as

$$H_m = LD_e N_p \bar{s}_u \quad (42)$$

where

- L penetration depth of the caisson,
- D_e external caisson diameter,
- N_p lateral bearing capacity factor (shown to depend only slightly on L/D_e in Randolph and Gourvenec (2011)), approximate values are given in Table 2.
- $\bar{s}_u$ average undrained shear strength over the embedded length of the caisson.

Table 2. Lateral bearing capacity factor for clays with various strength profiles.

N_p	Linearly increasing s_u with depth	Uniform s_u with depth
Horizontal translation	~10.5	~10
Horizontal load at mudline	~2.5	~4

The three formulations for the vertical capacity represent three failure modes: (1) presence of passive suction and reverse end bearing, (2) no passive suction, caisson pullout, (3) no passive suction, caisson and soil plug pullout (internal soil plug failure). Figure 3 shows the three failure modes and the formulations are as follows:

$$\begin{aligned} V_{m1} &= \text{submerged weight of the caisson} + \text{external friction} + \text{reverse end bearing} \\ V_{m2} &= \text{submerged weight of the caisson} + \text{external friction} + \text{internal friction} \\ V_{m3} &= \text{submerged weight of the caisson} + \text{external friction} + \text{weight of the soil plug} \end{aligned}$$

Figure 3. Pull-out failure modes of suction caissons.

Using the formulations of Randolph and Gourvenec (2011):

$$V_{m1} = W' + A_{se}\alpha_e\bar{s}_u + N_c s_u A_e \quad (43)$$

$$V_{m2} = W' + A_{se}\alpha_e\bar{s}_u + A_{si}\alpha_i\bar{s}_u \quad (44)$$

$$V_{m3} = W' + A_{se}\alpha_e\bar{s}_u + W'_{plug} \quad (45)$$

where

- A_{se} external shaft surface area $\approx D_e\pi \times L$,
- A_{si} internal shaft surface area $\approx D_i\pi \times L$,
- A_e external cross-sectional area $= D_e^2\pi/4$,
- α_e coefficient of external shaft friction between steel and soil,
- α_i coefficient of internal shaft friction between steel and soil,
- N_c reverse end bearing factor (~ 9),
- s_u representative undrained soil shear strength at caisson tip level,
- $\bar{s}_u$ average undrained soil shear strength over penetrated depth,
- W' submerged caisson weight,
- W'_{plug} effective weight of the soil plug.

3.4.2 Suction caisson bearing capacity in sand

The lateral capacity in sand can be calculated following

$$H_{m,sand} = -LQ_{av} = 0.5A_b N_q \gamma' L^2 \quad (46)$$

where the average soil strength may be determined following Miedema et al (2007) as

$$LQ_{av} = D_e N_q \int_0^L \gamma' z dz = \frac{1}{2} D_e N_q \gamma' L^2 \quad (47)$$

where

- γ' submerged unit weight of the soil,
 - N_q bearing capacity factor, calculated based on DNV Classification Note 30.4 (DNV 1992) as
- $$N_q = e^{\pi \tan \phi} \tan^2 \left(45^\circ + \frac{\phi}{2} \right) \quad (48)$$

with ϕ being the internal angle of friction of the soil.

The vertical capacity in sand accounting for the effects of stress enhancement can be calculated following Houlsby et al (2005a); Houlsby et al (2005b) as

$$V_{m,sand} = W' + \gamma' Z_e^2 y \left(\frac{h}{Z_e} \right) (K \tan \delta)_e (\pi D_e) + \gamma' Z_i^2 y \left(\frac{h}{Z_i} \right) (K \tan \delta)_i (\pi D_i) \quad (49)$$

where

$$y(x) = e^{-x} - 1 + x;$$

$K \tan \delta$ factor that only appears together. K is the effective stress factor used to calculate the horizontal effective stress as a constant times the effective vertical stress ($\sigma_H = K \sigma'_V$), δ is the mobilised angle of friction between the caisson wall and the soil, "e" representing the external and "i" the internal circumference of the caisson;

$Z = D/[4K \tan \delta]$ with "e" and "i" referring to external and internal values, respectively;

γ' submerged unit weight of the soil;

N_q bearing capacity factor as defined above.

3.4.3 Load transfer from the mudline to the anchor

The actual load on the anchor is obtained by taking into account the load reduction on the inverse catenary forming at the anchor. This is important as not only the magnitude but the angle at which the load is applied also changes through the inverse catenary shape at the anchor. The anchor padeye tension T_a and angle θ_a can be determined by simultaneously solving the following two equations following Randolph and Neubecker (1995), see Figure 4 for definition of the terms:

$$\frac{T_a}{2} (\theta_a^2 - \theta_m^2) = z_a Q_{av} \quad (50)$$

$$\frac{T_m}{T_a} = e^{\mu(\theta_a - \theta_m)} \quad (51)$$

where

T_a tension at the anchor padeye,

T_m tension at the mudline,

θ_a angle of the tension at the anchor padeye to horizontal,

θ_m angle of the tension at the mudline to horizontal,

z_a depth of the anchor padeye below mudline,

μ friction coefficient between the forerunner (chain, rope or wire) and the soil,

Q_{av} average soil resistance between the mudline and the padeye.

Figure 4: Loads on the anchor lines

The average soil resistance can be determined for clay as

$$z_a Q_{av} = A_b N_c \int_0^{z_a} s_u(z) dz \quad (52)$$

where

- A_b effective unit bearing area of the forerunner (equals the diameter of the rope or wire, and 2.5-2.6 times the bar diameter for a chain),
- N_c bearing capacity factor (between 9 and 14 based on *DNVGL-RP-E301 Design and installation of fluke anchors (DNV GL 2017)*).
- $s_u(z)$ distribution of the undrained shear strength with depth.

For sand, the average soil resistance is calculated following Miedema et al (2007) as

$$z_a Q_{av} = A_b N_c \int_0^{z_a} \gamma' z dz \quad (53)$$

3.4.4 Required caisson dimensions

The required dimensions of the suction caisson necessary to anchor the floating platform are calculated using the ultimate load and the equations of Sections 3.4.1-3.4.3. The caisson dimensions are determined for the three different soil types given in Section 3.4, including clay with constant undrained shear strength (s_u), clay with linearly increasing s_u and medium dense sand.

The anchor padeye is placed at z_a depth below mudline. This depth is determined based on moment balance such that soil resistance is mobilised due to horizontal translation of the anchor rather than rigid body rotation, see Randolph and Gourvenec (2011) and Figure 5. For sand, where strength increases linearly from zero at the mudline, this depth is $z_a/L = 2/3$. For clay, the value varies between about $z_a = 1/2$ and $z_a = 2/3$.

Figure 5. Failure modes of caissons under horizontal loading.

Several values of the length to diameter ratio L/D are chosen for the analysis, and the required parameters are determined for each. This is carried out by the following procedure:

- (1) An initial value D_0 of the caisson diameter is chosen.
- (2) The embedment depth of the caisson is calculated for the chosen length to diameter ratio.
- (3) The padeye depth is calculated from moment balance as described above as a portion of the embedment length.
- (4) The padeye tension T_a and forerunner angle θ_a are calculated using the soil data, the padeye depth and Equations 50 and 51.
- (5) The horizontal and vertical force components are calculated from T_a and θ_a .
- (6) The wall thickness is simplistically estimated using a wall thickness to diameter ratio of 70. A sensitivity study showed that this has very limited effect as it only affects the internal diameter of the caisson used for calculating the internal friction and the weight of the caisson. By changing the value from 70 to 40 or 100 the required caisson diameter changes by less than 0.1m.
- (7) The weight of the caisson and the horizontal and vertical capacities are calculated.
- (8) Equation 40 is used as failure criterion.
- (9) The process is repeated until the smallest diameter is found which satisfies the failure criterion.

Table 3-5 present the results of this analysis for soft clay with constant s_u , soft clay with linearly increasing s_u and medium dense sand, respectively. The actual Hywind project uses a length to diameter ratio of 3.2 with a caisson diameter of 5m and an embedded length of 16m.

The dimensions determined for the length to diameter ratio of 3.2 are as follows. For soft clay with constant undrained shear strength of $s_u = 30kPa$ the calculated diameter is identical to the actual dimensions ($D = 5m, L = 16m$). For soft clay with undrained shear strength profile given by $s_u(z) = 15kPa + 2 \frac{kPa}{m} \cdot z$, the dimensions are determined as ($D = 5.25m, L = 16.8m$). For the medium sand, the dimensions are higher at ($D = 6.9m, L = 22.1m$).

The dimensions approximate the actual anchors very well, however, applying load factors would increase the required dimensions given by this methodology.

Table 3. Minimum caisson dimensions for various length to diameter ratios – soft clay with constant s_u with depth.

Length-to-diameter ratio of caisson	Lc/De	3.2	2	2.5	3	3.5	4	4.5	5	5.5	6
Minimum required caisson diameter for L/D [m]	D _{min}	5.00	6.35	5.65	5.20	4.80	4.50	4.25	4.05	3.85	3.70
Corresponding length [m]	L _{min}	16.0	12.7	14.1	15.6	16.8	18.0	19.1	20.3	21.2	22.2
Wall thickness [m]	t _w	0.05	0.063	0.056	0.052	0.048	0.045	0.042	0.04	0.038	0.037
Average shear strength of the soil [kPa]	S _{avg}	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0
Shear strength of soil at caisson tip [kPa]	S _u	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0
Submerged weight of the caisson [kN]	W _c	893	1183	1032	927	849	788	740	701	669	642
Submerged weight of the soil plug [kN]	W _p	2697	3422	3054	2785	2579	2416	2283	2174	2082	2005
External shaft friction [kN]	F _e	4878	4889	4881	4878	4879	4883	4891	4903	4918	4938
Internal shaft friction [kN]	F _i	4781	4791	4784	4781	4781	4786	4793	4805	4820	4839
Reverse end bearing [kN]	F _{reb}	4104	6581	5257	4378	3753	3287	2926	2640	2407	2216
Maximum vertical load - Mode I [kN]	V _I	9875	12654	11170	10183	9481	8959	8558	8244	7994	7795
Maximum vertical load - Mode II [kN]	V _{II}	10552	10864	10697	10586	10509	10457	10425	10408	10406	10418
Maximum vertical load - Mode III [kN]	V _{III}	8468	9495	8967	8590	8307	8087	7914	7777	7669	7585
Maximum vertical load capacity [kN]	V _{max}	8468	9495	8967	8590	8307	8087	7914	7777	7669	7585
Maximum horizontal load capacity [kN]	H _{max}	21500	21548	21514	21500	21503	21523	21557	21608	21676	21762
Anchor padeye depth	Z _a	7.98	6.32	7.06	7.73	8.35	8.93	9.48	10.00	10.51	11.00
Padeye location [%]	r _z	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500
Angle at the padeye [deg]	θ _a	14.79	13.11	13.88	14.54	15.13	15.67	16.16	16.62	17.05	17.46
Tension at the padeye (variable) [kN]	T _a	21657	21816	21743	21680	21624	21574	21527	21484	21444	21406
Horizontal load on the anchor [kN]	H _a	20940	21248	21109	20985	20874	20772	20677	20587	20502	20420
Vertical load on the anchor [kN]	V _a	5527	4947	5214	5443	5645	5826	5991	6144	6287	6421
Horizontal load check exponent [-]	a	3.7	2.5	3	3.5	4	4.5	5	5.5	6	6.5
Vertical load check exponent [-]	b	5.57	5.17	5.33	5.50	5.67	5.83	6.00	6.17	6.33	6.50
Horizontal utilisation [-]	V _a /V _{max}	0.653	0.521	0.581	0.634	0.68	0.72	0.757	0.79	0.82	0.847
Vertical utilisation [-]	H _a /H _{max}	0.974	0.986	0.981	0.976	0.971	0.965	0.959	0.953	0.946	0.938

Table 4. Minimum caisson dimensions for various length to diameter ratios – soft clay with linearly increasing s_u with depth.

Length-to-diameter ratio of caisson [-]	Lc/De	3.2	2	2.5	3	3.5	4	4.5	5	5.5	6
Minimum required caisson diameter [m]	D_{min}	5.25	6.90	6.05	5.45	4.95	4.60	4.25	4.00	3.80	3.60
Corresponding length [m]	L_{min}	16.8	13.8	15.1	16.4	17.3	18.4	19.1	20.0	20.9	21.6
Wall thickness [m]	t_w	0.052	0.069	0.06	0.054	0.049	0.046	0.042	0.04	0.038	0.036
Average shear strength of the soil [kPa]	S_{avg}	27.48	25.31	26.29	27.16	27.94	28.67	29.34	29.98	30.59	31.18
Shear strength of soil at caisson tip [kPa]	S_u	39.96	35.62	37.58	39.32	40.89	42.33	43.68	44.96	46.18	47.36
Submerged weight of the caisson [kN]	W_c	1012	1524	1253	1069	937	838	760	698	647	606
Submerged weight of the soil plug [kN]	W_p	3055	4410	3706	3213	2848	2567	2344	2164	2016	1894
External shaft friction [kN]	F_e	4856	4884	4867	4858	4855	4858	4868	4885	4908	4941
Internal shaft friction [kN]	F_i	4759	4786	4770	4761	4758	4761	4771	4787	4810	4842
Reverse end bearing [kN]	F_{reb}	5941	9253	7493	6312	5465	4829	4336	3944	3627	3367
Maximum vertical load - Mode 1 [kN]	V_{m1}	11809	15661	13613	12239	11257	10525	9964	9527	9183	8914
Maximum vertical load - Mode 2 [kN]	V_{m2}	10627	11195	10890	10688	10550	10457	10399	10369	10366	10388
Maximum vertical load - Mode 3 [kN]	V_{m3}	8923	10818	9827	9141	8641	8263	7972	7746	7572	7441
Maximum vertical load capacity [kN]	V_m	8923	10818	9827	9141	8641	8263	7972	7746	7572	7441
Maximum horizontal load capacity [kN]	H_m	21403	21526	21453	21411	21398	21413	21455	21528	21633	21775
Anchor padeye depth [m]	Z_a	9.58	7.81	8.61	9.32	9.96	10.56	11.12	11.65	12.16	12.65
Padeye location [%]	r_z	57.6%	56.8%	57.2%	57.5%	57.7%	57.9%	58.1%	58.3%	58.5%	58.6%
Angle at the padeye [deg]	θ_a	13.90	12.12	12.93	13.64	14.28	14.87	15.41	15.93	16.42	16.90
Tension at the padeye (variable) [kN]	T_a	21740	21910	21833	21765	21705	21649	21598	21549	21503	21458
Horizontal load on the anchor [kN]	H_a	21103	21421	21279	21151	21034	20925	20821	20722	20625	20531
Vertical load on the anchor [kN]	V_a	5224	4601	4885	5133	5353	5554	5740	5914	6079	6237
Horizontal load check exponent [-]	a	3.7	2.5	3	3.5	4	4.5	5	5.5	6	6.5
Vertical load check exponent [-]	b	5.57	5.17	5.33	5.50	5.67	5.83	6.00	6.17	6.33	6.50
Horizontal utilisation [-]	V_a/V_{max}	0.585	0.425	0.497	0.562	0.62	0.672	0.72	0.764	0.803	0.838
Vertical utilisation [-]	H_a/H_{max}	0.986	0.995	0.992	0.988	0.983	0.977	0.97	0.963	0.953	0.943

Table 5. Minimum caisson dimensions for various length to diameter ratios – medium sand.

Length-to-diameter ratio of caisson	L/D [-]	3.2	2	2.5	3	3.5	4	4.5	5	5.5	6
Minimum required caisson diameter [m]	D_{min}	6.90	7.35	7.16	7.15	6.78	6.58	6.38	6.18	5.98	5.79
Corresponding length [m]	L_{min}	22.07	14.70	17.91	21.46	23.73	26.32	28.71	30.90	32.90	34.73
Wall thickness [m]	t_w	0.099	0.105	0.102	0.102	0.097	0.094	0.091	0.088	0.085	0.083
Effective weight of the caisson [kN]	W_c	3361	2652	3002	3529	3468	3594	3659	3674	3650	3594
	$(K \tan \delta)_o$	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00
	$(K \tan \delta)_i$	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00
External shaft friction [kN]	F_e	7343	5512	6407	7668	7634	7987	8194	8280	8266	8174
Internal shaft friction [kN]	F_i	6901	5168	6015	7204	7177	7513	7711	7793	7782	7697
Maximum vertical load capacity [kN]	V_{max}	14247	10682	12425	14875	14814	15503	15909	16077	16052	15875
Maximum horizontal load capacity [kN]	H_{max}	278358	131492	190360	272710	316219	377625	435429	488462	536069	577996
Anchor padeye depth [m]	z_a	14.72	9.80	11.94	14.31	15.82	17.55	19.14	20.60	21.93	23.15
Padeye location [%]	r_z	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67
Angle at the padeye [deg]	θ_a	50.14	32.10	39.77	67.81	54.40	61.25	67.76	73.91	79.70	85.13
Tension at the padeye (variable) [kN]	T_a	18561	20081	19420	16065	18219	17682	17187	16732	16315	15933
Angle at the padeye [deg]	H_a	11897	17012	14926	6068	10605	8504	6505	4638	2918	1353
Tension at the padeye (variable) [kN]	V_a	14247	10670	12424	14875	14814	15503	15909	16077	16052	15875
Horizontal load on the anchor [kN]	a	3.7	2.5	3	3.5	4	4.5	5	5.5	6	6.5
Vertical load on the anchor [kN]	b	5.5667	5.1667	5.3333	5.5	5.6667	5.8333	6	6.1667	6.3333	6.5
Horizontal utilisation [-]	V_a/V_{max}	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Vertical utilisation [-]	H_a/H_{max}	0.04	0.13	0.08	0.02	0.03	0.02	0.01	0.01	0.01	0.00

4.0 Conclusions

This paper presents a very simple methodology to obtain the loads on anchors for a floating offshore wind turbine. The loads derived are calculated for the floating spar – mooring line connection point and then conservatively transferred to the anchor through an inverse catenary. It has been shown that the actual anchor load estimated is conservative due to fact that the load taken by the weight of the mooring line and the forerunner, as well as the friction of the horizontal section (touchdown zone) on the seabed is ignored in the analysis. However, the values obtained here are upper bound estimates for the actual ultimate load expected, and therefore can be considered conservative and may be used for tender design and feasibility studies. The methodology may be further improved and some conservatism relieved by taking a dynamic approach. The example of the first floating turbine (Hywind) is taken to show the application of the formulation whereby the anchor loads are calculated. The values obtained by the proposed methodology are similar to those estimated for the planned floating wind farm. A simple approach is also presented for estimating the minimum size of the caisson for various length to diameter aspect ratios. The estimated foundation size is shown to be reasonably close to the preliminary conservative upper bound estimations for the Hywind floating offshore wind turbine structures. The approach presented can be used as a quick method for conservative conceptual design.

References

- Arany, L., Bhattacharya, S., Macdonald, J., & Hogan, S. J. (2017). Design of monopiles for offshore wind turbines in 10 steps. *Soil Dynamics and Earthquake Engineering*, 92, 126-152
- Arany L, Bhattacharya S, Macdonald J, Hogan SJ (2015) Simplified critical mudline bending moment spectra of offshore wind turbine support structures. *Wind Energy* 18:2171–2197. doi: 10.1002/we.1812
- DNV (2014) Offshore Standard DNV-OS-J101 Design of Offshore Wind Turbine Structures. Høvik, Norway
- DNV (2013) Offshore Standard DNV-OS-J103 - Design of Floating Wind Turbine Structures.
- DNV (1992) Classification Note 30.4 Foundations. 54.
- DNV GL (2016) DNVGL-ST-0437 Loads and site conditions for wind turbines.
- DNV GL (2017) DNVGL-RP-E301 - Design and installation of fluke anchors.
- edp (2012) The WindFloat Project.
- European Wind Energy Association (2013) Deep water - The next step for offshore wind energy.
- Frohboese P, Schmuck C (2010) Thrust coefficients used for estimation of wake effects for fatigue load calculation. *Eur. Wind Energy Conf.* 2010, Warsaw, Pol. pp 1–10
- George J (2014) WindFloat design for different turbine sizes.
- Henderson AR, Leutz R, Fujii T (2002) Potential for Floating Offshore Wind Energy in Japanese Waters. 3:505–512.
- Ho A, Mbistrova A, Corbetta G (2016) The European Offshore Wind Industry: Key Trends and Statistics 2015. Brussels, Belgium
- Houlsby G, Kelly R, Byrne B (2005a) The tensile capacity of suction caissons in sand under rapid loading. *Front Offshore Geotech* 405–410. doi: 10.1201/NOE0415390637.ch40
- Houlsby GT, Ibsen LB, Byrne BW (2005b) Suction caissons for wind turbines. In: Gourvenec, Cassidy (eds) *Front. Offshore Geotech. ISFOG 2005 - Gourvenec Cassidy*. Taylor & Francis Group, London, UK, pp 75–94
- Jonkman J (2007) Dynamics Modeling and Loads Analysis of an Offshore Floating Wind Turbine Introduction Background — The Big Picture.
- Miedema SA, Lagers GHG, Kerkvliet J (2007) An overview of drag embedded anchor holding capacity for dredging and offshore applications. In: Randall REP (ed) *Proc. 18th World Dredg. Congr.*

- (WODCON XVIII). Newman Printing Company, Lake Buena Vista, FL, USA, pp 245–274
- Myhr A, Bjerkseter C, Agotnes A, Nygaard TA (2014) Levelised cost of energy for offshore floating wind turbines in a life cycle perspective. *Renew Energy* 66:714–728. doi: 10.1016/j.renene.2014.01.017
- Nielsen FG, Hanson TD, Skaare B (2006) Integrated Dynamic Analysis of Floating Offshore Wind Turbines. *Proc 25th Int Conf Offshore Mech Arct Eng*. doi: 10.1115/OMAE2006-92291
- Principle Power I (2010) WindFloat Offshore Wind Power Plant.
- Randolph M, Gourvenec MRS (2011) *Offshore Geotechnical Engineering*. Spon Press (Taylor & Francis), New York, USA
- Senders M, Kay S (2002) Geotechnical suction pile anchor design in deep water soft clays. *Conf. Deep. Risers Mooring Anchorings*, London
- Statoil (2015) Hywind Scotland Pilot Park - Executive Summary of the Environmental Statement.