

REVIEW OF DESIGN PROCEDURES FOR MONOPILE OFFSHORE WIND STRUCTURES

ORLANDO BARBOSA LEITE

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Orientador: Professor Doutor José Miguel de Freitas Castro

Coorientador: Engenheiro Tiago João Fazerres Ferradosa

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DEPARTAMENTO DE ENGENHARIA CIVIL

Tel. +351-22-508 1901

Fax +351-22-508 1446

✉ miec@fe.up.pt

Editado por

FACULDADE DE ENGENHARIA DA UNIVERSIDADE DO PORTO

Rua Dr. Roberto Frias

4200-465 PORTO

Portugal

Tel. +351-22-508 1400

Fax +351-22-508 1440

✉ feup@fe.up.pt

🌐 <http://www.fe.up.pt>

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To my Father, Mother and Brother

A mente que se abre a uma nova ideia jamais voltará ao seu tamanho original

ALBERT EINSTEIN

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RESUMO

O investimento em energias renováveis sobretudo em energia eólica tem vindo a aumentar ao longo dos últimos anos, existindo um forte desenvolvimento das tecnologias implementadas nesta área. Estas técnicas de exploração eólica iniciaram-se em terra com a designação “Onshore” e posteriormente se estenderam para o mar denominando-se por “Offshore”, face à velocidade do vento mais elevada, que permitia obter melhor rentabilidade da energia produzida, associada à vasta área de implantação disponível que não carecia de expropriação, nem estava limitada pelos eventuais índices de urbanização existentes.

O presente trabalho tem como objetivo a realização de um estudo sobre o dimensionamento da fundação mais utilizada em offshore para este tipo de estruturas eólicas, o monopilar.

As ações consideradas no dimensionamento deste tipo de estruturas eólicas implementadas no mar foram, a ação do vento combinada com a ação das ondas mais correntes marítimas. A quantificação destas ações seguiram sobretudo as normas DNV e API, embora recorrendo e referenciando, devidamente o contributo fornecido por outras normas e padrões de dimensionamento.

A presente dissertação apresenta as verificações para um caso de estudo localizado no mar do norte, bem como os fundamentos e cálculos que sustentam os valores de cargas adotados para o respetivo dimensionamento.

PALAVRAS-CHAVE: Offshore, onshore, monopilar, DNV, API.

ABSTRACT

The investment in renewable energy, especially wind energy, has been increasing over the past few years, with a strong development of technologies implemented in this area. These wind exploitation techniques began on land with the designation of "Onshore " and later extended to the sea calling itself as " Offshore " , by the reason of the wind speed being higher, thus assuming more profitability of energy produced. Besides not being dependent on urban planning and on private property compensations.

This dissertation aims to carry out a study of the design of the foundation most frequently used in offshore wind structures, i.e. the monopile.

The actions considered in the design of this type of wind structures implemented at sea were, the wind action combined with the action of the waves and sea currents. The quantification of these actions mainly followed the DNV and API standards, while using and referencing the contribution provided by other design standards. This thesis presents the values obtained for a case study located in the North Sea, as well as the theoretic basis and calculations that were performed in order to obtain the loads and safety verifications for a specific design.

KEYWORDS: Offshore, onshore, monopile, DNV, API.

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NOMENCLATURE

API - American Petroleum Institute

DNV – Det Norske Veritas

DEC - Departamento de Engenharia Civil

FEUP – Faculdade de Engenharia da Universidade do Porto

GL - Germanischer Lloyd

IEC - International Electrotechnical Commission

ISO - International Organization for Standardization

a: Wave amplitude; Axial induction factor

d: Mean water depth; Cross-section internal diameter

e: Pile thickness

f: frequency

f_{nat} : First natural frequency

f_p : Spectral peak frequency

f_{yd} : Yield strength of the steel

g: Acceleration of gravity

k: interaction factor; Wave number

h: Water depth from still water level

h_0 : Reference depth from wind-generated current

m_{top} : Turbine mass

r: Radius of the rotor

u: Horizontal particle velocity

w: Wave angular frequency

x : Reduction factor for the relevant buckling mode

y: Equal to the radius of the cross-section

y: Lateral displacement

A: Exposed area to the breaking wave; Projected area of the member, normal to force direction; Rotor area; Area of the cross-section; Factor for cyclic or static loading conditions

A_{eff} : Effective area of the cross-section

A_{disk} : Swept area of the wind turbine rotor

C_a : Airfoil chord length

Cd: Hydrodynamic drag coefficient

C_D : Aerodynamic drag coefficient

C_L : Aerodynamic lift coefficient; Von Kármán lift coefficient

Cs: Shape coefficient; Slamming coefficient

Cm: Hydrodynamic inertia coefficient

D: Pile diameter; Cross-section external diameter

E: Modulus of elasticity

EI: Tower bending stiffness

H: Wave height

H_s : Significant wave height

I: Moment of inertia

K: Stiffness of the structure

L: Tower length

L_k : Integral length scale parameter

M: Mass of the complete structure

M_{Ed} : Actuating bending moment

M_{el} : Elastic moment

M_{pl} : Plastic moment

N_{cr} : Critical load

N_{Ed} : Actuating axial force

P: Lateral load

$S_{wind,k}$: Characteristic wind load effect.

$S_{wave,k}$: Characteristic wave load effect

S_t : Strouhal number

T: Wave period

T_p : Peak period

U: Wind velocity

U_{cr} : Critical wind velocity

$U_{project}$: Project wind velocity

U_0 : 1-hour mean wind speed at 10 m height
 U_{10} : 10-minute mean wind speed at 10 meters height
 U_c : Current velocity
 V : Acting wind speed on the turbine rotor
 V_{disk} : Wind velocity at airfoil
 V_{Rot} : Linear rotation speed at a blade section
 V_1 : Incident wind speed at the turbine rotor high
 Z : Vertical coordinate from still water level, positive upwards

Greek Letters:

α : Attack angle; Generalised Phillips' constant; Imperfection factor
 γ : Peak-enhancement factor
 γ_{f_1} and γ_{f_2} : Load coefficients for the ULS
 γ_{M0} : Partial safety coefficient
 γ_{M1} : Partial safety coefficient
 Δ_r : Radial length of the blade
 η : Sea surface elevation
 Θ : Phase angle; Pitch angle
 λ : Wave length
 μ : Tower mass per meter
 u_{max} : Maximum particle velocity at still water level
 ξ : Damping viscous coefficient
 ρ : Water density
 ρ : Air density
 σ : Spectral width parameter
 σ_v : Effective vertical stress at the considered depth
 σ_u : Standard deviation of the wind speed
 ν : Kinematic viscosity of seawater
 ν_{wind0} : Wind-generated current at still water level
 ν_{tide0} : Tidal current at still water level
 $\emptyset$: Inflow angle
 Ω : Angular rotation speed of the rotor

1

INTRODUCTION

1.1. PRESENTATION, CONTEXT AND MOTIVATION FOR THE THEME

According to the climate change, the international agreement of Kyoto protocol has an important rule to reduce the emission of greenhouse gases. The objective of this protocol is to reduce the consumption of energy and increase the production of renewable energy worldwide [1].

The world is now conscious that the problems caused by the dependence on oil and increasing carbon emissions must be solved. So, presently we are living in an era of a turnaround in energy policy, where strong political and industrial discussions happen, in order to apply renewable energies as main energy source. However, current non-renewable resources like oil, natural gas, nuclear power and coal are still the primary energy sources of many countries around the world [2]. Nonetheless, the sources' supplies are limited, the burning of fossil fuels is very harmful to the environment, inserting carbon dioxide into the atmosphere and the storage of radioactive waste from nuclear power stations is very dangerous, as an accident can occur, leading to a radioactive contamination. For these reasons, it is necessary to reduce society's dependence on fossil fuels and focus on efficiency and green sustainable energy sources, so that our emissions will not increase so fast or even stabilize or decline.

The most commonly renewable energies used are the hydro, the wave, the solar and the wind energy. All of these renewable energies have advantages and disadvantages, but in recent years, wind energy is the one that has had the most development and investment.

Observing figures 1-1 and 1-2, we can see that the wind energy has increased from 2.4% in 2000 to 14.1% in 2014, and that the natural gas had also increased in 2014. However, fuel oil, nuclear power and coal had a significant decrease in 2014 compared to 2000.

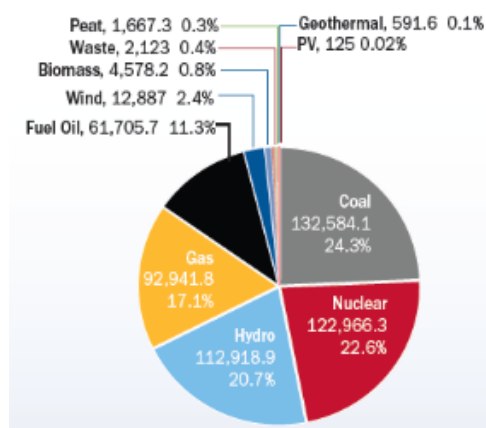


Figure 1- 1: Europe power mix 2000 (MW), [3]

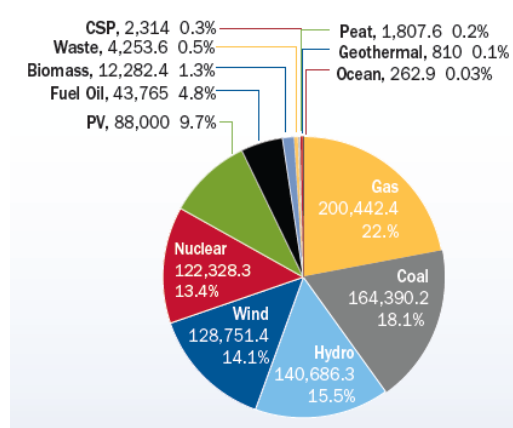


Figure 1- 2: Europe power mix 2014 (MW), [3]

Due to renewable energy, the European Union power sector is moving away from fuel oil and coal, with both technologies continuing to decommission more than they install.

The fossil fuels are now becoming scarcer and increasingly expensive, the world seeks a solution that serves the interests of the economic development and preservation of nature. This concern of the world is leading to a growing demand for renewable energy sources, within an integrated perspective of energy use and environmental preservation.

Thus, wind energy plays an increasingly important role in the global energy landscape as it is one of the energy sources that has less impact on nature.

The growing wind energy demand has led to a great development of the technology of wind turbines and the supporting columns, seeking to greater energy production, a reduction in installation costs and cheaper maintenance. Due to the existence of better wind resources in high sea locations, the wind farms are now moving further away from the shore, into deeper water depths and increasing distances from land. These requires new challenging conditions for wind turbines' projects, that now must be design to resist to some of the most severe conditions existent in the globe, i.e. offshore environment.

In fact, wind power is now seen as one of the most promising renewable energy sources, characterized by a mature technology developed mainly in the EU and the USA. The onshore wind farms technologies are in a higher development stage, so it becomes common to see this turbines as a part of a countryside, in small or big groups. This is evident in countries like Portugal and Spain, but mostly in other European countries like Denmark, Germany and The Netherlands, where the development of the wind energy started earlier [4].

For the objective of obtaining larger amounts of energy, the improving of the equipment related to the exploration of wind resources was needed but not sufficient, so investors started thinking about putting turbines in an offshore environment, due to offshore wind potential being much higher than onshore, as well as there being still many areas close to the coast line with potential to be explored.

Wind power has a great potential because it is a non-polluting and inexhaustible source, and an excellent alternative, in environmental terms, to conventional electricity production. For this reason, onshore and offshore wind energy have a promising future, globally.

1.2. ONSHORE / OFFSHORE WIND TURBINES – GENERAL INFORMATION

For more than a decade, onshore wind turbines have been the world's fastest growing energy source, therefore, the majority of wind farms installed are located onshore. Relative to the construction method, the onshore wind farms has lower costs than offshore. Along the years, the efficiency of wind turbines has increased significantly as wind turbines became bigger, but for onshore wind farms there are some restrictions such as [5]:

- the limitations of land, as dense populations areas with so many buildings compromise the development of onshore wind farms, since so many square meters of land are needed to install this equipment;
- the turbines are limited to size, due to logistic problems during transportation of large wind turbine components. However, in offshore this limitation does not exist, as it is easy to transport large structures in vessels and barges;
- the visual aspect is also a problem, because the wind farms ruins the beauties of a landscape;
- if the wind farms are installed near population centres, the problem with noise pollution is very relevant.



Figure 1- 3: Onshore wind Turbines [6]

On the other hand, wind farms can also be installed offshore, being more profitable than onshore. The development of the offshore wind energy industry has increased significantly over the last 22 years, since the installation of the first offshore wind turbine “Vindeby” in Denmark, in 1991 [7].

The offshore wind farms have a clear number of advantages [8], such as:

- the wind tends to blow with more strength and stability at the sea, which generally increase with distance from the shore, resulting in considerably higher production of energy per unit installed;
- less turbulence from the wind reduces the fatigue loads on the turbine;
- lower wind-shear, which allows the use of shorter towers;
- the size of an offshore wind turbine is not limited, since it is easier to transport very large turbine components by sea;
- the visual and noise impacts of wind turbines can be avoided if the turbines are installed at a sufficient distance from shore;
- large continuous areas are available, so the installations will not occupy and interfere with land uses.

Besides these advantages, there are also several disadvantages to installing wind turbines offshore, related to the capital investment:

- a higher investment is required for this type of structures because of the costs associated with the foundation, transport of all components to the placed area, marine construction equipments, installation and decommissioning;
- these structures have limited access, which raises the operations and maintenance costs during exploration;
- lifecycle of offshore structures is lower than onshore due to the sea environment that is very aggressive and the corrosion that decreases the durability of the equipment. Usually the lifetime is more or less 20 years [9];
- the design of these structures is more complex because of the environmental loads such as the wind, hydrodynamic loads from waves and sea currents;
- the connection to the electrical network is more expensive, and the necessary expansion of the capacity of weak coastal grids is also needed due to the creation of new wind farm parks.



Figure 1- 4: Offshore wind turbines park [10]

There are also other factors not mentioned above which need to be taken into account before the installation of the turbines in a marine environment [4]:

- Water depth is a very important element for the project because of the investment costs;
- Average wind speed is at least more than 7 m / s;
- Study the terrain properties of the ocean floor must be done through a geological analysis;
- Examination of the wind speeds and directions on the local project;
- Investigation of the wave height and period on the respective site location;
- Analyse the distance to shore, and until possible support stations in order minimize the construction and maintenance costs;
- Pre-existing marine ecosystems;
- Migratory route of birds, airplanes and ships;

1.3. ANALYSIS OF ONSHORE / OFFSHORE WIND ENERGY SECTOR

1.3.1. WORLDWIDE

Countries like Japan, South Korea, the United States, Canada, Taiwan and India have shown a huge enthusiasm for developing offshore wind energy in their waters. According to the projections, a total of 80 GW of offshore wind energy could be implemented by 2020 worldwide, with three-quarters of it in Europe [11].

The year of 2014 has crossed all the goals proposed in the last few years, with 51 GW of new wind power capacity connected to line, in comparison with the year of 2013 with 35.6 GW, and the year of 2012 with 45 GW of new capacity installed globally [12].

The largest market for wind energy belongs to China since 2009. During the year of 2014, their industry grew even more, keeping the Chinese market in the top spot. So, with the growth of the Chinese industry, the Asian continent once more led the global markets in 2014, with Europe in the second position, and North America in third position. For 2015, it is expected another good year, European framework legislation and its 2020 targets ensure a degree of stability; the US and Canada are both anticipating strong years; China, Africa and Latin America are expected to continue growing [12].

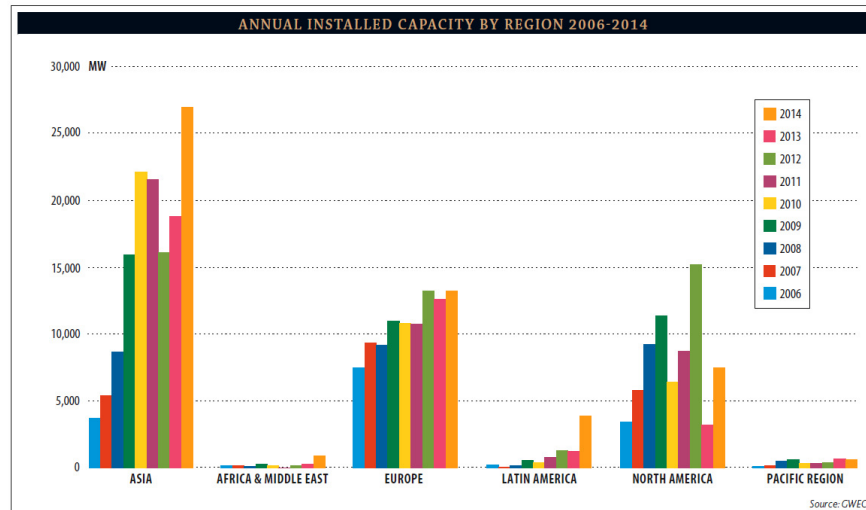


Figure 1- 5: Worldwide annual installed capacity by region [12]

According to Figure 1-7, Asia is the lead in wind energy, due to the installed offshore wind turbines and wind farm parks that already exist and the ones that are still under construction. Through the below figures, it is possible to observe the wind capacity already installed around the world, with the real values in megawatts:

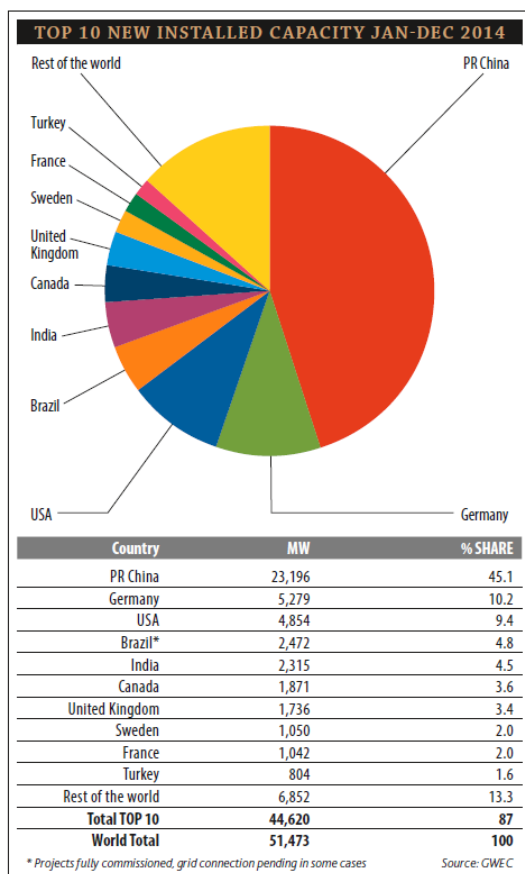


Figure 1- 6: Worldwide installed capacity in 2014, [12]

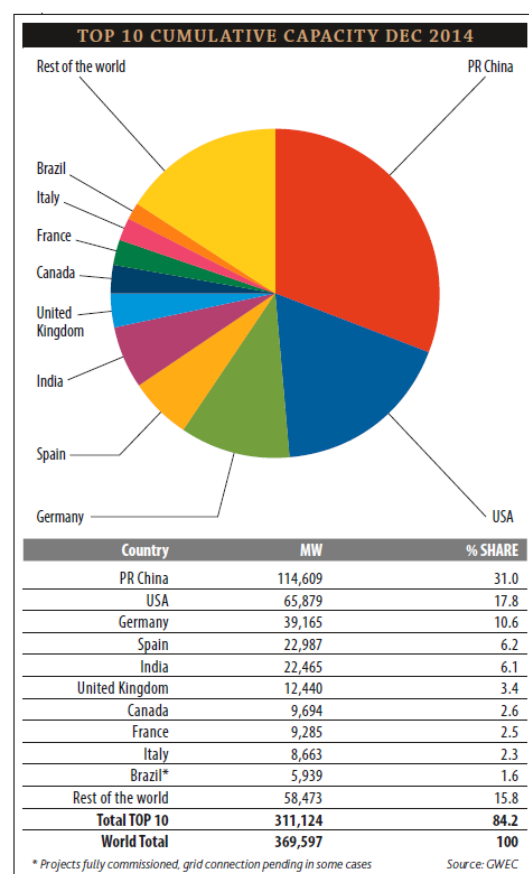


Figure 1- 7: Worldwide cumulative capacity until december of 2014, [12]

Brazil, Canada, Mexico and the US are expected to have a strong 2015. Global installations will be reinforced with new projects arising in Japan, Australia, Pakistan, Kenya, and South Africa [12].

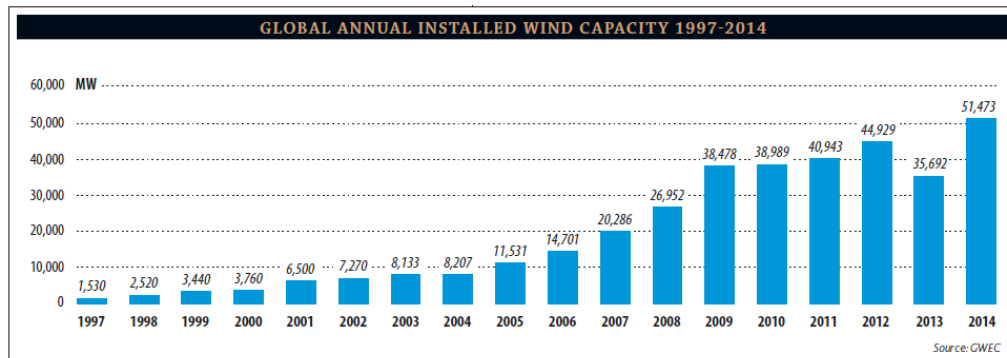


Figure 1- 8: Worldwide annual installed wind capacity since 1997 until 2014 [12]

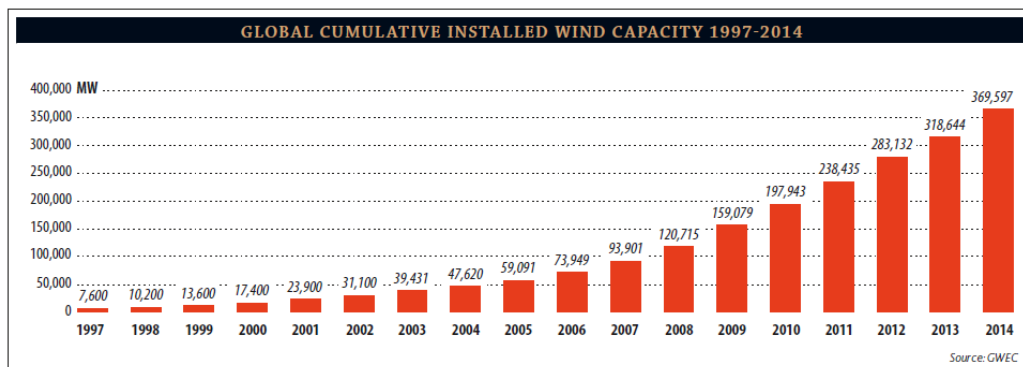


Figure 1- 9: Worldwide cumulative installed wind capacity since 1997 until 2014 [12]

After a slowdown in 2013, the wind industry set a new record for annual installations in 2014. All over the world, 51,473 MW of new wind generating capacity were installed, representing a 44% increase in the annual market, so this demonstrates the recovery of the wind sector in relation to the past few years. Total cumulative installations stood at 369,597 MW at the end of 2014 [12].

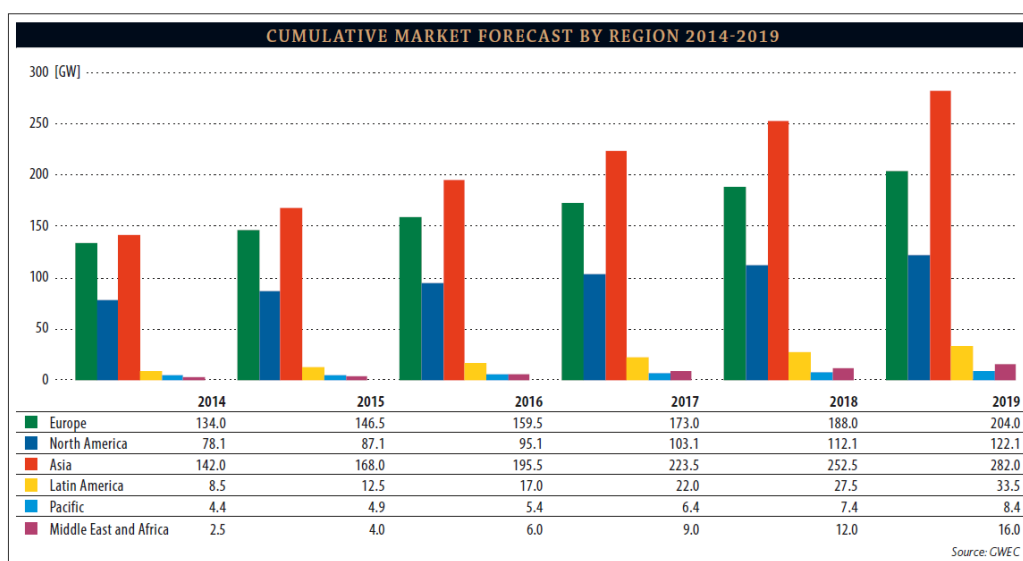


Figure 1- 10: Worldwide cumulative forecast by region since 2014 until 2019 [12]

Europe, North America and Asia will continue to grow in the coming years, these regions will continue to be the leaders of the market.

In Latin America, the Brazilian main market is expected to install another 12-13 GW on the country over the next 5 years of wind energy capacity. Wind energy will surpass gas energy in terms of installed capacity by the end of 2017, but hydropower will continue to be the main energy source of the country's electricity system [12].

The Pacific region had a total installed capacity crossing the 4.4 GW mark last year. The Australian main market added a capacity of 567 MW in 2014, and less than 655 MW in 2013, bringing its total installed capacity up to 3,806 MW. This decrease in generated capacity is derived from the current government which does not support renewable energy, but this might change, and Australia can easily become a competitive market again [12].

Africa and the Middle East are awakening to the opportunity of their enormous wind power potential. Nearly 1 GW (933 MW) of wind capacity was connected in the year of 2014, originating a total cumulative capacity for the region up to 2,545 MW, and it is expected that it will pass the 1 GW with room to spare in 2015 [12].

1.3.2. EUROPE

The European Wind Energy Association (EWEA, 2014), and the information reported from Lindoe Offshore Renewables Center (LORC) and 4C Offshore Limited (4CO Ltd.), states that Europe is in the lead regarding the number of installed offshore wind farms around the World [13].

The EWEA onshore and offshore wind power statistics stated that, in 2014, a capacity of 11,791.4 MW in the European Union was reached, an increase of 3.8% compared to annual installations in 2013. Of the capacity installed in the EU, 10,308.1 MW was onshore and 1,483.3 MW offshore.

The leaders of the European market in 2014 were Germany and United Kingdom with 59.5% of all new installations, an increase of 13.5% compared to the previous year. However, Denmark, Spain and Italy saw their rates of wind energy installations decrease significantly in 2014, due to political discussions.

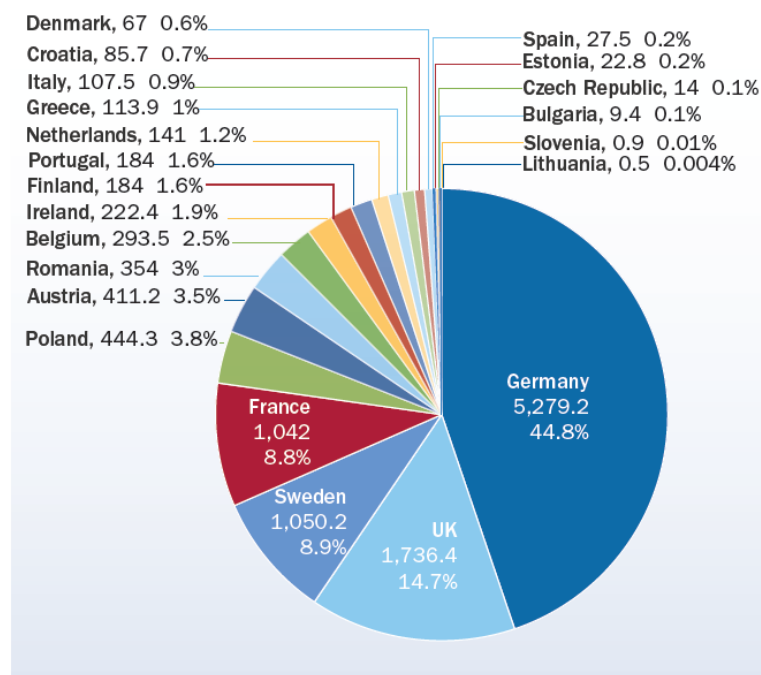


Figure 1- 11: New wind energy capacity installed during 2014 in the EU (MW), [3]

Wind power installations increased over the last 14 years, from 3.2 GW in 2000, to 11.8 GW in 2014, and it is possible to see in Figure 1-12 the growth along the years:

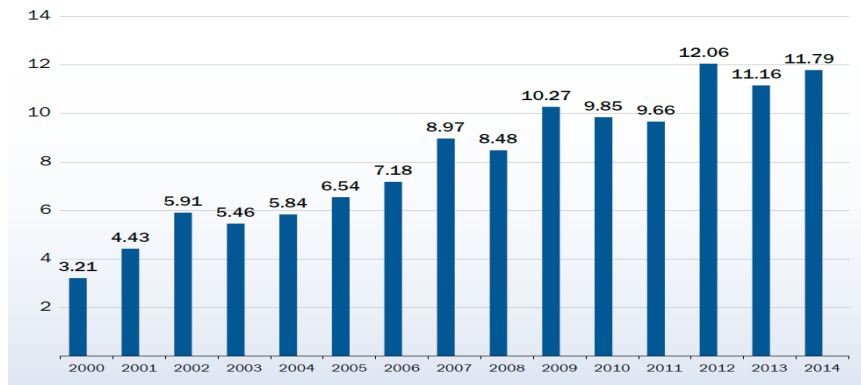


Figure 1- 12: Annual wind power installations in EU (GW), [3]

In the European Union, the cumulative wind power capacity is now 128.8 GW, an increase of 9.8% compared to 2013. Approximately 120.6 GW onshore and just over 8 GW offshore. In figure 1-5, it is possible to see the cumulative wind power capacity:

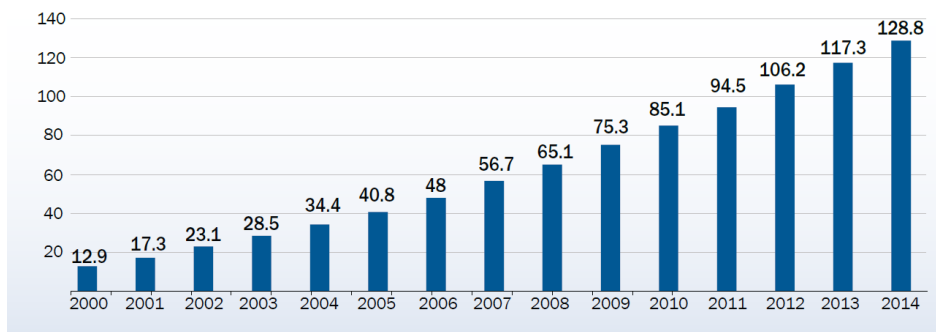


Figure 1- 13: Cumulative wind power installations in the EU over the years (GW), [3]

Focusing on the offshore statistics revealed in 2014, we can see that, in nine wind farms parks, 408 new offshore wind turbines were placed and fully grid connected, promoting a new capacity total of 1,483.3MW, 5.34% less than in 2013.

The cumulative offshore wind power market has now 2,488 turbines installed and grid connected, making a cumulative total of 8,045.3 MW in 74 wind farm parks in 11 European countries.

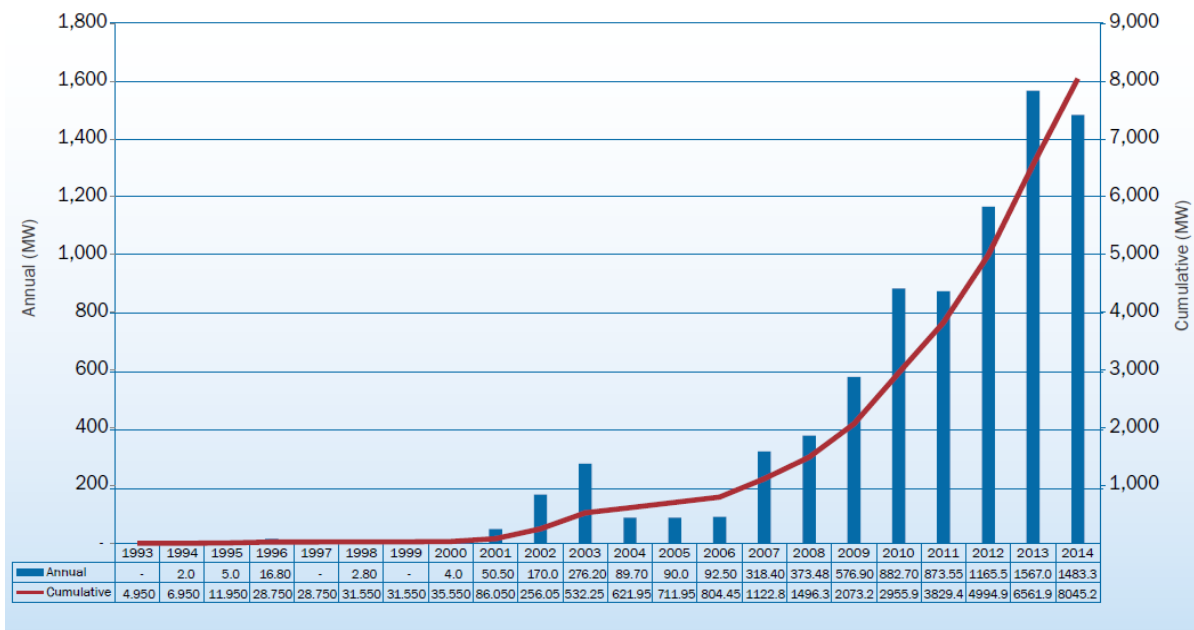


Figure 1- 14: Cumulative and annual offshore wind installations in the EU (MW), [3]

Europe has experienced an exponential evolution along the years (Figure 1-14), due to the investments made in the offshore wind energy industry across the European countries, [7], but this rate of growth is expected to slow down, according to the information given by the Global Wind Energy Council [12].

Table 1- 1: Installed capacity (MW), number of wind farms and turbines by European country, [3]

Country	BE	DE	DK	ES	FI	IE	NL	NO	PT	SE	UK	Total
No. of farms	5	16	12	1	2	1	5	1	1	6	24	74
No. of turbines	182	258	513	1	9	7	124	1	1	91	1,301	2,488
Capacity installed (MW)	712	1,048.9	1,271	5	26	25	247	2	2	212	4,494.4	8,045.3

Regarding to Table 1-1, the United Kingdom is the country with the largest amount of installed offshore wind capacity in Europe, following Denmark and then Germany, which made significant investments in offshore wind energy infrastructures during the year of 2014. Due to these improvements, Germany will pass Denmark and will become the country in Europe with second most offshore installed wind capacity.

Most of the 8,045.3 MW of offshore wind capacity are located in the North Sea with 5,094.2 MW – 63.3%. In the Atlantic Ocean are installed 1,808.6 MW – 22.5% and in the Baltic Sea are installed 1,142.5 MW – 14.2%, as shown in Figure 1-15:

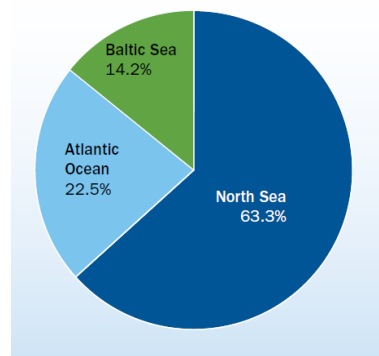


Figure 1- 15: Cumulative installed capacity in the sea basin (MW), [3]

Figure 1-16 compiles the information regarding to the annual onshore and offshore industry in 2014. Offshore wind installations were 5.3% less than in 2013, with 1,483.3 MW. Offshore wind power installations represent 12.6% of the annual EU wind energy market, down from 14% in 2013.

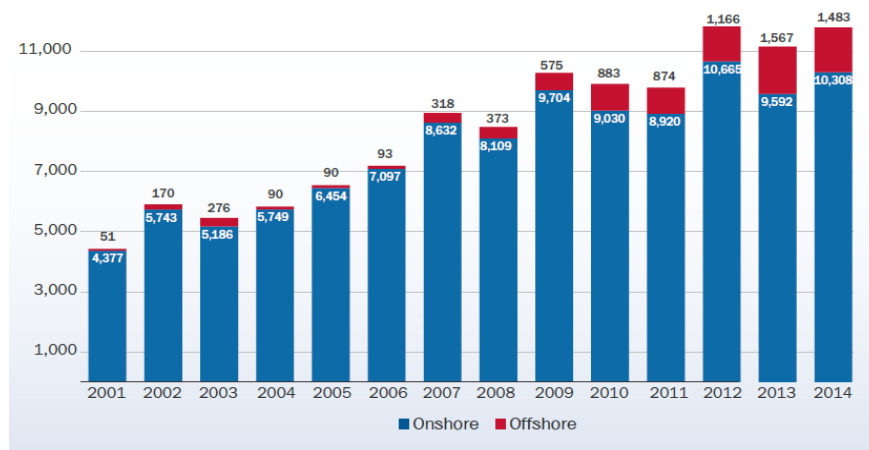


Figure 1- 16: Annual Onshore and Offshore Installations (MW), [3]

The average water depth of wind farms placed in 2014 was 22.4 metres (m) and the average distance to shore was 32.9 km.

For 2015 and 2016, the markets are expected to fully complete the 12 offshore projects that are currently under construction, increasing the energy capacity in 2.9 GW, reaching a total cumulative capacity in Europe of 10.9 GW.

The European Union will be able to produce 284 TW.h of electricity, enough to cover 10.2% of the EU's electricity consumption, 8% more than 2013, with all the wind farms placed until the end of 2014 in Europe.

Table 1- 2: Estimated values until the end of 2014, [3]

Total EU electricity consumption (TWh)	Onshore wind energy production (TWh)	Offshore wind energy production (TWh)	Share of EU consumption met by onshore wind (TWh)	Share of EU consumption met by offshore wind	Share of EU consumption met by wind
2,798	254.43	29.59	9.1%	1.1%	10.2%

Figure 1-17 shows the current wind farm parks in the north of Europe:



Figure 1- 17: Offshore wind farm parks in EU, [14]

Table 1-3 provides some information about the characteristics of the wind turbines installed in the offshore wind farm parks in European Union:

Table 1- 3: Specifications of some wind turbines installed in the EU offshore wind parks [13]

Offshore Windfarms	Year installed	Country	Type of Foundation	Pile Diameter	water depth
Teesside	2014	UK	monopile	4.6	8 - 22 m
Thanet substation	2010	UK	4 skirt piles	1.83 (4 skirt piles)	22 m
Thornton Bank	2006	Be	GBF	6.5-17	10 - 24m
Scarweather sands	2002	UK	monopile	2.2	11.7 m
Arklow Bank	2004	Ireland	monopile	5	2 - 6 m
Lynn & Inner Dowsing	2007	UK	monopile	4.74	6 - 13 m
N7	1997	Ger	monopile	6	7 m
Otzumer baldje inlet	2003	Ger	monopile	1.5	11.7 m

Humber Gateway	under construction (fully operational in 2015)	UK	monopile	4.2	10 - 18 m
Westerrmost Rough	partial generation and under construction	UK	monopile	6.25	12 - 22 m
Gwynt Y Mor	partial generation and under construction	UK	monopile	6	13 - 32 m
West of Duddon Sands	2014	UK	monopile	5 - 6	17 - 21 m
Gunfleets sands	2010	UK	monopile	5	0 - 13 m
Sheringham shoal	2012	UK	monopile	4.3 - 5.7	14 - 23 m
Rhyl Flats	2009	UK	monopile	4.7	6.5 - 12.5 m
Ormonde	2012	UK	group of 4 piles	1.83	17 - 21 m
Greater Gabbard	2013	UK	monopile	6.3	4 - 37 m
London Array Phase 1	2013	UK	monopile	4.7 - 5.7	0 - 25 m
Robbin Rigg	2010	UK	monopile	4.3	4 - 13 m
Vindeby (Denmark)	1991	Den	Gravity based	-	2.5 - 5 m
Lely (Netherlands)	1994	NL	Monopile	3.2 - 3.7	4 - 5 m
Tun Knob (Denmark)	1995	Den	Gravity based	-	3 - 5 m
Dronten (Netherlands)	1996	NL	Monopile	?	1 - 2 m
Bockstigen (Sweden)	1998	Swe	Monopile	2.25	5.5 - 6.5 m
Blyth Offshore (UK)	2000	UK	Monopile	3.5	6 - 11 m
Utgrunden (Sweden)	2000	Swe	Monopile	3	7 - 10 m
Middelgrunden (Denmark)	2001	Den	Gravity based	-	5-10 m
Yttre Stengrund (Sweden)	2001	Swe	Monopile	3 - 3.5	8 m
Horns Rev (Denmark)	2002	Den	Monopile	4	6 - 14 m

Frederikshaven (Denmark)	2003	Den	1 bucket foundation, monopiles	3 ?	4 m
Samsø (Denmark)	2003	Den	Monopile	4.2	11 - 18 m
North Hoyle (UK)	2003	UK	Monopile	4	12 - 20 m
Nysted (Denmark)	2004	Den	Gravity based	-	6 - 10 m
Arklow Bank (Ireland)	2004	Ire	Monopile	5.1	5 m
Scroby Sands (UK)	2004	UK	Monopile	4.2	21 (max)
Ems-Emden (Germany)	2004	Ger	Like land based	-	2 (max)
Kentish flats (UK)	2005	UK	Monopile	4	5 m
Breitling (Germany)	2006	Ger	Like land based	-	2 m
Barrow (UK)	2006	UK	Monopile	4.75	15-20 m
Beatrice (Moray Firth)	2007	UK	Lattice towers	-	45 m
Egmond aan zee	2007	NL	Monopile	4.6	19-22 m
Burbo	2007	UK	Monopile	4.7	1-8 m
Lillgrund	2007	Swe	Gravity based	-	4-8 m
Q7	2008	NL	Monopile	4	20-24 m

Observing the above table, it is possible to view the growth of these wind parks during the years. The increase of the distance to shore and the water depth of the wind turbines, as well as the number of turbines placed is very notorious in the table. So, the necessity of studying this type of structures is really important, due to the projects that are being developed, and since the market still present's remarkable growing tendencies.

Some of the new offshore wind projects under construction can be checked at 4C offshore data base [15], for example:

- In the Netherlands: Eneco Luchterduinen, Gemini, Westermeerwind;
- In United Kingdom: Kentish Flats 2;
- In Germany: Amrumbank West, Borkum Riffgrund I, Butendiek, EnBW Baltic 2, Global Tech I, Nordsee Ost, Trianel Windpark Borkum – Phase 1, Gode Wind 01, Gode Wind 02;
- In China: Longyuan Rudong Intertidal Trial Wind Farm – Extension, CGN Rudong Offshore Demonstration Project, Donghai Bridge Offshore Wind Farm Phase II, Hydropower Rudong Offshore Wind Farm, Jiangsu Longyuan Intertidal Wind Farm Demonstration Project Phase II, Longyuan Putian Nanri Island, Putian Pinghai Bay Offshore Demosntration Project Phase A,

Xiangshui Offshore Demonstration Project, Zhuhai Guishan Offshore Wind Farm Demonstration Project.

The research hereby developed allowed for interesting conclusions, which are summarised in the following paragraphs. The technological review of the offshore wind market and offshore structural design led to conclude that:

- Europe is still in the lead of the offshore structures' market and China is now growing at a remarkable rate;
- the new arising projects are becoming more demanding in terms of design requirements that must resist to the harsh conditions of the sea state and environment loads;
- The new tendencies show that monopiles are still the most common type of substructure used worldwide;
- The growing tendencies are leading to increasing diameters and increasing weights of the components (e.g. rotors, blades, hub towers), which also lead to higher solicitations and internal forces that must be accounted for;
- Unlike other design fields of civil engineering, where there are very restrict and specific codes that must be applied for design purposes, in offshore engineering the norms and standards that can be applied vary considerably. However, some internationally recognised guidelines can be found, as DNV standards, API standards or GL, IEC and ISO standards. The present thesis provided a case studied mainly based on DNV, Eurocode and API's documents. However, the comparisons with other guidelines and recommendations must be further analysed in following investigations.

The present thesis intends to continue the research that has been made in this field. It is the aim of the study to contribute for a deeper and more solid knowledge in offshore engineering and offshore structures. This research will also provide a first approach to the complex reality of structures subjected to harsh environmental loads, such as the marine ones, hence providing a starting point for future analysis and researches to be performed on a more complex level.

1.4. CURRENT STANDARDS USED FOR DESIGN

Offshore projects are subject to legislation which depends on the geographic location of the structures, such as National, European or International codes. In Portugal, there is no National legislation relating to offshore platforms, so the legislation used has to be based in European or International codes.

In this area, the reference certification entities used by designer engineers for offshore projects are:

- Det Norske Veritas (DNV);
- American Petroleum Institute (API);
- Germanischer Lloyd (GL);
- International Electrotechnical Commission (IEC);
- International Organization for Standardization (ISO).

The standards from these entities allow for good bases for design procedures, respecting the safety of the structures, but these standards are still little specific, enabling the use of different design procedures, as well as having a few gaps in some fields, such as probability of failure [4].

Below are presented the standards for the design of offshore wind turbines that were consulted during the writing of this thesis:

- DNV-OS-J101: Design of Offshore Wind Turbine Structures, May 2014, [16];
- DNV-DS-J102: Design and Manufacture of Wind Turbine Blades, Offshore and Onshore Wind Turbines, October 2010, [17];

- API-RP2A: Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress Design, December 2002, [18];
- GL: Guideline for the Certification of Offshore Wind Turbines, 2005 [19];
- Danish Energy: Recommendation for Technical Approval of Offshore Wind Turbines, December 2001, [20];
- IEC 61400-1: Wind Turbines – Design Requirements, 2005, [21];
- IEC 61400-3: Design Requirements for Offshore Wind Turbines, 2002, [22];
- Eurocode 3: Design of Steel Structures, NP EN 1993-1-1, 2010, [23].

1.5. OBJECTIVE AND THESIS OUTLINE

Along the years, the market of offshore wind farms has been growing with new engineering challenges. Nowadays, with the expansion of the offshore wind projects and the modern offshore wind turbines that offer competitive production prices, the global costs of offshore wind energy can be reduced. Therefore, it is possible to reach this goal by optimizing and identifying the critical design parameters.

The objective of this thesis is the design of a wind turbine with a monopile foundation, since this is the more common structure for such kind of projects. Other point is that nowadays, another generation of piles is being developed, the so-called XL-monopiles which can reach greater depths and support bigger turbines. This type of foundations are able to reach up to 45 meters of water depth [24].

The thesis is structured in the logical order of cause and event. In chapter 1, we have an introduction of wind turbines, showing the advantages and disadvantages of onshore and offshore wind farms. The statistics and data of the worldwide offshore and onshore wind industry are also shown.

Chapter 2 concerns the foundation design parameters and the general types of foundation used for offshore wind turbines. The turbine specifications are also presented in this chapter.

Chapter 3 defines the several types of load actions that are present in this kind of offshore structures, referring which loads will be studied over the course of the thesis. The loads' quantification and the calculation method are also described during this chapter.

In chapter 4 is presented the case study with all the results obtained from the loads' quantification and stresses in the critical section. At the end of this chapter, the safety verifications of the structure are made.

Chapter 5 summarizes the conclusions and ends with recommendations for further works.

2

A Review of Wind Turbines Characteristics and Their Foundations

2.1. WIND ENERGY

2.1.1. INTRODUCTION

Wind energy has been used by farmers and ranchers for more than two thousand years for pumping water and grinding grain, so it was mainly used for transforming wind energy into mechanical energy. Nowadays, wind energy is utilized to generate electricity through wind turbines requiring a huge development in engineering techniques, focusing in high efficiency [25].

2.1.2. PROCESS OF OFFSHORE WIND ENERGY

Offshore wind farm parks have multiple wind turbines linked to each other, where the energy produced from all of them is collected and transported to a grid connection point in shore, and then integrated into the public grid.

The electrical energy that is collected from the wind turbines is sent to the offshore transformer substation platform which converts the electricity from around 700 Volts (V) to the correct voltage for distribution, normally 33 KV [26]. After that, the energy is transported to shore and distributed around the country. The electrical energy transportation is made through subsea cables and it has two types of technologies, alternating current (AC) and direct current (DC) [27]. Figure 2-1 represents what was described before:

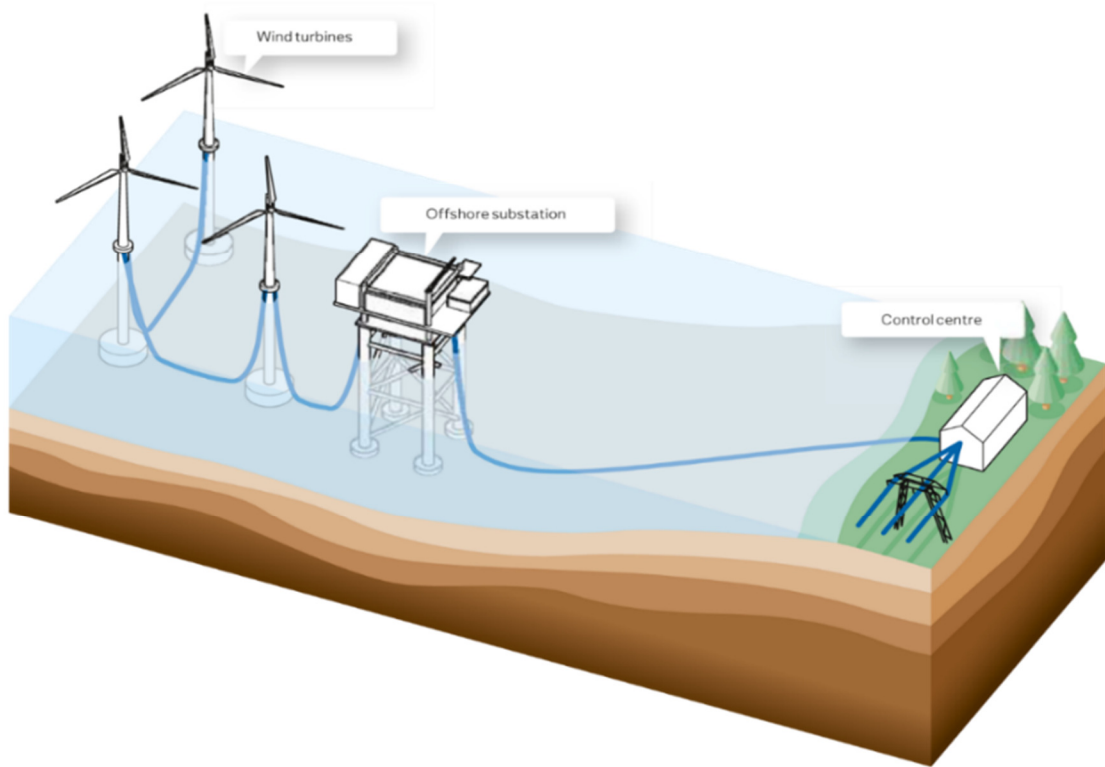
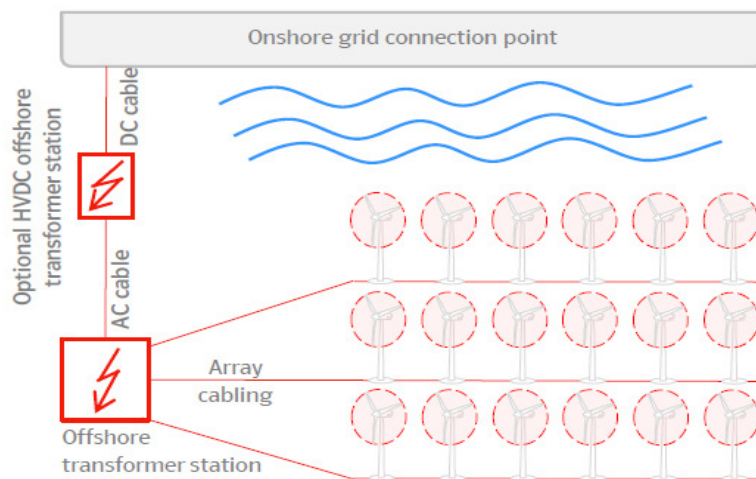


Figure 2-1: Process of offshore wind energy near from shore [28]

For long distances from shore, DC cables are used for power transport, which requires an additional HVDC transformer station offshore to transmit the electricity via DC. Then, at the grid connection point onshore, another transformer station is needed to convert the electricity via AC, in order to be distributed to our homes. This situation is represented in Figure 2-2 below:



In the offshore wind farm, array cabling (AC) joins the rows of wind turbines until the offshore transformer station, then an AC cable is connected to the HVDC transformer station.

Figure 2-2: Diagram for long distances from shore, [29]

For safe operations, offshore cables are buried up to three meters deep into the seabed, [29].

2.2. OFFSHORE WIND TURBINE

The offshore wind turbines market has been experiencing huge improvements along the years, such as the dimensions of the wind turbine, rotors diameter and hub heights. These innovations happened in order to accompany the evolution of the market and to reduce the costs per energy unit produced [24]. Offshore wind turbines must be highly robust and reliable to avoid costly standstills, as a result, they sometimes cannot be accessed because of the weather conditions, wind, wave and huge currents. Another concern factor is the aggressive salty environment that damages the equipments through the corrosion process [29].

Through the past decades, the size and capacity of offshore wind turbines have increased considerably. In these days, wind turbines with a capacity of up to 7 MW are being tested [29].

The first turbines appeared with a capacity of energy production of 2 MW. In more recent days, 5 MW turbines are assembled, 10 and 15 MW are being developed, and even more capacity of wind turbine production is expected in the future [11].

2.2.1. CHARACTERIZATION

Wind turbines can be characterized through several factors, but below is presented the most significant ones, such as:

- **Location of implementation:** the wind turbines can be installed onshore (in land) or offshore (at the sea), like it was shown above in chapter 1.
- **Rotation axis of the turbine:** the wind turbines, due to their axis of rotation, can be classified as turbines with a vertical axis of rotation or turbines with horizontal rotation axis, as shown in the figures below:



Figure 2- 3: Wind turbine with a vertical axis of rotation, [30]



Figure 2- 4: Wind turbine with a horizontal axis of rotation, [30]

Throughout the thesis, it is going to be study only the wind turbines with a horizontal axis of rotation, since they are more efficient, as well as the only type installed at offshore locations. Currently, turbines with a vertical axis of rotation see almost no use.

- **Type of tower used:** there are several types of towers for wind turbines, differing in the type of material used and the type of structure of the tower, as shown below:

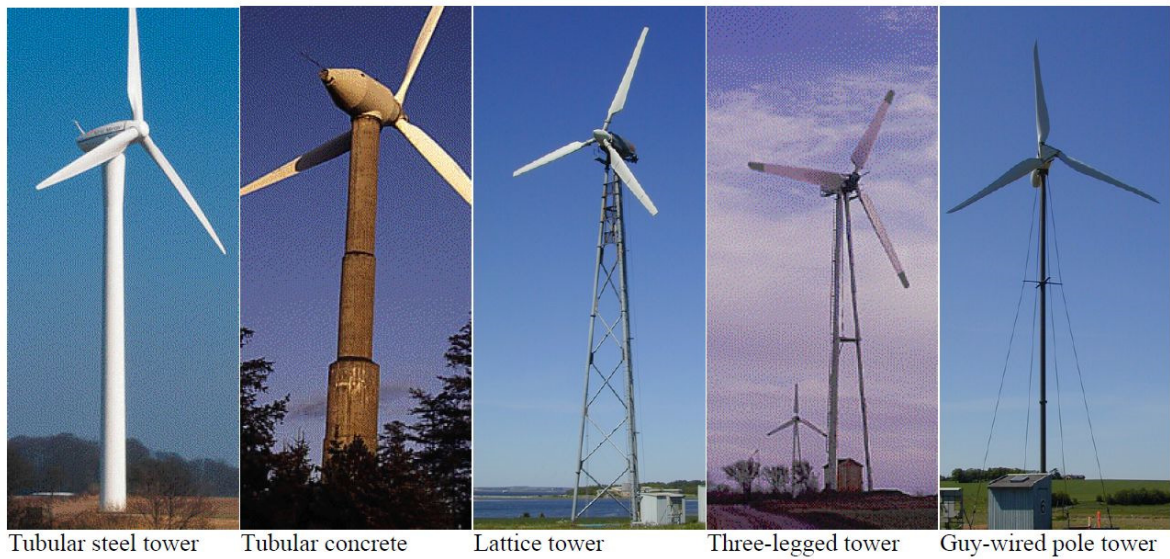


Figure 2-5: Types of towers used for wind farms projects [31]

Nowadays, the wind turbine market is dominated by the tubular steel towers, so for this reason these are the type of towers that are going to be studied during the thesis.

These towers are constituted by cylinders made of steel plates welded longitudinally. The cylinders are all connected by transverse welds, in order to obtain one tower section. Each section then finishes with a steel flange on both end, which bolts the sections to each other.

In this type of towers, the increase of the diameter corresponds to a reduction of the plate thickness, thus increasing the tension on the tower, but decreasing the buckling [32].

- **Position of the blades relative to the wind:** in the turbines with a horizontal axis, the blades can be positioned by two ways, like it is shown below:

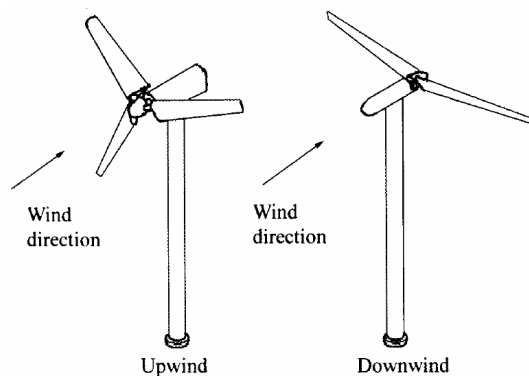


Figure 2-6: Distinction between Upwind and Downwind turbines [33]

Along this work we are going to study the upwind turbines, being the most usual on the wind farm projects.

2.2.2. COMPONENTS

As the wind blows, it flows over the airfoil-shaped blades of wind turbines, causing the turbine blades to spin. The blades are connected to a drive shaft inside the nacelle, the shaft goes into a gearbox which increases the rotation speed and the generator converts the rotational energy into electrical energy.

Figure 2-7 gives an introduction of the components that constitute an offshore wind turbine:

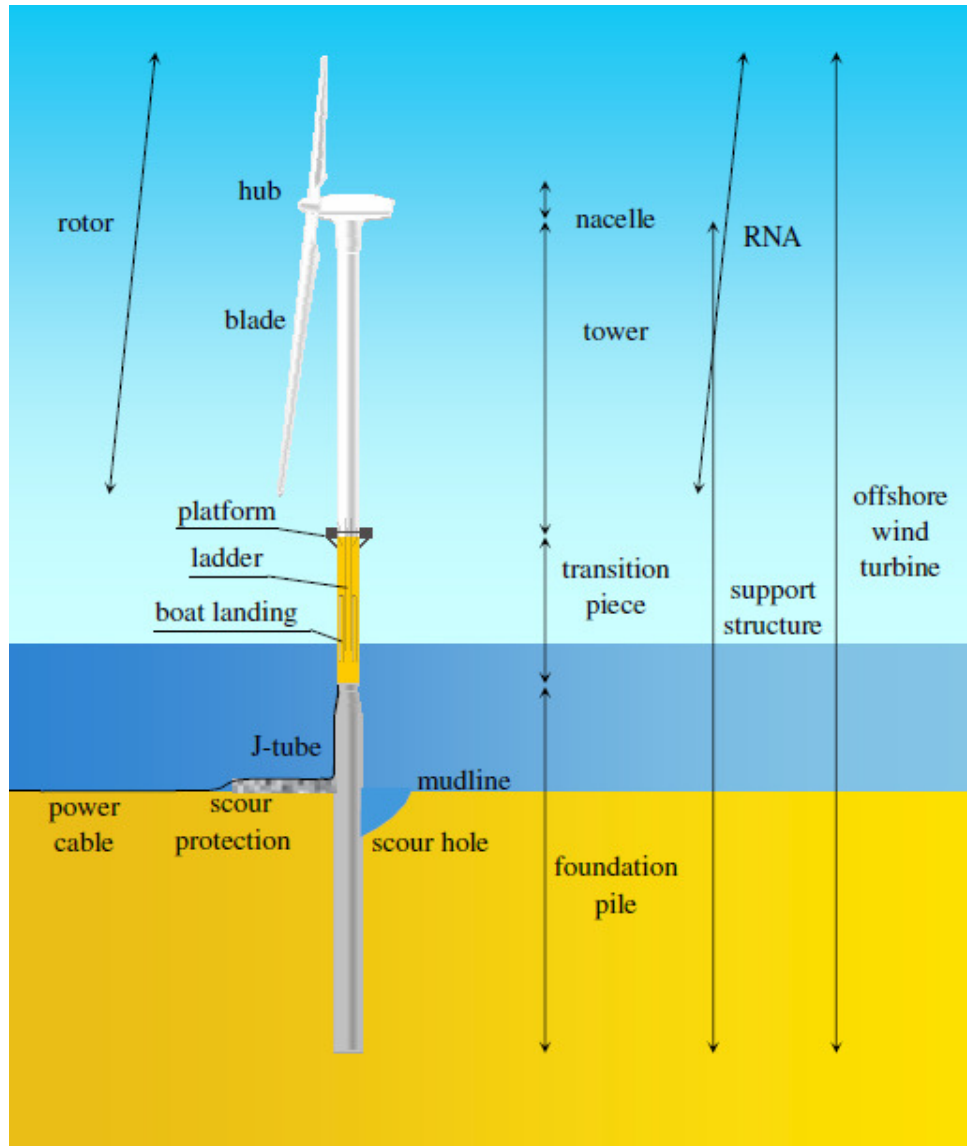


Figure 2-7: Offshore wind turbine terminology, [34]

The characteristics of the principal components will be presented below [34] and [11]:

- **Nacelle:** contains the electromechanical components of the wind turbine, including the generator which converts the mechanical rotational energy from the wind into electrical energy. Above the nacelle, an optional helicopter pick-up area is used for an easier maintenance. Figure 2-8 shows the different types of components located inside of the nacelle.

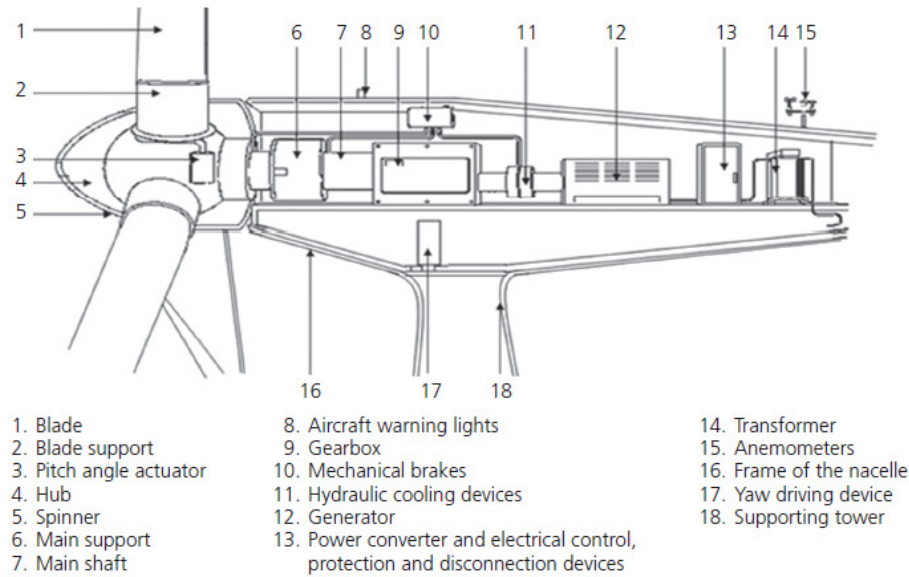


Figure 2-8: Nacelle electromechanical components [35]

- **Rotor:** contains the hub and the blades. These can be made of plastic reinforced with fibreglass, or manufactured in steel for bigger turbines. The blades are connected to the hub, which transmits the rotational energy to the gearbox via the main shaft. The blades' size usually have between 80 and 100 meters in diameter [11], and their rotation speed is between 10 to 30 rpm [26]. The bigger they are, the more energy it is possible to obtain. Bigger rotor blades up to 200 meters are in the testing phase [36].
- **Tower:** provides support to the assembly of the nacelle, blades and hub. Dependent on the emplacement location and height, it is a tubular structure made of steel or cement, and it is constructed through several sections. Typical tower heights range from 80 to 130 meters and it contains a ladder or elevator inside of it in order to reach the nacelle.
- The **transition piece** connects the tower to the driven pile foundation. This component is provided with a boat landing, a ladder and platform which gives access to the entrance of the tower. This element is only used in monopile support structures.
- The **foundation** contributes for the support of the wind turbine. Different types of foundation structures exist and will be presented in this chapter.

2.3. TYPE OF FOUNDATIONS USED FOR OFFSHORE WIND TURBINES

2.3.1. INTRODUCTION

The objective of the present thesis is the design of an offshore wind turbine foundation, particularly the monopile foundation, since it is the most used for this kind of offshore substructures. By the end of 2014, the monopiles continued to be the most common type of structure being installed [3] and [24], Figure 2-9 shows the percentage of the different types of foundations used until the recent days:

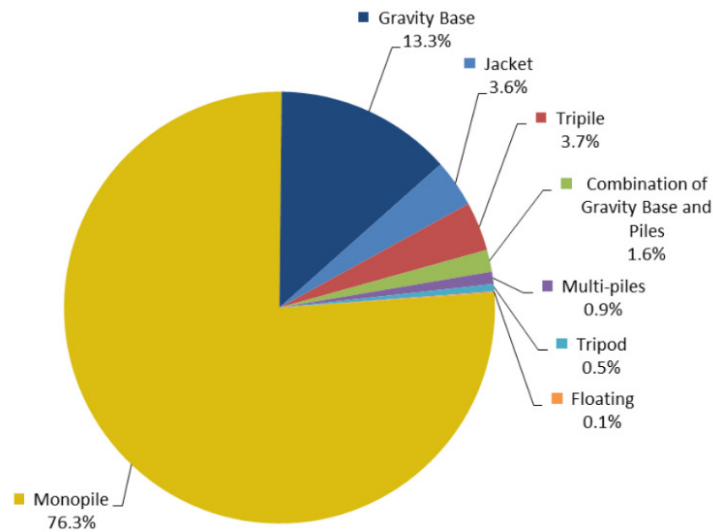


Figure 2-9: Distribution of foundation types for offshore wind turbines until the end of 2014, [24]

This chapter will present the various existent types of foundation structures that can be used in offshore wind turbines, including the improvements that have been taken in the past years, due to the increasing distances from shore and water depths. The technical issue about the monopile design will be studied in detail in the following chapters.

Several support structure options are available to be adopted, dependent on the site conditions, the installation process, water depth and the size of the turbine for employment [37].

The knowledge of the offshore oil and gas sector in support structure design has been transported to the wind industry to provide concepts for foundations of offshore wind turbines.

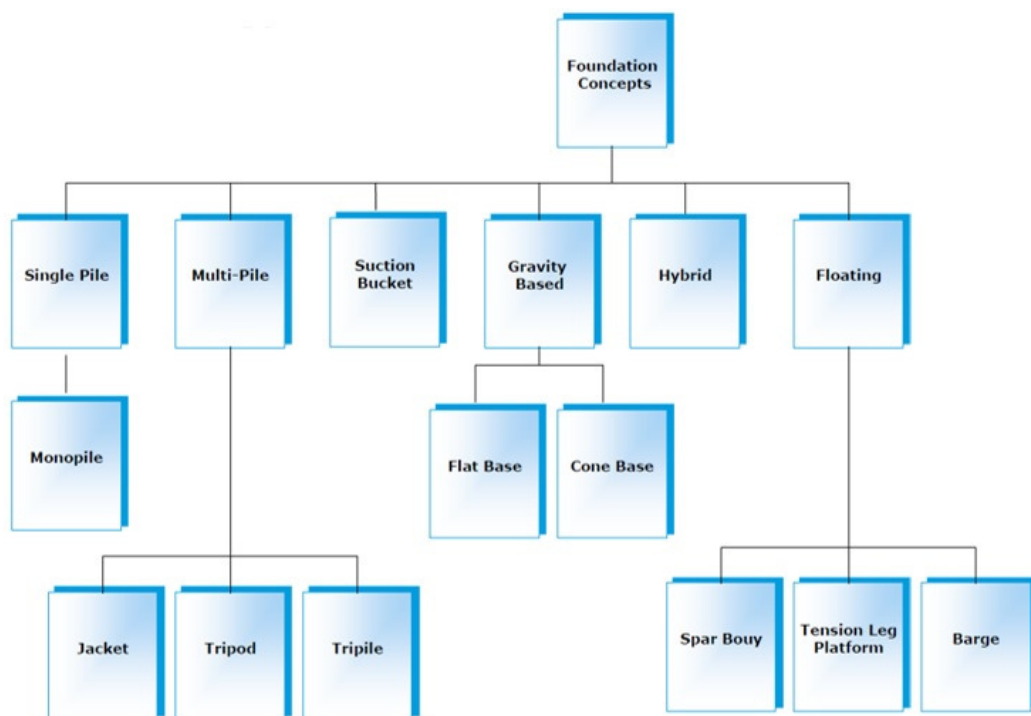


Figure 2-10: Different types of foundations used for offshore wind turbines [38]

For **shallower water**, the monopile and the concrete gravity bases foundations are the greater solutions. For **some depth water** foundations, the tripods and jackets which have been predominantly used in offshore oil and gas structures are a better solution. For this type of support structures, it is mandatory to maintain the strength and stiffness requirements, to prevent the fatigue on the structures elements, due to unacceptable displacements or rotations of the structure at the lowest possible cost.

At **deep water**, floating support platforms, tied to the seabed with tension anchors, are the most economical solution.

These foundation concepts are illustrated schematically as a natural progression in Figure 2-11:

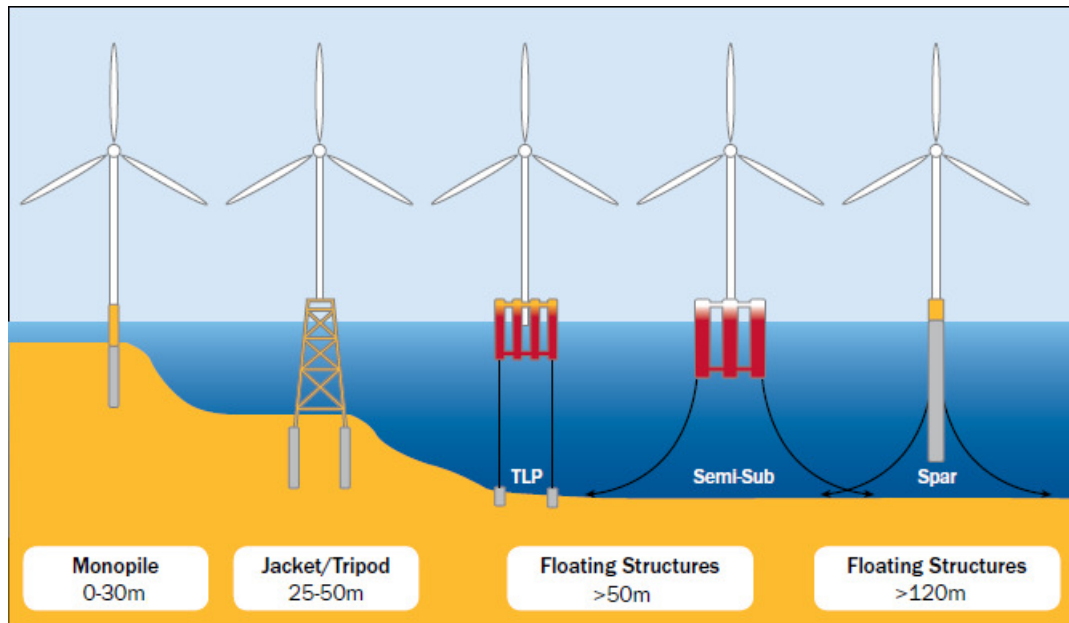


Figure 2-11: Type of support structure according to the water depth [13]

The water depths values presented may vary a little, so they only serve as a reference, and they can be consulted in DNV [16] or [39]. For deeper structures and greater distances from shore, there is a variety of offshore support structure that has been increasing in the past years [13].

Much of the offshore wind resource potential in the United States, China, Japan, Norway, and many other countries is available in water deeper than 30 m. However, all the European offshore wind turbines installed to date are in water shallower than 20 m on fixed-bottom substructures [2].

In offshore wind farms, the foundation represents an important part of the offshore total investment, while in onshore substructures the capital cost percentage is approximately 6.5% on average [40] and [13].

Offshore foundations are the most expensive elements of the entire structure, after the turbines, and may represent about 20 to 35% of an offshore wind turbine [40] and [7]. If these costs only concern the installation, then the foundations are actually the most expensive component [13].

For these reasons, it is necessary to reduce the costs, with the goal of having a better economic efficiency and design by optimizing the substructures [40].

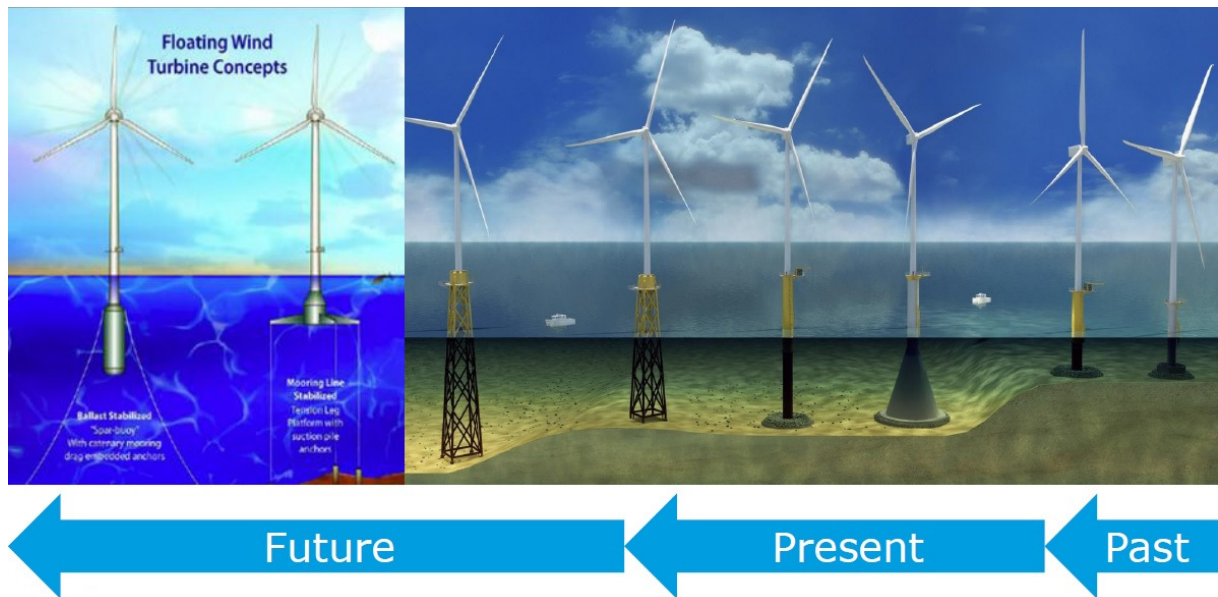


Figure 2- 12: Development of the offshore wind turbine foundations [38]

2.3.2. Gravity Based Foundations

Gravity based substructures are the second most used in offshore wind farms foundations, but in recent years, the applications of this type of substructures have been decreasing, due to the necessity of going more into deep waters and their use is not so profitable (EWEA, 2014). The limitation becomes more evident for water depths greater than 15 meters [24] and [13].

During the year of 2013, the Karendamn Offshore Wind Farm (48 MW) in Sweden was the only project installed. According to EWEA, gravity-based foundations corresponded only to 0.1% of the installed foundations in that same year [3].

In 2014, there is no record of any emplacement of this type of foundations support, unlike in the early days of offshore wind energy in Denmark, when they were very popular [3].

These type of substructures are not as used in the recent days, because of the construction method, the spent time for curing the concrete, the dredging requirements for seabed preparation, and the heavy lift vessels that are needed, so it is not profitable to use [13].

Typically, gravity based foundations are a huge concrete structure designed to support the moments and forces generated by the turbine and by the environment conditions, and they are used for shallow water depths. Figure 2-13 presents a conical base foundation.

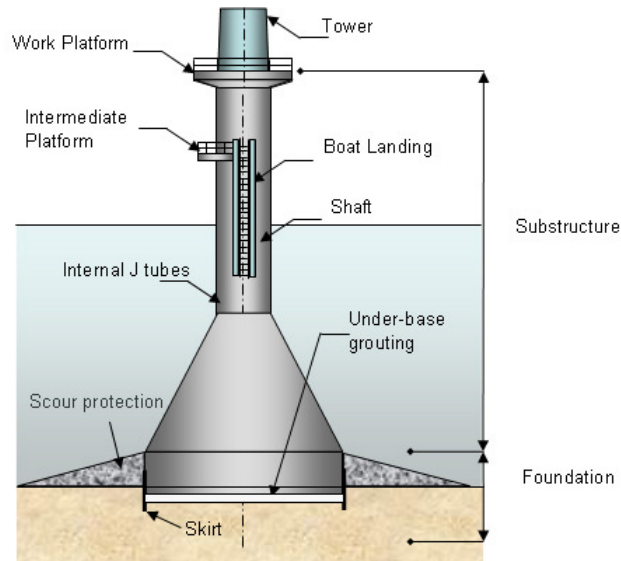


Figure 2- 13: Components that constitute a possible gravity based foundation [41]

The gravity based structure is hollow to reduce the weight for an easier transportation and emplacement. After the seabed preparation, the substructure is placed in position and its interior is filled with ballast providing the overall design weight. Scour protection is essential for this type of foundation due to the large diameters involved, which can be as great as 22m [42]. These structures normally do not penetrate the seabed, but are supported by it, and they can also be designed with a flat base.

Gravity based principal characteristics [43]:

Advantages:

- the material that it is made of is not expensive (concrete), and it is readily available in terms of raw material;
- the complete structure has a lower weight due to the hollow section that it is built on, being easier to handle;
- Smaller wind turbines can be manufactured onshore and then transported to offshore site, reducing the global costs.

Disadvantages:

- once the gravity based foundation does not penetrate the seabed ground, the overturning moments has to be considered and designed;
- the substructure is placed directly on top of the seabed, so its superficies needs to be prepared, in order to level the ground for the correct positioning and completely upright. This process increases the installation costs;
- due to the larger base it requires an extensive scour protection, more than a monopile foundation.
- installation is limited to deeper waters.

2.3.3. Tripod / Tripile Foundations

Tripod, as the name suggests, is a three-legged support, capable of providing greater stiffness and lateral stability than a single monopile [43].

The emplacement of this type of substructures requires a pre-installation of three monopiles driven into the seabed using a vibratory hammer technique to a depth of 21 meters, while the remaining depth is achieved by a hydraulic hammer [44]. The last part of the piles is hammered to prove the required bearing capacity of the piles. This question was discussed in an interview with big knowledge experts [45].

The diameter of the three monopiles is less than a single monopile that would be required to support the same wind turbine.

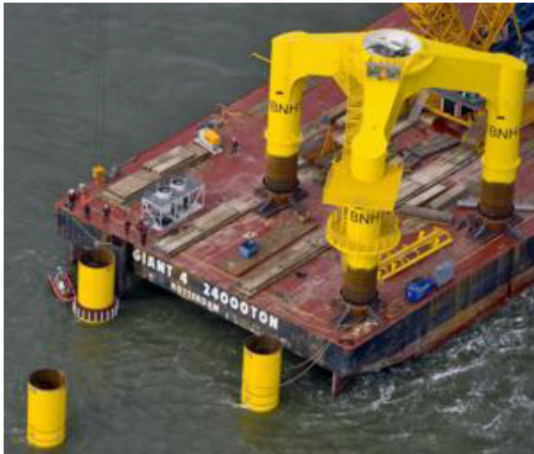


Figure 2-14: Tripile foundation support [43]

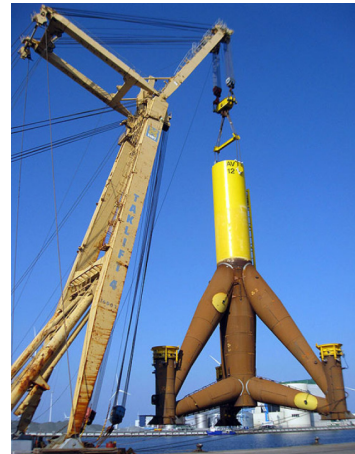


Figure 2-15: Tripod foundation support [42]

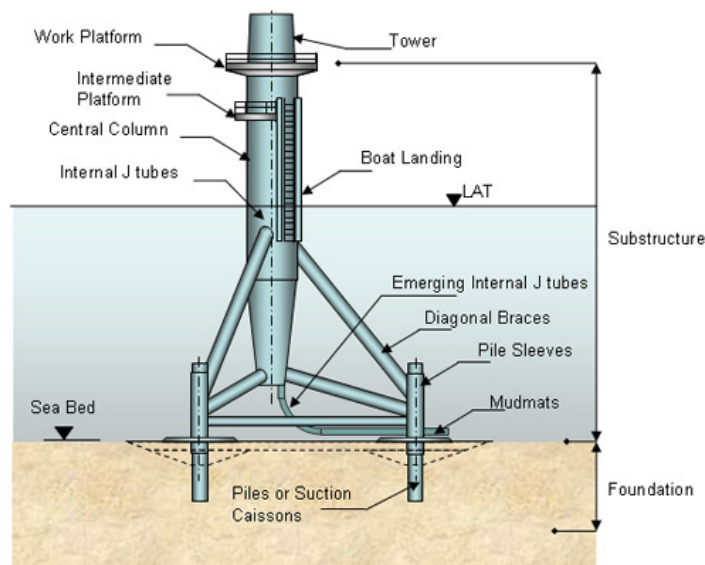


Figure 2-16: Elements that constitute a tripod foundation [41]

Like is shown in Figure 2- 16, the central column in the tripod transfers the loads from the tower through the diagonal braces to the pile sleeves which carry the load to the driven piles. The pile sleeves and the driven piles are connected between a grouted connection to allow the verticality correction through the tripod and the driven piles.

The tower is connected to the top flange of the tripod structure by bolts. Further on, the nacelle, hub and braces are assembled to the tower.

This foundation support is designed for water depths from 25 to 50 meters.

Advantages:

- can be placed in water depths up to 50 meters, much deeper than a standard monopile;
- better in transferring the loads from the tower, providing a greater lateral stability and stiffness;
- uses less material to be manufactured than a single monopile for greater depths.

Disadvantages:

- the process of installing the three driven piles needed by this structure increases the costs of this solution;
- all the three driven piles have to be designed for the extreme load case, because of the weather conditions, wind and waves that come from every direction and are constantly changing. This makes the structure heavier and more expensive;
- the transportation of this type of structures is more complex than a simple monopile, requiring bigger vessels.

2.3.4. Jackets Foundations

Based on the oil and gas industry technology, jacket support structures consist of a combination of circular hollow sections welded together with fabricated nodes at the joints, in other words, it uses the basic truss structure to provide stability and strength and it is easier to manufacture into large sizes.

Jackets were the preferred offshore support structures, but as the water depths increases, the placement of the offshore rigs requires other types of solutions that are more profitable.

The wind energy sector is improving and the turbines are getting bigger, heavier, and required to be assembled in deeper waters, so the designers opted to the jacket support structure. The jacket foundation is fixed to the seabed using piles that are driven through pile sleeves. These piles are installed through impact and vibratory hammers [43]. Figure 2-17 demonstrates the characteristics of a jacket support foundation.

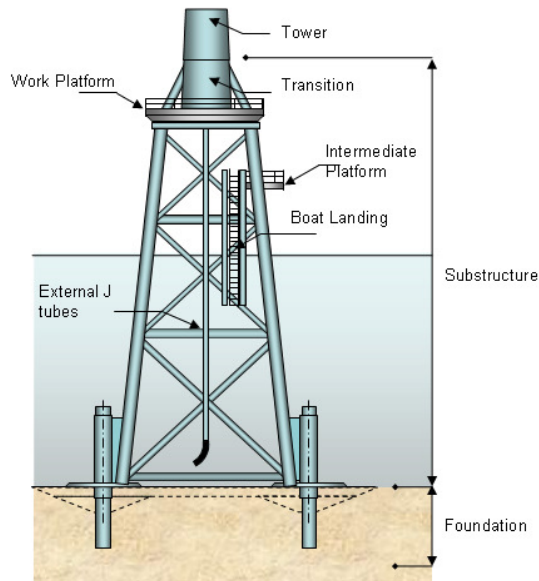


Figure 2- 17: Elements that constitute a jacket foundation [41]

Jackets can be fabricated in three or four legs, comprising tubes of diameter between 864 mm and 1200 mm, depending on the structure configuration, site conditions and the weight of the turbine applied [42]. As with the tripod foundation, a grouted joint is used to connect the jacket to the driven piles, in order to improve the axial capacity at the pile head. A transition piece is fitted above the jacket, allowing the connection of the tower to the jacket by bolts to the top flange.

Dynamic load actions resulting from the wind, waves and operations are divided into single axial pile forces that are transmitted to the driven piles. Depending on the stiffness of the soil, scour protection may or may not be required.

The jacket truss structure is very sensible to fatigue problems, and using fabricated joint connections decreases the fatigue lifetime, so it is better to use castings in the joints.

Cast nodes are single elements which provide an improvement of the connections, having a better performance relative to fatigue, reducing the need for complex weld details. The legs and braces are connected to the cast node by means of circumferential welds giving a simpler connection for design [42]. Figure 2-18 shows a sample of a cast joint:



Figure 2-18: Example of a cast join [42]

Jacket principal characteristics [43]:

Advantages:

- easy to design in order to improve the structure stiffness. However, to obtain more stiffness on the monopile structure is necessary to increase the diameter or the thickness wall of the pile (additional steel is needed);
- The complete support structure is not so heavy, being better for transportation, and using less quantity of steel when compared to the monopile.

Disadvantages:

- Higher manufacturing costs due to the complexity of the structure, many connections have to be done between components (legs and braces);
- In the situations where it is necessary to use a scour protection, it is not easy to install, due to the inner parts of the jacket being difficult to reach;
- The design and analysis of a jacket is more complex than a monopile's, and it is necessary to make an additional stress check (showed in Figure 2-19) for the joints and members, leading to more time consumption.

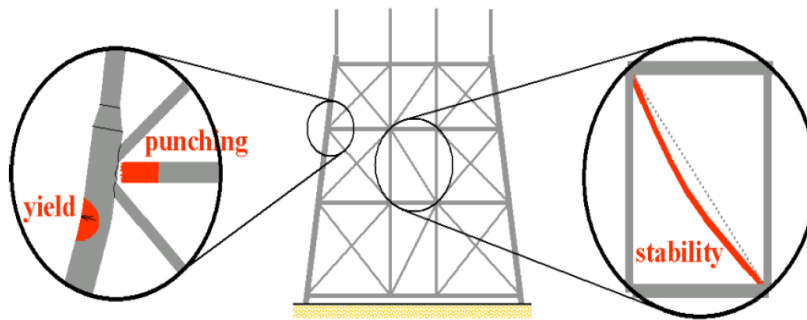


Figure 2-19: Stress checks for jacket support [43]

2.3.5. Floating Foundations

Local zones with better wind conditions are generally found in deeper water zones, usually with more than 60 meters depth (Navigant Consulting Inc., 2014). Therefore, the necessity of designing new types of foundation support structures such as floating solutions appeared, in order to reach greater depths. This new concept of foundation, compared to the traditional support structures that have been applied until the recent days, reduces the quantity of material needed to manufacture the complete substructure, eliminates the complex installation process until the seabed, and the decommissioning process is much easier.

Applying this new type of foundations in the available area for wind energy production overseas increases the power capacity, and efficiency will be maximized in deep sea locations [13].

After the construction of floating oil rigs, the new concept of offshore floating wind turbines appeared. These structures are more complex than a floating oil rig because of the huge mass of the nacelle and blades that are supported only by a unique tower, being very difficult to sustain the complete structure. Unlike the floating oil rig, it is much easier to be laterally stabilized due to the large platform area. Therefore, the mass supported by the single tower has to be balanced with a huge mass submerged underwater to obtain the desirable stability. Figure 2-20 portrays this situation [43].

Thus far, new concepts of floating foundations have been developed, like it shows in Figure 2-21 an additional extra floater that is added to the single tower, increasing the floating area underneath the turbine making it stable. In figure 2-22 is represented another possible solution that uses an active balancing method, the extra floater (extra mass) balancing the weight that arrives from the tower. For these two substructures, it is not necessary to use such huge mass submerged underwater like it is shown in figure 2-20.



Figure 2-20: Floating foundation [46]

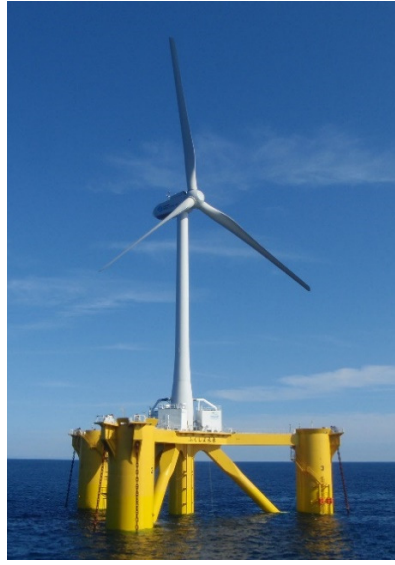


Figure 2-21: Floating foundation [47]

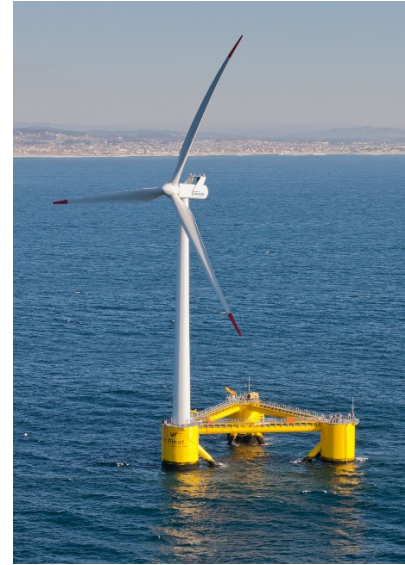


Figure 2-22: Floating foundation [15]

The floating turbine substructure is held in place by anchors installed into the seabed.

In some countries, like Japan, Portugal, Italy, Spain and U.S.A., the shore can have a reasonable extension of deep water areas, so it is expected that these markets develop new techniques in order to improve the floating offshore wind turbines foundations industry [7].

In the future, offshore floating foundations will be the most requested, due to the capacity of energy that can be produced far away from shore, but is still a very expensive technology because it is very recent, and more studies have to be done in order to improve the technology and decrease the costs associated to its substructures.

Consequently, some problems are being analysed, like the better configuration for the axis of the wind turbine [48], the cost of energy produced during the life cycle of the complete structure [49] and the substructures' dynamic behaviour under extreme conditions [13].

Floating foundations main characteristics [43]:

Advantages:

- easy to transport, due to the floatable base of the wind turbine. It can be towed to the site, reducing the process costs of loading and unloading all the components into barges;
- possible to install in deeper waters;
- no scour protection is required;
- these substructures can be manufactured and assembled completely onshore and then towed to the correct location, allowing to reduce the production and installation costs. Regarding maintenance, it is also possible to bring the substructure to shore for repairs;
- the installation process of these structures do not cause noise.

Disadvantages:

- in shallow waters, it is not rentable to install this type of substructures due to the expensive technology used;
- the stability of the wind turbine is a huge concern.

2.3.6. MONOPILE

Monopile foundations have been used for offshore oil and gas platform foundations for decades. They are the most common support structures for offshore wind turbines, and as we can observe in Figure 2-9, around 76 % of all turbine foundations installed to date are founded on monopiles.

These substructures have proven to be an efficient solution in reasonable ground conditions and in water depths up to 35 meters. These piles resist lateral wind and wave loading (and resulting moments) by mobilising horizontal earth pressures in competent near-surface soils [50].

The complete structure consists of a single large-diameter, thick walled, steel cylindrical tube (pile) driven into the seabed (using hammering or vibration techniques), a transition piece with a grouted connection that joins the pile, and a tower in which the turbine is mounted on top. The substructure includes a boat landing and a work platform for the maintenance of the structure and turbine components. The J tubes, which can be external or internal, transport the cables from the nacelle until the seabed.

The description above is illustrated in Figure 2-23:

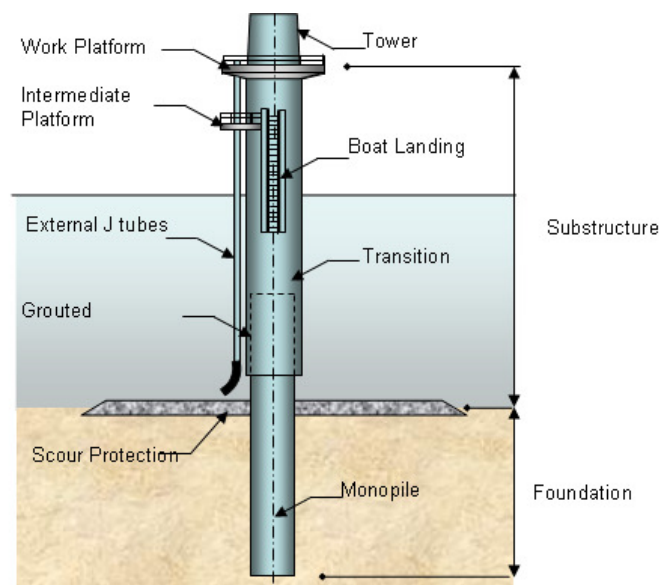


Figure 2- 23: Components that constitutes a monopile [41]

Since its introduction in the offshore industry, the monopile has become larger, heavier and has been installed in deeper depths waters [43]. These substructures will continue to be used in the coming years due to the huge advantages that it brings, and they will be presented in the next points below.

In the recent days, we heard about XL-monopiles for XL turbines. At EEW Special Pipe Constructions, in Rostock, Germany, XL-monopiles of 940 tonnes are being manufactured, and will be deployed during 2015 at DONG Energy's offshore wind farms Gode Wind 1 and 2. This XL-monopile has a length of 66,5 meters, a diameter of 7,5 meters, besides the total weight of 940 tonnes [51].

So, it is proven that this type of monopiles can reach greater depths and has a capacity to support bigger wind turbines.

As a result of the diameter's size and the applied loads, the foundation needs a scour protection that is usually estimated at 2.5 times the diameter of the tube, which can be composed by rocks or geotextiles around the circumference of the pile [42].

Monopiles principal characteristics [43]:

Advantages:

- simple design compared to the other types of support structures
- easy to be manufactured in serial production due to the simple geometry. Also, very convenient to transport and install in series with specialized installation vessels
- technology used for many years that proves the efficiency and cost effective of this solution
- it is possible to be installed in almost all kinds of soils conditions, due to the installation techniques and the shape of the pile, being a versatile solution
- Efficient in transferring the forces from the turbine to the ground

Disadvantages:

- is not so profitable at greater depths due to the huge quantity of steel that is needed, larger diameters, thickness and length, increasing the costs of the structure because the steel is expensive. So, monopile is not the best solution financially for larger scales, but research carries on in order to optimise the monopile to be more economically feasible on greater depths.
- the installation process and the project in deeper waters are more complex, the structure is bigger and heavier due to the stiffness that is needed, requiring huge machinery for the implementation.
- After service lifetime, the structure is not totally removed, the standards require the support to be catted 1.5 meters below the seabed. This process over the years will originate the corrosion of the rest of the pile under the seabed level, being dangerous for the sea life, and causing pollution of the sea waters.

2.3.6.1. CHANGING PILE-TOE SHAPE

The monopile's first contact at the seabed level is the pile-toe, so the energy is directly transmitted to the ground through this point. The idea to change the pile-toe shape is to reduce the resistance of the penetration force into the ground soil, reducing also the applied energy to push the pile into the ground. This question was inspired in medical syringes, the needle's shape was changed to have less resistance force [43].

The costs of the manufactured pile shape will increase, but the installation process costs will be cheaper, as less energy is needed during the implementation process, and it also means less production of noise by the hydraulic hammer.

The noise and the vibrations produced by the construction equipments is very relevant because it affects the sea life, the marine ecology, so in these days, studies are done in order to minimize these problems. Figure 2-24 shows the introduction of three possible types of bevel used in needles that can be also utilized in piles:

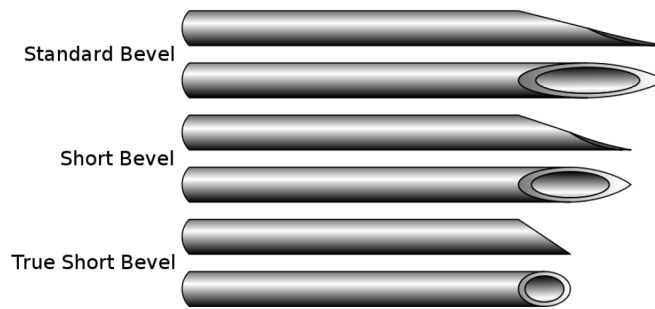


Figure 2- 24: Different bevels used in needles [43]

2.3.6.2. MONOPILE MANUFACTURING AND INSTALLATION PROCESS

The manufacturing process of the monopile is resumed in two phases, rolling and welding, being illustrated on the figures below:



Figure 2- 25: Steel storage, [52]



Figure 2- 26: Rolling process, [52]



Figure 2- 27: Inside welding [53]



Figure 2- 28: Assembly / welding [53]



Figure 2- 29: Outside welding [53]



Figure 2- 30: Ultrasonic testing, [52]

The success of offshore wind energy comes from making the components pre-assembly as much as possible onshore, because every step completed onshore saves time and money during offshore installation processes, and does not depend on offshore wind, wave and weather conditions [29].



Figure 2- 31: Pile delivered to harbour [38]

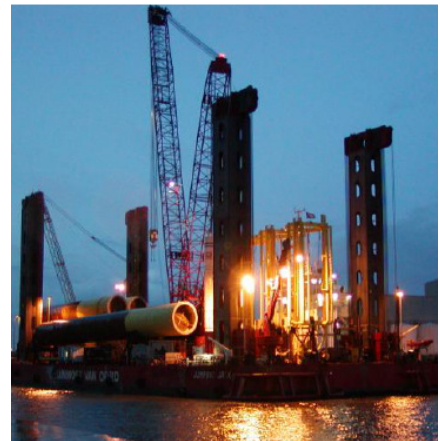


Figure 2- 32: Pile loaded out on jack-up barge [54]

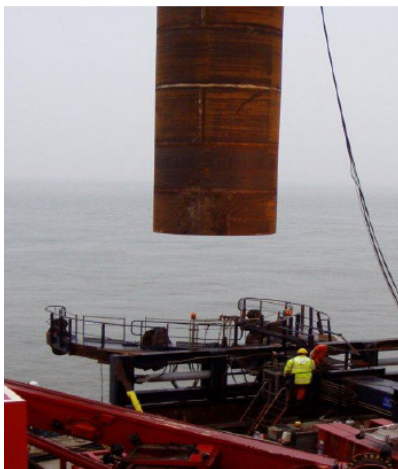


Figure 2- 33: Pile positioning [38]

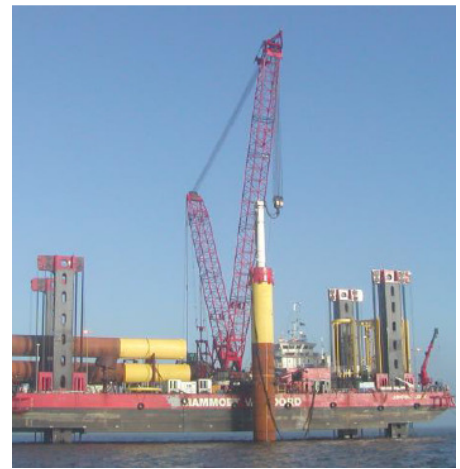


Figure 2- 34: Pile hammer / driving monopile [54]

The monopiles are usually driven into the seabed but, sometimes, drilling may be required in harder ground soils. While it is impossible to drive the pile absolutely vertically, the installation tolerance is normally up to 0.5 degrees out of vertical [38].

The process of installation starts with the construction of the scour protection, then the pile is driven between the middle of the scour protection circumference into the seabed, using vibration and hammering techniques.

Once the pile is in position, the larger diameter transition piece is fitted with a grouting connection over the top of the smaller diameter of the monopile, leaving an annulus for the grout connection to fill. This connection is obtained by the static friction due to the surface roughness of the contact areas. The annulus allows some tolerance that enables verticality correction through hydraulic jacks inside the transition piece [38].

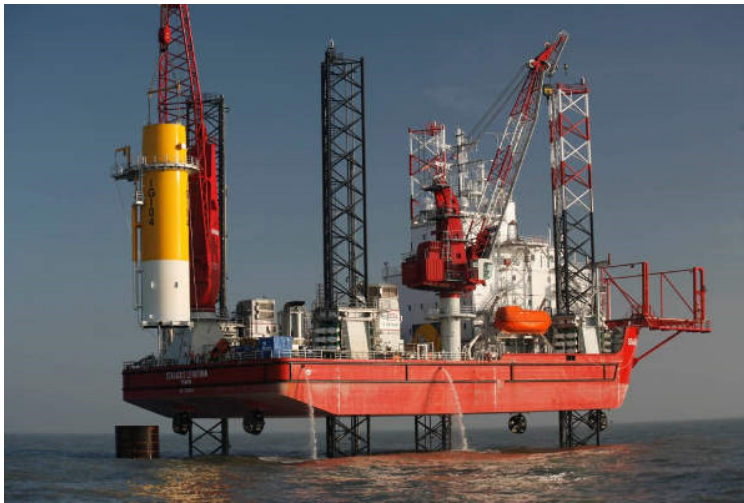


Figure 2- 35: Installation of the transition piece [54]



Figure 2- 36: Positioning of the transition piece [38]



Figure 2- 37: Transition piece with the annulus for the grout connection [38]

The monopile and transition piece are fabricated with a small cone angle in the grouted section. Additional mechanical interlocks in terms of weld beads, so-called shear keys, are adjusted in circumferential direction on the facing surfaces of outer and inner steel tube to increase the load bearing behaviour. This connection was used already in the offshore oil and gas industry, giving the possibility to compensate the inclinations induced by the installation of the driven piles [55].

The design of the grouted connection from the offshore wind turbines is more complex due to the structures that are mainly affected by dynamic actions, unlike the offshore oil and gas platforms, which are loaded by static axial actions, being much easier the design.

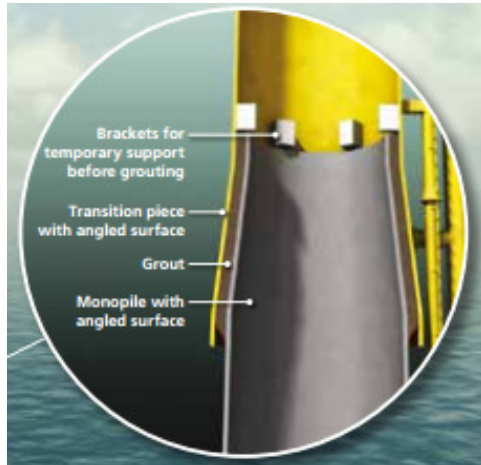


Figure 2- 38: Grouted connection detail, [55]

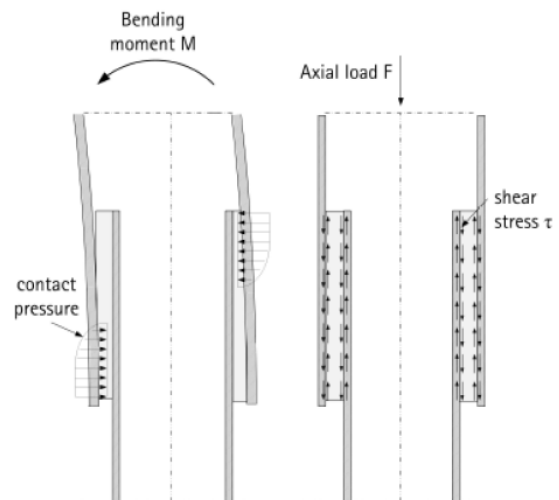


Figure 2- 39: Forces transferred to the grouted connection [38]

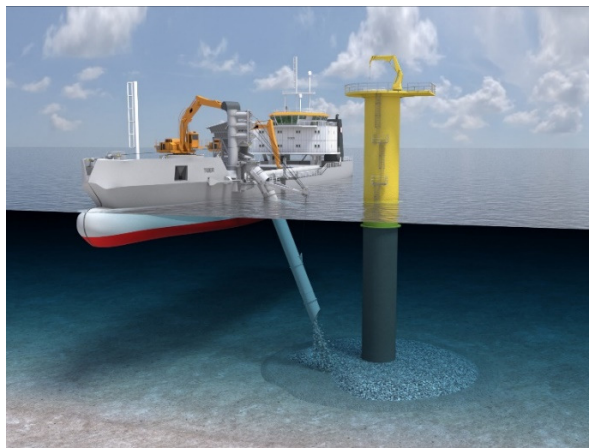


Figure 2- 40: Scour finishing's after monopile installation [56]

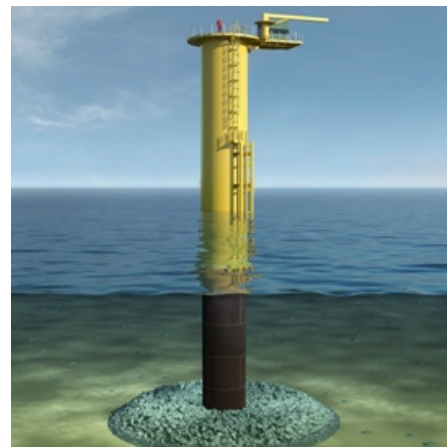


Figure 2- 41: Monopile and transition piece complete installation [38]

After the monopile foundation and the transition piece are in place, the assembly of the wind turbine begins with the installation of the tower above the transition piece platform, which is fitted and bolted into position being assembled in phases.

In order to allow a quick installation at offshore, the wind turbine components must be pre-assembled onshore. The most important components to be pre-assembled are the tower segments and the blades connected to the hub, then it is easier to attach them to the nacelle.

Vessels with high cranes are needed to install the nacelle and the blades. The installation process can only be done in good weather conditions and with low wind speeds, due to the high level of precision that is required.

With all turbine components at site, the installation of the wind turbine takes a minimum of 24 hours [29].



Figure 2- 42: Tower assembled in phases [57]

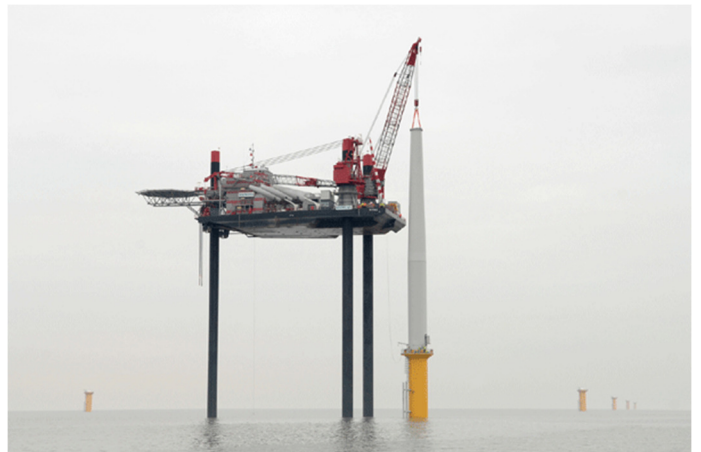


Figure 2- 43: Tower fully installed [57]



Figure 2- 44: Crane installing the nacelle [57]



Figure 2- 45: Jack-up barge preparing to install the blades [57]



Figure 2- 46: Installation of a pre-assembled hub with three blades [57]



Figure 2- 47: Installation complete [54]

In short, for the complete complex construction process of foundations and offshore wind turbines, offshore vessels need [29]:

- sufficient storage capacity for large components
- sufficient height and lifting power
- the ability for a faster positioning and jack-up at the installation site
- the ability to operate in water depths, wave heights and currents

2.4. COMPARATIVE ANALYSIS

After the presentation of all available types of foundations for offshore wind energy production, it is not possible to find the best substructure to use, due to the needed knowledge of the local conditions that it would be installed.

The below statement was discussed in a magazine, Iken, Movement in foundations, 2010, in [43] that says:

“After many years of discussion, it is gradually being accepted that there is no individual foundation type which is equally suitable for all locations.”

Foundation design is one of the major topics in the field of offshore wind turbines due to the large costs involved in the design and fabrication of this kind of structures. The foundation can form around 25% of the overall costs of an offshore wind farm, therefore evaluation of the design could lead to a significant reduction in project cost [7].

Although not being a universal solution, the monopile is the cheapest, most reliable and versatile solution, compared to the other alternatives. Such advantages gain a new and improved scale with the XL monopiles concept [58].

3

Aspects Involved in the Design of Offshore Wind Turbines

3.1 DESIGN PRINCIPLES

According to some of the available standards, the structural project of offshore wind turbine can be done through the following general methodologies of design, such as [16]:

- design by partial safety factors method with linear combination of loads or load effects;
- design by partial safety factor method with direct simulation of combined load effect of; simultaneous load processes;
- design assisted by testing;
- probability-based design.

The DNV standard is based on the partial safety factor method, which is based on separate analysis of the load effect in the structure due to each applied load. Therefore, the structural reliability is guaranteed by the partial safety factors used on the several combinations of actions, verifying that with application of the partial coefficients the limit states are not exceeded.

Separate assessment of the load effects is accurate when the load effects as well as loads are independent. Otherwise, direct simulation of the combined load effects of simultaneously applied load processes is needed. This is demanding for offshore wind turbines structures as the wave and wind loads affects the behaviour of the structure. The dynamic motions due to wave loads cause the turbine see an extra velocity and hence, produce more power. This means wave and wind actions are tightly connected, and the structure should be simultaneously subjected to both loads in a coupled integrated time-domain dynamic analysis [14].

The design assisted by testing can be done to verify the material proprieties due to the fact that the specifications that may be not sufficient for the design of the required structure. The resistance definition of the structural elements can also be evaluated by validating the design process through load trials. This design method is also applicable as a supplement to analytical methods. The use of the IEC standard requires the use of a structural dynamic model to predict the design loads. Relatively to the structural project it is mentioned that it shall be verified that limit states are not exceeded for the wind turbine design and it should be used a testing model as a substitute for calculation to verify the structural design. This is applicable to the other standards two.

The probability-based design is used to special case design problems, such as calibration of the load and material factors to be applied in the partial safety factor method, as well as for cases where limited or uncertainty experience exists [16].

Structures and structural components shall be designed to [16]:

- sustain loads liable to occur during all temporary, operating and damaged conditions if required;
- ensure acceptable safety of structure during the design life of the structure;
- maintain acceptable safety for personnel and environment;

- have adequate durability against deterioration during the design life of the structures;
- possess an adequate ductile resistance;
- minimise stress concentrations on structural connections.

3.1.1. LIMIT STATES

An offshore wind turbine must be designed and comply with the following limit states:

- Ultimate Limit State (ULS): corresponds to the maximum load-carrying resistance or capacity of the structure supporting actions and influences that during its implementation and lifespan may occur;
- Serviceability Limit State (SLS): corresponds to tolerance criteria applicable to normal use or ability of the structure to remain able to the use that is required. The SLS for offshore steel structures are associated to the deflections and vibrations of the structure which may cause deformations of the same.
- Fatigue Limit State (FLS): corresponds to failure of the structure due to the effect of cyclic loading
- Accidental Limit State (ALS): corresponds to the maximum load-carrying capacity for (rare) accidental loads, like fire, explosions and impacts.

3.1.2. STRUCTURAL MODELLING

Structural analysis can be performed based on different models, which allow the prediction of the structural behaviour and to check the various limit states.

Static analysis must be supported on models that consider in an appropriate manner the elastic geometric characteristics of the structure, which takes into account the stiffness characteristics of the structural elements and connections.

The dynamic analysis must be carried out based on a modal analysis methodology. It should be taken into account the energy dissipation capacity of the structure and normally nonlinear material behaviour.

3.1 TYPE OF LOADS

The next points will present the loads that are normally taken into account on the design of offshore wind turbines, based on the standards exposed on subchapter 1.4.

3.1.1. PERMANENT LOADS (G)

Permanent loads are loads that will not vary in magnitude, position or direction during the period considered. This type of loads refer to the mass of the wind turbine (rotor, hub, blades, nacelle and tower), transition piece and monopile. The external and internal hydrostatic pressure, acting on the monopile and transition piece, is also considered a permanent load on the structure.

3.1.2. VARIABLE FUNCTIONAL LOADS (Q)

Variable functional loads, as the name implies, are loads that vary in magnitude, position and direction along the considered period. In offshore wind farm projects, the variable loads are related to the installation operations, ship impacts and maintenance of the wind turbine, so it is necessary to consider the load of the personnel, equipment and materials.

This type of loads were not taken into account on this study due, to the lack of information.

3.1.3. ENVIRONMENTAL LOADS (E)

Environmental loads are caused by environmental phenomena, which can contribute to structural damage. These loads, unlike the permanent ones, may vary in magnitude, position and direction during the period under consideration. They shall be based on the environmental data of the site location in study, for different kinds of return period. These loads are defined as [16]:

- Earthquakes
- Soil Conditions
- Temperature
- Snow and Ice
- Tides
- Marine Growth

Throughout this chapter, the three most important environmental loads for the design of offshore wind turbines will be presented and studied. These are:

- Waves
- Current
- Wind

Due to the limited resources available on time, these ones were considered the most important in terms of a pre-assessment of the design. Therefore it must be taken into account that a proper and accurate design of this complex structure must, by all means, include the analysis of the other extreme environmental loads and events. Nonetheless, the ones hereby considered serve as a good starting point in terms of the preliminary analysis made in this thesis.

3.1.4. ACCIDENTAL LOADS

Accidental loads are related to technical failure or abnormal operations, caused by:

- dropped objects;
- collision impact from vessel, helicopter or other objects;
- fire;
- explosions;
- load from rare, large breaking wave.

For the design of the present wind turbine these loads were not considered, mainly due to the lack of time associated to this project, but also due to the number of accidental events that should be considered, in order to provide an accurate analysis. Besides that, for example, in terms of vessels' collisions or explosions, it is often very difficult to find available data to support a reasonable evaluation of such loads. Usually, since offshore wind farms comprise considerable amounts of investments and a very high level of competition, between the stakeholders and companies encompassed in this economic activity, the existent data about collapses or impacts in the serviceability of the structure become practically unknown. Note that the companies are not interested in providing information about structures which were designed by them. However, interesting works such as Storheim M. & Amdahl J. [59] have recently been performed in the field of accidental loads, providing new models for prediction and analysis of these cases.

3.1.5. DEFORMATION LOADS (D)

Deformation loads are caused by unwanted events that the structure is subjected to, such as:

- settlement of the support structure and its foundation due to the deformations of the soil;
- temperature loads, where the structures must be designed for the most extreme temperature differences they may be exposed to.

The deformation loads were not considered also for the study of these wind farms for similar reasons to ones given in the previous section. Once again, the data regarding temperatures and also regarding soils

conditions is often rare. Some records on this variables are available, for specific locations, in worldwide known data bases, as 4COffshore or LORC. However, the available information comes at very high financial costs, which weren't supported by the present master thesis research. Another thing that must be noted is the fact that the analysis about the generalized or differential settlements of the structure is dependent on the soils' conditions and the soils' structure, i.e. if it is well compact, if it is organized in sedimentary layers or as a mixture of different sediments. Besides that, the soils' cohesion and liquefaction phenomenon must also be analysed. Despite the data that was used in further analysis of the soils' stiffness (subchapter 4.8) the analysis of the structure settlement wasn't the main purpose of this research. Nevertheless, if such analysis is required, particularly for complex soils' configurations, interesting works on soils-structure interaction can be consulted in the literature, e.g. Ferradosa T. (2012) [13].

3.1.6. DYNAMIC LOADS

Dynamic loads are the ones applied to the structure due to an effect caused by an excitation of cyclic nature. These loads tend to generate a structural response which is a dynamic one, hence producing vibrations that can cause the serviceability damage or the total collapse. The excitation of a wind turbine that leads to a dynamic response might be caused by the following events:

- waves;
- wind;
- earthquake;
- rotor' modes of operation.

Some of these events/loads will be further analysed and discussed, in the next subchapters. However, complex phenomena, such as earthquakes, weren't studied, since they could lead to a whole new research topic. For detailed analysis of dynamic responses and dynamic loads in an offshore wind turbine further consultancy of specialised literature is suggested beyond the considerations made in the present thesis.

3.2 QUANTIFICATION OF ENVIRONMENTAL LOADS

3.2.1. WAVES

Waves occur on the free surface of the sea resulting from the wind blowing over a certain area of the ocean, the random of the sea waves is showed in Figure 3-1. To obtain the wave parameters, it is necessary to make some measurements, so if we focus in one point of the sea, measuring the surface elevation in time, it will result in a graph like Figure 3-2, and the variation of the surface elevation during that time can be transformed into an energy density spectrum. Hence, for the determination of the spectral energy density of the sea elevation on a certain site location, the Pierson-Moskowitz and JONSWAP in Figure 3-3 are the two most commonly used spectra, which try to reproduce the actual measured spectra at the respective site location under specific circumstances [60].



Figure 3- 1: Sea Surface [6]

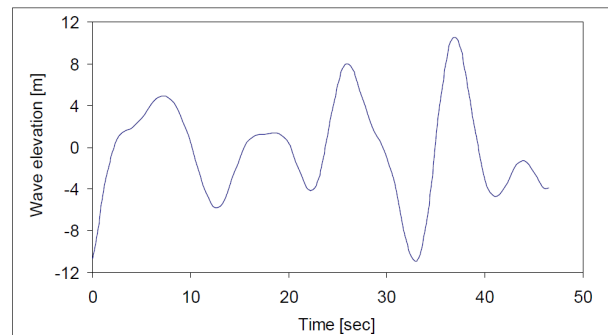


Figure 3- 2: Time recording of sea surface elevation, [61]

In order to evaluate the spectral energy density of the sea elevation, below are presented the two most used spectra, with their respective expressions:

JONSWAP Spectrum [16]:

$$S(f) = \frac{\alpha g^2}{(2\pi)^4} \times f^{-5} \exp\left(-\frac{5}{4}\left(\frac{f}{f_p}\right)^{-4}\right) \gamma \exp\left(-0,5\left(\frac{f-f_p}{\sigma \times f_p}\right)^2\right) \quad (3.1)$$

With:

f: Wave frequency (f=1/T) [Hz]

T: Wave period [s]

f_p: Spectral peak frequency (f_p=1/T_p) [Hz]

T_p: Peak period [s]

g: Acceleration of gravity [m/s²]

α: Generalised Phillips' constant = 5.(H_s².f_p⁴/g²). (1-0,287.ln γ).π⁴ (3.2)

σ: Spectral width parameter =0,07 for f ≤ f_p and σ = 0.09 for f > f_p

γ: Peak-enhancement factor

$$\gamma = \begin{cases} 5 & \text{for } \frac{T_p}{\sqrt{H_s}} \leq 3,6 \\ \exp\left(5,75 - 1,15 \frac{T_p}{\sqrt{H_s}}\right) & \text{for } 3,6 < \frac{T_p}{\sqrt{H_s}} \leq 5 \\ 1 & \text{for } 5 < \frac{T_p}{\sqrt{H_s}} \end{cases} \quad (3.3)$$

Pierson-Moskowitz Spectrum [60]:

$$S_{PM}(f) = \frac{5}{16} \times \frac{H_s^2}{T_p^4 \times f^5} \exp\left(-\frac{5}{4}(f \times T_p)^{-4}\right) \quad (3.4)$$

With:

H_s: Significant wave height [m]

T_p: Peak period [s]

f: Wave frequency [Hz]

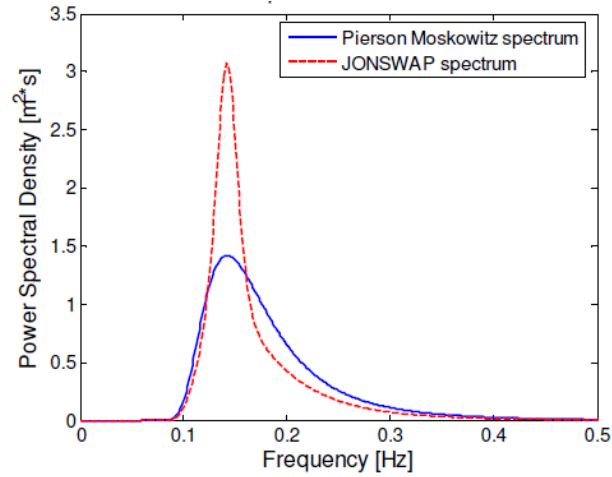
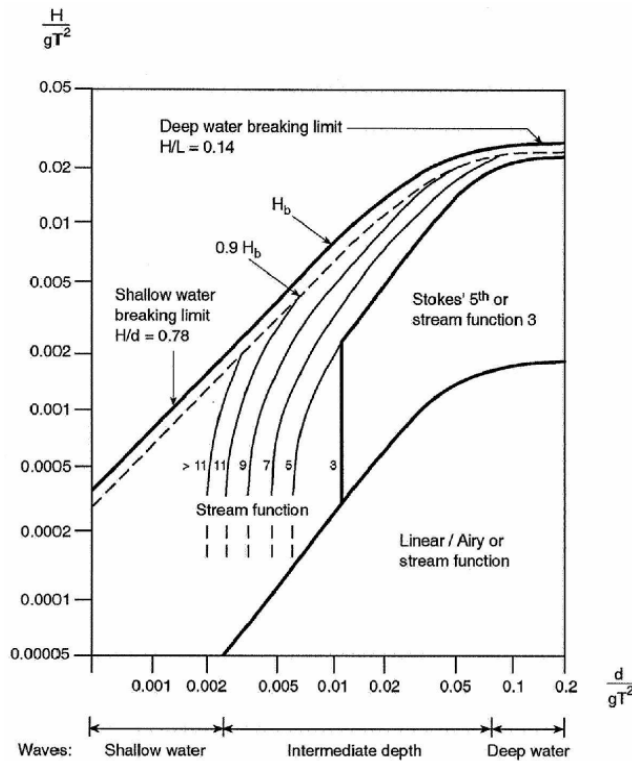


Figure 3- 3: Wave Pierson-Moskowitz and JONSWAP spectrum of measured time recording of sea surface elevation [60]

Waves do not have a regular direction of propagation. The wave shape is irregular, causing changes on the sea crests and troughs on the water surface. This demonstrates the nonlinearity description of the water-wave, so it is difficult and complex to obtain a mathematical characterization. After all, two theories of water waves have been developed to solve this nonlinearity problem, the Airy linear wave theory in 1845 and the Stokes nonlinear wave theory in 1880 [62].

Following the standards, it is possible to choose the correct theory to use for a particular offshore design location, this selection is defined by the curves in Figure 3-4:



$\frac{H}{gT^2}$: Dimensionless wave steepness

$\frac{d}{gT^2}$: Dimensionless relative depth

d: Mean water depth [m]

T: Wave period [s]

H: Wave height [m]

g: Acceleration of gravity [m/s^2]

Figure 3- 4: Regions of applicability of wave theories, [16]

From the regions of applicability of wave theories shown in Figure 3-4, the Airy linear theory is the simplest one, however, it is the most used by the designer engineers.

In cases where the wave height is small compared to the water depth, the linear Airy wave theory can be used. This considers a sinusoidal shape of the surface elevation and circular movements of the water particles, varying over depth [63].

3.2.1.1. LINEAR AIRY WAVE THEORY

For every harmonic wave, the motion of the water particle can be described by linear wave theory according to Airy, which gives a linear description of the propagation of gravity waves on sea surfaces [60]. Airy linear wave theory is the most used and straightforward way of calculating water particle velocities and accelerations due to waves [64]. Figure 3-5 shows the positive x-direction of the wave propagation.

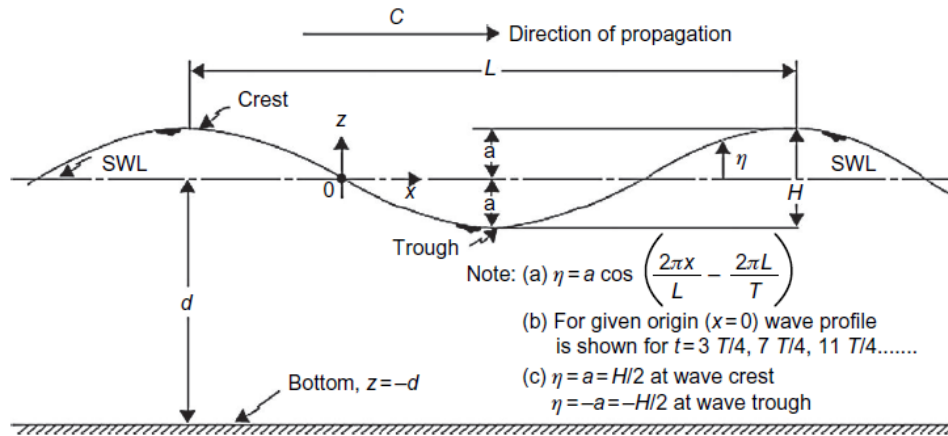


Figure 3- 5: Parameters of a sinusoidal progressive curve, [62]

- Mean water level (MWL) is equal to mean sea level (MSL) and still water level (SWL);
- a : Wave amplitude;
- η : Sea surface elevation.

With the wave circular frequency (ω), the wave number (K) and the wave height (H), the surface elevation can be expressed as follows:

$$h = \frac{H}{2} \times \cos(kx - \omega t) = a \times \cos(\theta) \quad (3.5)$$

Regarding the water particle displacement shown in Figure 3-6, in deep waters the water particles are moving in circles in accordance with the harmonic wave propagation, and the diameter of the circles reduces with the increasing of the water depth. In intermediate water depth and shallow waters, the particles move in elliptical motion due to the effect of the seabed that transforms the circular motion into an elliptic motion.

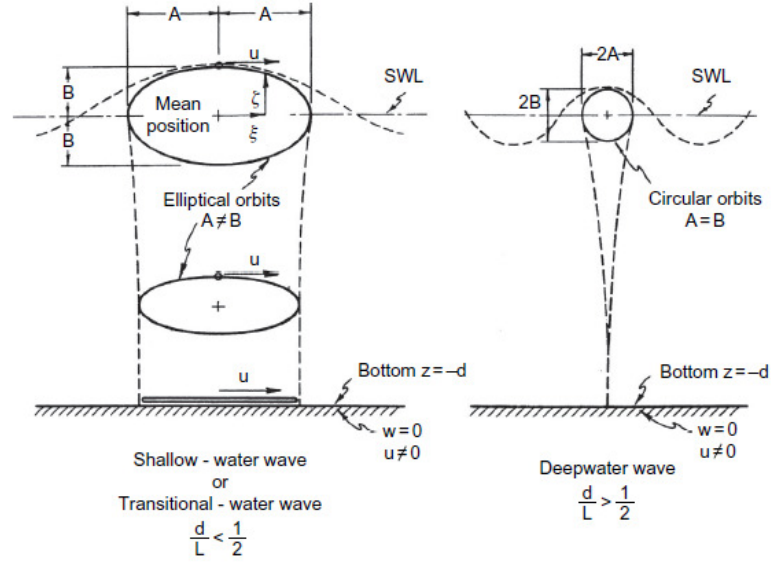


Figure 3- 6: Water particle orbits according to Airy theory, [62]

3.2.1.2. WAVE PARTICLE KINEMATICS

Airy theory is only applicable for intermediate depth waters and deep waters, as shown in Figure 3-4. The wave propagation is considered positive along the x-axis and the origin is located at the sea surface with the z-axis positive upwards, as it presented in Figure 3-5. The vertical water particle kinematics are not relevant for the wave calculations because the foundation structure does not have any inclined or horizontal members, only a vertical pile. The horizontal water particle is the most important for the design of the monopile, and it will be described below [61]:

For intermediate depth waters where $\frac{d}{L} < \frac{1}{2}$:

The horizontal particle velocity is given by:

$$u(x, z, t) = \frac{a \cdot w \cdot \cosh(k(z + d))}{\sinh(k \cdot d)} \cos(\theta) \quad [\text{m/s}] \quad (3.6)$$

The horizontal particle acceleration is given by:

$$\dot{u}(x, z, t) = \frac{a \cdot w^2 \cdot \cosh(k(z + d))}{\sinh(k \cdot d)} \sin(\theta) \quad [\text{m/s}^2] \quad (3.7)$$

The hydrodynamic pressure is given by:

$$p(x, z, t) = -\rho \cdot g \cdot z + \rho \cdot g \cdot a \cdot \frac{\cosh(k(z+d))}{\cosh(k \cdot d)} \sin(\theta) \quad [\text{N/m}^2] \quad (3.8)$$

The first term is the static part and the second term is the dynamic contribution.

For waters depth where $\frac{d}{L} > \frac{1}{2}$:

The horizontal particle velocity is given by:

$$u(x, z, t) = a \cdot w \cdot e^{k \cdot z} \cdot \cos(\theta) \quad [\text{m/s}] \quad (3.9)$$

The horizontal particle acceleration is given by:

$$\dot{u}(x, z, t) = a \cdot w^2 \cdot e^{k \cdot z} \cdot \sin(\theta) \quad [\text{m/s}^2] \quad (3.10)$$

The hydrodynamic pressure is given by:

$$p(x, z, t) = -\rho \cdot g \cdot z + \rho \cdot g \cdot a \cdot e^{-k \cdot z} \cdot \sin(\theta) \quad [\text{N/m}^2] \quad (3.11)$$

The first term is the static part and the second term is the dynamic contribution associated with the hydrodynamic effects.

With:

w: Wave angular frequency [rads/s]

k: Wave number [-]

Θ : Phase angle [rad]

Z: Desired depth [m]

ρ : Water density [kg/m^3]

d: Water depth [m]

a: Wave amplitude

g: Gravity acceleration [m/s^2]

At Figure 3-7 is presents the directions of the water particles' velocities and accelerations for the most common phase angles:

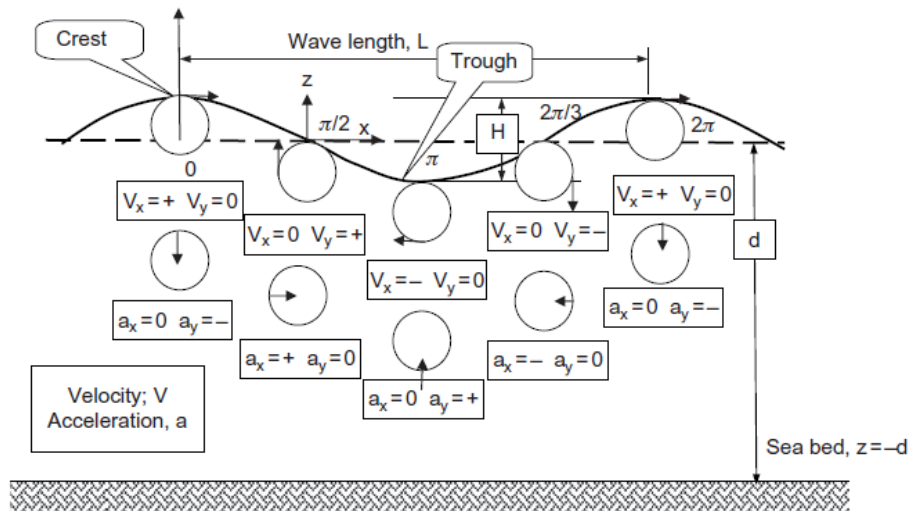


Figure 3- 7: Directions of the water particles' parameters, [62]

Figure 3-8 shows the distribution of the water pressure at the wave crest and wave trough along the water depth:

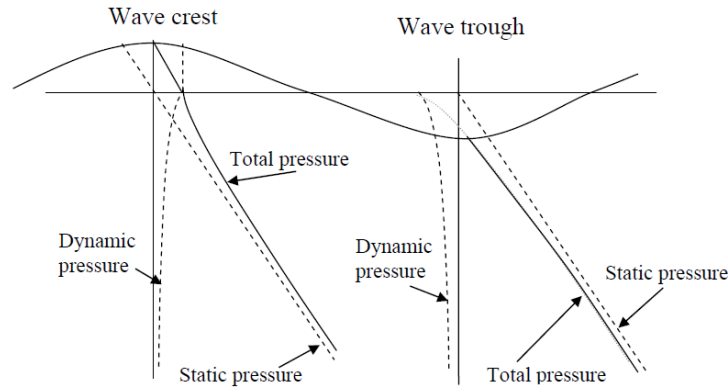


Figure 3- 8: Hydrodynamic pressures in wave crests and wave troughs, [61]

3.2.1.3. WAVE PARAMETERS

The design of any structure in the ocean environment depends on the sea elevation (wave height), the direction of the waves, wave length and the wave period. Below, are exposed the expressions used to define these wave parameters:

Wavelength:
$$L = \frac{g}{2\pi} \cdot T^2 \cdot \tanh\left(\frac{2\pi \cdot d}{L}\right) \quad [\text{m}] \quad (3.12)$$

Wave amplitude:
$$a = \frac{H}{2} \quad [\text{m}] \quad (3.13)$$

Wave celerity:
$$C = \frac{L}{T} = \frac{W}{K} \quad [\text{m/s}] \quad (3.14)$$

Wave angular frequency:
$$\omega = \frac{2\pi}{T} \quad [\text{rad/s}] \quad (3.15)$$

Wave frequency:
$$f = \frac{1}{T} \quad [\text{Hz}] \quad (3.16)$$

Wave number:
$$k = \frac{2\pi}{L} \quad [\text{m}^{-1}] \quad (3.17)$$

3.2.1.4. PHASE ANGLE

The phase angle is necessary in the equations of the water particle velocity, acceleration and pressure. This angle may vary between 0 radians and 2π radians. The angle that is considered is the one which causes the higher stresses (shear force, bending moment and pressure). Figure 3-8 demonstrates the development of the phase angle with the time domain of the wave:

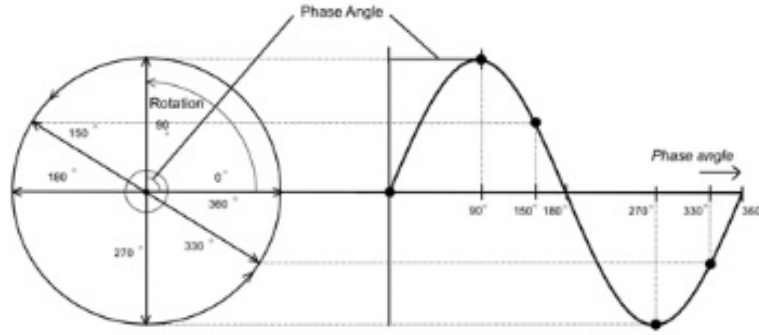


Figure 3- 9: Rotating phasor and sinusoidal waveform in the time domain [65]

$$\theta = k \cdot x - \omega \cdot t \quad [\text{rad}] \quad (3.18)$$

3.2.1.5. HYDRODYNAMIC FORCES DEFINITION

The wave particle kinematics from the Airy theory can now be used in the Morison Equation in order to calculate the loads on the structure, the forces (F) and moments (M), which will be presented in subchapter 3.2.1.8. This equation is an empirical formula, and can be applied to determine the hydrodynamic loads per unit length on slender members, such as jacket structure components and monopile structures [64]. The Morison equation is presented below:

Total hydrodynamic force (Morison force):

$$f(x, z, t) = f_d(x, z, t) + f_i(x, z, t) \quad [\text{N/m}] \quad (3.19)$$

Hydrodynamic drag force:

$$f_d(x, z, t) = C_d \cdot \frac{1}{2} \cdot \rho \cdot D \cdot |u(x, z, t)| \cdot u(x, z, t) \quad [\text{N/m}] \quad (3.20)$$

Hydrodynamic inertia force:

$$f_i(x, z, t) = C_m \cdot \frac{\rho \cdot \pi \cdot D^2}{4} \cdot \dot{u}(x, z, t) \quad [\text{N/m}] \quad (3.21)$$

Where:

C_d : Hydrodynamic drag coefficient [-]

C_m : Hydrodynamic inertia coefficient [-]

ρ : Water density [kg/m^3]

D : Pile diameter [m]

Following the DNV-OS-J101 [16], the drag and inertia coefficients depend on the Reynolds number (Re), the Keulegan-Carpenter number (KC) and on the relative roughness (K). For a vertical cylinder pile with diameter D, the Reynolds number and the Keulegan-Carpenter number are defined as:

$$Re = \frac{u_{max} \cdot D}{\nu} \quad (3.22)$$

$$KC = \frac{u_{max} \cdot T}{D} \quad (3.23)$$

Where:

u_{max} : Maximum particle velocity at still water level [m/s]

T: Wave period [s]

D: Pile diameter [m]

ν : Kinematic viscosity of seawater [$m^2 s^{-1}$]

The drag coefficient C_{DS} for steady-state flow depends on the roughness of the structural member surface and can be used as a basis for determination of C_d and C_m . The drag coefficient C_{DS} for steady-state flow can be obtained as:

$$C_{DS} = \begin{cases} 0,65 & \text{for } \frac{k}{D} < 10^{-4} \text{ (smooth)} \\ \frac{29 + 4 \cdot \log_{10} \left(\frac{k}{D} \right)}{20} & \text{for } 10^{-4} < \frac{k}{D} < 10^{-2} \\ 1,05 & \text{for } \frac{k}{D} > 10^{-2} \text{ (rough)} \end{cases} \quad (3.24)$$

With:

D: Structural member diameter [m]

K: Surface roughness [m]

New uncoated steel and painted steel can be assumed as smooth, for concrete and highly rusted steel, $K = 0.003$ m, and for marine growth, $k = 0.005$ to 0.05 m.

The drag coefficient C_d depends on C_{DS} and on the KC number being calculated as:

$$C_D = C_{DS} \cdot \psi(C_{DS}, KC) \quad (3.25)$$

The wake amplification factor ψ can be evaluated through the graph in Figure 3-10. For intermediate cylinder surface roughness between smooth and rough, linear interpolation is allowed between the smooth curve and rough curve.

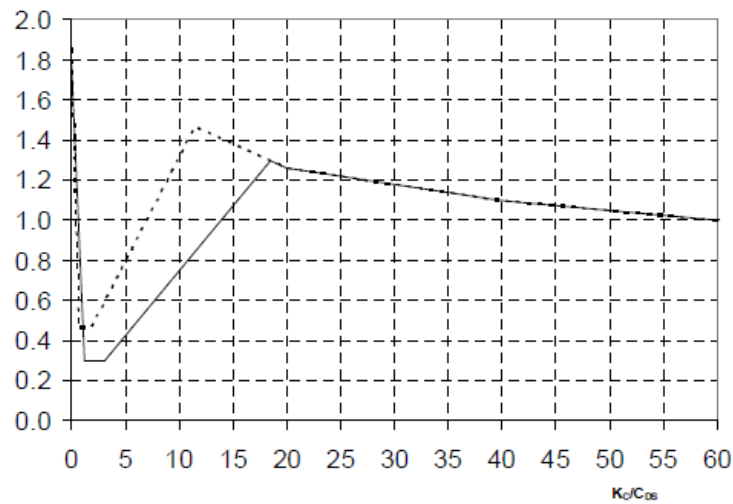


Figure 3- 10: Wake amplification factor as function of KC number for smooth (solid line) and rough (dotted line), [16]

The inertia coefficient follows the criteria below:

For $KC < 3 \longrightarrow C_m = 2.0$

For $KC > 3 \longrightarrow C_m = \max\{2 - 0.044(KC - 3); 1.6 - (C_{DS} - 0.65)\} \quad (3.26)$

To evaluate the wave-induced loads on offshore structures, it may be important to verify the viscous effects and potential flow effects. The potential flow includes the wave diffraction and radiation around the structure. Figure 3-11 can be used to verify how relevant the viscous and potential effects are. This figure is established for horizontal wave-induced forces on a vertical cylindrical pile, which stands on the seabed and penetrates the free water surface [16].

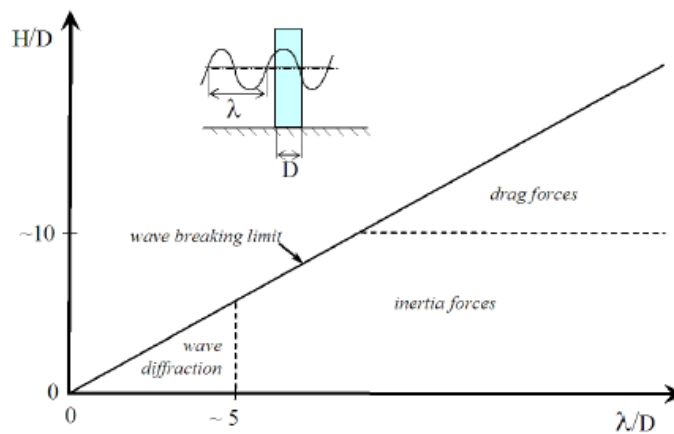


Figure 3- 11: Relative importance of wave forces on marine structures, [16]

When the dimension of the structure is large compared with the wave length, typically when $D > 0.2\lambda$, Morison's equation is not valid. The inertia force will be dominating and can be predicted by diffraction theory [16].

3.2.1.6. CURRENT

The current consists of wind-generated current velocity at still water level and a tidal current velocity also at still water level.

The tidal current is originated by the combined effects of gravitational forces exerted by the Moon, Sun and rotation of the planet, leading to the rise and fall of the sea levels. The wind-generated currents are caused by wind stress and atmospheric pressures.

The calculation process of the current velocity is based on the DNV-OS-J101 [16], standard and it is presented below:

Total current velocity at level (z):

$$v(z) = v_{tide}(z) + v_{wind}(z) \text{ [m/s]} \quad (3. 27)$$

Tidal current velocity at level (z):

$$v_{tide}(z) = v_{tide0} \times \left(\frac{h+z}{h} \right)^{1/7} \text{ [m/s]} \quad (3. 28)$$

Wind-generated current velocity at level (z):

$$v_{wind}(z) = v_{wind0} \times \left(\frac{h_0 + z}{h_0} \right) \text{ [m/s]} \quad (3. 29)$$

Wind-generated current at still water level may be evaluated by the following expression:

$$v_{wind0} = k \times U_0 \quad (3. 30)$$

Where:

K: 0,015 to 0,03

U_0 : 1-hour mean wind speed at 10 m height [m/s]

h: Water depth from still water level (taken as positive) [m]

h_0 : Reference depth from wind-generated current, $h_0 = 50$ [m]

v_{wind0} : Wind-generated current at still water level [m/s]

v_{tide0} : Tidal current at still water level [m/s]

Z: Vertical coordinate from still water level, positive upwards [m]

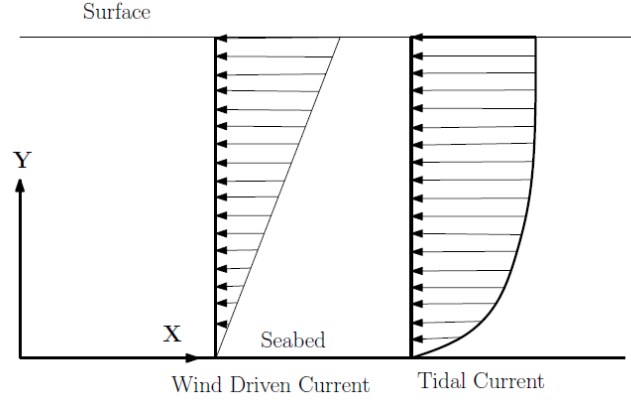


Figure 3- 12: Current profiles, [65]

In order to determine the current force, the current velocity must be incorporated in the calculation of the total hydrodynamic load in the Morison equation. The velocity is only a parameter on the drag force expression, so the equation will be taken as:

$$f_d(x, z, t) = C_d \cdot \frac{1}{2} \cdot \rho \cdot D \cdot |u(x, z, t) + U_c| \cdot (u(x, z, t) + U_c) \quad [\text{N/m}] \quad (3.31)$$

With:

U_c : Current velocity [m/s]

u : Horizontal particle velocity [m/s]

3.2.1.7. BREAKING WAVES

Breaking wave loads on structures is still an area where much uncertainty remains. The probability of breaking waves is relatively small on a nearly horizontal seabed with normal bathymetrical characteristics. Most waves break at or near the coast, not within the offshore wind farm [60].

Usually, waves break when $H/d > 0,78$. The breaking wave load can be calculated through the following equation [60]:

$$F = \frac{1}{2} \times \rho \times C_s \times A \times u^2 \quad [\text{N}] \quad (3.32)$$

With:

C_s : Slamming coefficient [-]

ρ : Water density [kg/m^3]

A : Exposed area to the breaking wave [m^2]

u : Water particle velocity at breaking wave crest [m/s]

3.2.1.8. WAVE LOADS QUANTIFICATION

The total hydrodynamic shear force (F) and bending moment (M) are obtained through an analytical solution by integrating the drag force and inertia force from the Morison equation, showed in subchapter 3.2.1.5, from the seabed with $z=-d$ until the instantaneous water surface elevation (η). It is taken as [64]:

$$F(x, t) = \int_{-d}^h \{f_i(x, z, t) + f_d(x, z, t)\} dz \quad [\text{N}] \quad (3.33)$$

$$M(x, t) = \int_{-d}^h \{f_i(x, z, t) + f_d(x, z, t)\} \times (d + z) dz \quad [\text{N.m}] \quad (3.34)$$

It is also possible to determine the wave loads through a simplified method considering an integration from the sea floor until the still water level ($z=0$). This simplification does not interfere in the inertia force, which reaches a maximum when the wave surface has zero crossing, but it discards the additional wave drag load during the passage of the wave crest. However, this effect can be significant if drag force dominates [64]. Below are shown the calculation equations for this simplified method:

$$\hat{F}_I = \rho g \frac{C_m \pi D^2}{4} \hat{\zeta} \cdot \tanh kd \quad (3.35)$$

$$\hat{F}_D = \rho g \frac{C_d D}{2} \hat{\zeta}^2 \cdot \left[\frac{1}{2} + \frac{kd}{\sinh 2kd} \right] \quad (3.36)$$

$$\hat{M}_I = \rho g \frac{C_m \pi D^2}{4} \hat{\zeta} \cdot d \left[\tanh kd + \frac{1}{kd} \left(\frac{1}{\cosh kd} - 1 \right) \right] \quad (3.37)$$

$$\hat{M}_D = \rho g \frac{C_d D}{2} \hat{\zeta}^2 \cdot \left[\frac{d}{2} + \frac{2(kd)^2 + 1 - \cosh 2kd}{4k \sinh 2kd} \right] \quad (3.38)$$

Figure 3-13 shows the shape of the wave and the current load on the monopile:

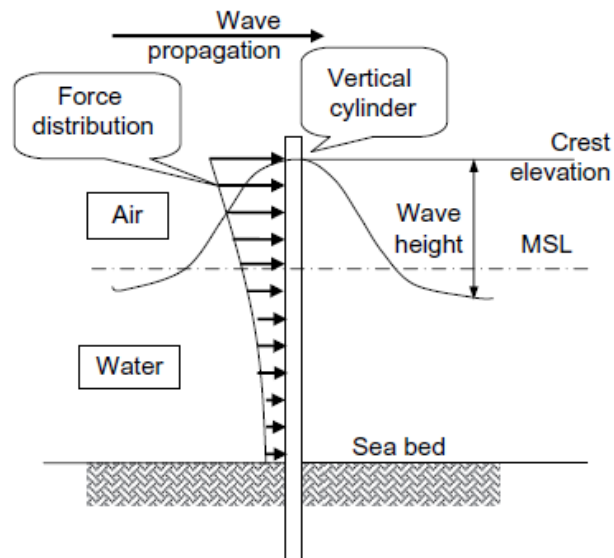


Figure 3- 13: Hydrodynamic loads on a slender member, [62]

Figure 3-14 sums up all the calculation processes in order to determine the hydrodynamic forces on the monopile. It mentions the components that are needed in the linear wave theory to evaluate the water particle velocity and acceleration, like it was presented in subchapter 3.2.1.2. After obtaining these water particle results, the Morison equation is applied to calculate the total hydrodynamic force, shown in subchapter 3.2.1.5. At the end, to obtain the total hydrodynamic shear force and bending moment at the seabed level, where the efforts are higher, the equations 3.33 and 3.34 are implemented, which makes an integration from the seabed level until the instantaneous water surface elevation.

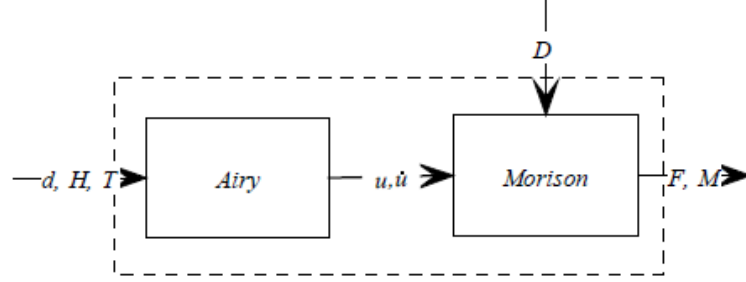


Figure 3- 14: Process to evaluate the total hydrodynamic force and bending moment, [64]

3.2.2. WIND

The wind speed and the wind direction vary in space and time. Offshore wind farms have an advantage over onshore projects due to the higher wind speeds available over the sea.

The measured wind speed magnitude and its variation in time can be transformed into an energy density spectrum, which indicates how the energy of the wind turbulence is distributed between different frequencies. Figure 3-15 shows the two most used spectra, the von Karman spectrum, which has a good description of turbulence in wind tunnels, and the Kaimal spectrum, that fits empirical observations in atmospheric turbulence better [66]. These spectra are based on the 10-minute mean wind speed, the turbulence intensity and a length scale [16]:

Kaimal Spectrum:
$$S_U(f) = \sigma_U^2 \frac{4 \times \frac{L_k}{U_{10}}}{\left(1 + 6 \times \frac{f \times L_K}{U_{10}}\right)^{\frac{5}{3}}} \quad (3.39)$$

Von Karman Spectrum:
$$S_U(f) = \sigma_U^2 \frac{4 \times \frac{L_k}{U_{10}}}{\left(1 + 70,8 \times \left(\frac{f \times L_K}{U_{10}}\right)^2\right)^{\frac{5}{6}}} \quad (3.40)$$

With:

σ_U : Standard deviation of the wind speed [m/s]

L_k : Integral length scale parameter [m]

$$L_k = \begin{cases} 5,67 \cdot z & \text{for } z < 60 \text{ m} \\ 340,2 \text{ m} & \text{for } z \geq 60 \text{ m} \end{cases} \quad (3.41)$$

U_{10} : 10-minute mean wind speed [m/s]

f : Wind frequency [Hz]

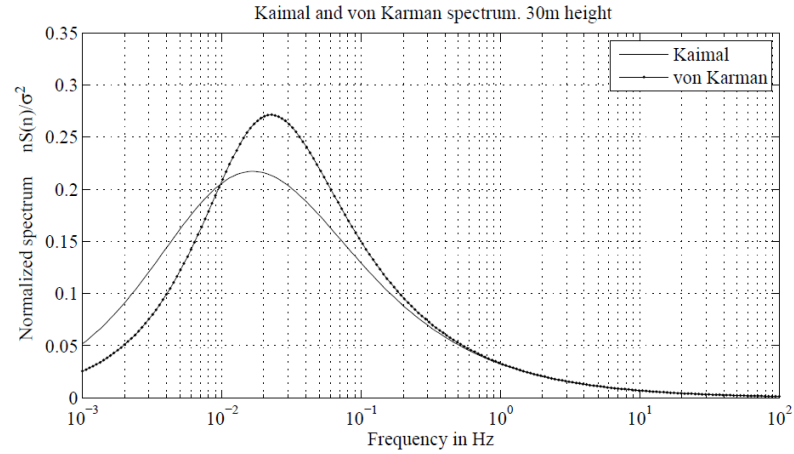


Figure 3- 15: Kaimal and von Karman spectrum, [66]

In the wind industry, it is very important to obtain the information of the variation of the wind speeds. Designers need this type of information to optimise and minimise the costs of their wind projects, and the investors in the wind energy sector need this kind of information to predict their income from electricity generation. The most common way to describe this wind variation is through the Weibull wind distribution, which is a probability density function. It is used in connection with wind resource assessment as a parametric description of the distribution of the 10-minute mean wind speed (U_{10}). The Weibull distribution is defined through two parameters, the scale parameter (A) and the shape parameter (K). Using them, the probability of exceeding the 10-minute mean wind speed in a certain 10 minute period may be calculated as [67]:

$$\text{Distribution function:} \quad F(U_{10}) = 1 - \exp\left(-\left(\frac{U_{10}}{A}\right)^k\right) \quad (3.42)$$

In Figure 3-16, it is possible to observe that at offshore locations the wind speed values are bigger than at onshore locations:

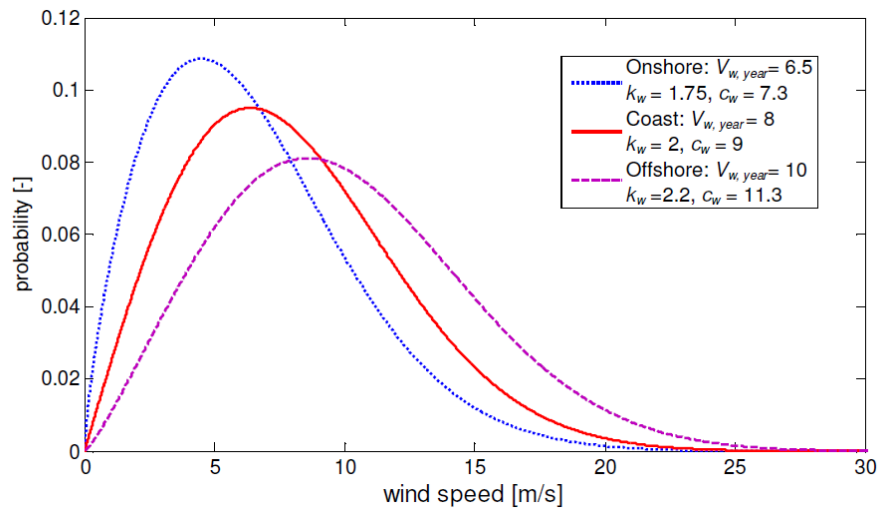


Figure 3- 16: Weibull annual wind distribution for onshore, coast and offshore locations [60]



Figure 3- 17: Mast M6 at Horns Rev 1 wind farm park, [68]

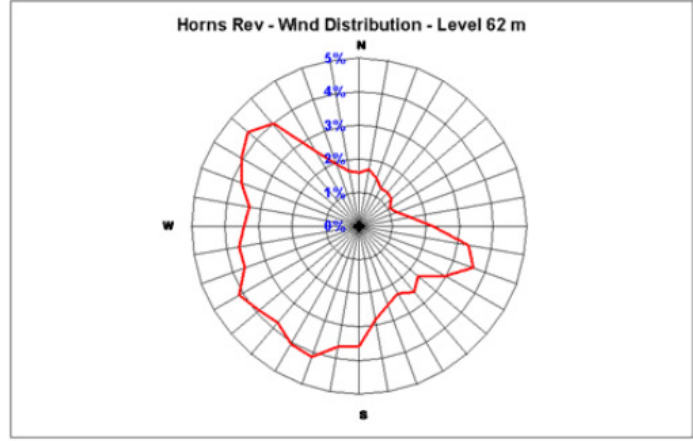


Figure 3- 18: Wind rose distribution on mast M2 at 62 m height from water level in Horns Rev 1 wind farm park, [68]

The masts are built in lattice structures and are instrumented with anemometers, wind vanes as well as sensors to measure the temperature, relative humidity, pressure, irradiation and rain [68].

The wind rose is a graphic tool used to observe how wind direction and speed are distributed at a particular location, being very useful for a quick view of a particular site.

The DNV offshore standard [16], makes a distinction between normal wind conditions and extreme wind conditions. The normal wind conditions method are used to evaluate the primary fatigue loads in standard structural loading conditions, while the extreme wind conditions method refers to an extreme environment which originates extreme loads on the wind turbine structure and foundation support. So, for the offshore foundation design, it is recommended to use the extreme wind conditions method because it is the most conservative method [69].

The wind speed varies with time and altitude, increasing with the high above sea surface. For this reason, the DNV considers the 10 minutes mean wind speed at the height of 10 meters, and the 1 hour mean wind speed also at 10 meters high, for the wind calculation formulas presented in the standard.

These expressions are recognised as the Froya wind profile, and are used to determine the wind speed at any requested height and time for offshore locations, taken as [16]:

Froya wind profile for normal wind conditions:

$$U(T, z) = U_{10} \times \left[1 + 0,137 \times \ln \left(\frac{z}{h} \right) - 0,047 \times \ln \left(\frac{T}{T_{10}} \right) \right] \quad [\text{m/s}] \quad (3.43)$$

Where:

U_{10} : 10 minute mean wind speed at 10 meters height

h : 10 meters

T_{10} : 10 minutes

z : Requested height above sea level

T : Requested time

Froya wind profile for extreme wind conditions and strong gusts, considering the wind turbulence (IU):

$$C = 5,73 \times 10^{-2} \times \sqrt{1 + 0,15 \times U_0} \quad [\text{m/s}] \quad (3.44)$$

$$I_U = 0,06 \times (1 + 0,043 \times U_0) \times \left(\frac{z}{h}\right)^{-0,22} \quad [\text{m/s}] \quad (3.45)$$

$$U(T, z) = U_0 \times \left\{1 + C \times \ln\left(\frac{z}{h}\right)\right\} \times \left\{1 - 0,41 \times I_U \times \ln\left(\frac{T}{T_0}\right)\right\} \quad [\text{m/s}] \quad (3.46)$$

With:

U_0 : 1 hour mean wind speed at 10 meters height [m/s]

h : 10 meters

T_0 : 3600 seconds and $T < T_0$

z : Requested height above sea level

T : Requested time

As DNV is a Norwegian standard, also known as NORSOK, the formula for normal and extreme wind conditions are based on data from the North Sea and Norwegian Sea locations, so it may not give reasonable values when applied to other offshore locations and they should not be used for heights above 100m [16].

3.2.3. AERODYNAMIC LOADS AND CONCEPTS

This subchapter studies the aerodynamic loads and loads effects on the main parts of a wind turbine, such as the blades and tower, explaining also some important concepts for this type of project.

Wind turbine loads result from the operation of the wind turbine, as well as the direct wind-generated loads on the rotor when it is completely stopped. This consists of the aerodynamic loads that must be determined.

3.2.3.1. WIND FORCE ON THE TOWER

After the determination of the wind speed at every requested height, it is possible to calculate the wind force on the tower and on the transition piece according to the expression below, from the API standard [18]:

$$F = \frac{1}{2} \times \rho \times C_s \times A \times U^2 \quad [\text{N}] \quad (3.47)$$

Where:

ρ : Air density [kg/m^3]

C_s : Shape coefficient (see DNV 30.5) [-]

A : Projected area of the member, normal to force direction [m^2]

U : Wind velocity, according to Froya equations [m/s]

3.2.3.2. WIND FORCE ON THE ROTOR

The kinetic energy available from the wind cannot be totally extracted and transformed in mechanical energy by the wind turbine, so part of this energy is expelled back to the environment. This is explained through the Betz theorem, which defines a power coefficient (C_p) that characterizes the performance level of a wind turbine [70].

Betz theorem was published in the year of 1919 by the German physicist Albert Betz, and it determines the maximum power that can be extracted from the wind, independent of the wind turbine design in open flow. The turbine is represented by a uniform “actuator disk” which creates a discontinuity of pressure in the stream tube of air flowing through it, showed in Figure 3-19:

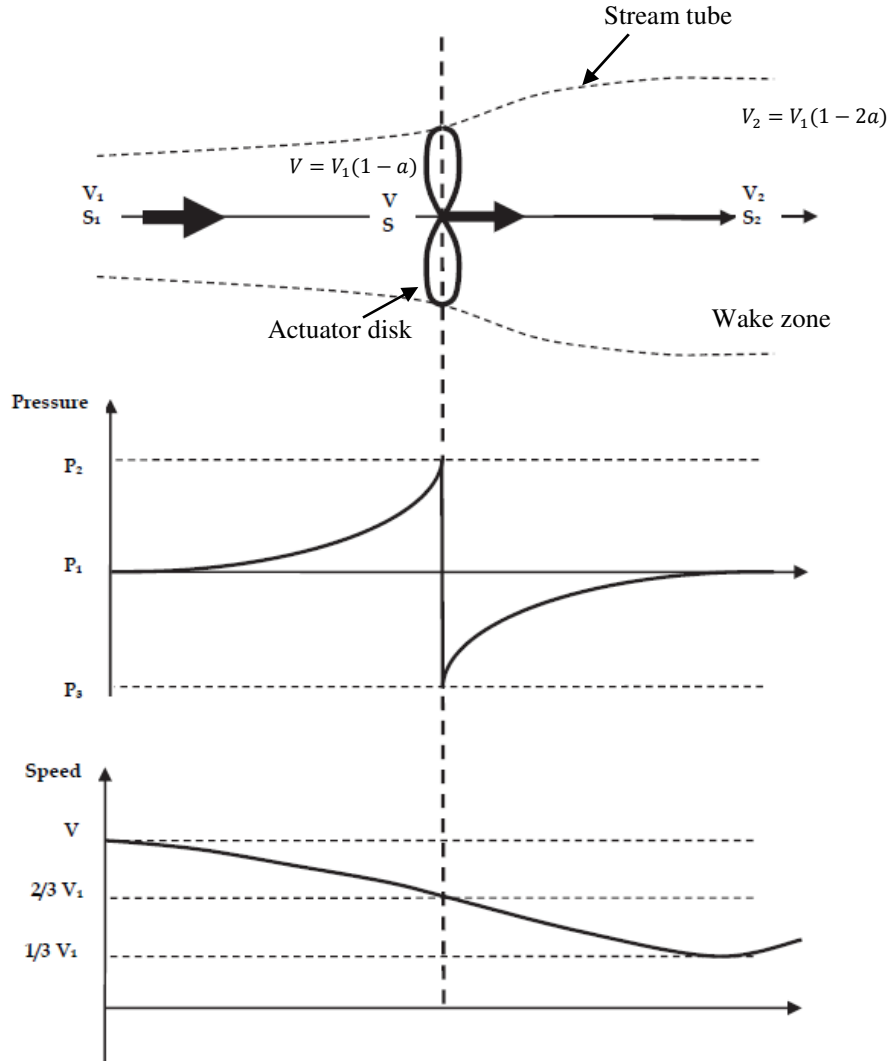


Figure 3- 19: Wind pressure and speed variation in an ideal wind turbine model, [70]

In figure 3-16, V_1 is the acting wind speed at the turbine rotor, before being disturbed by the turbine (upstream), and V_2 is the wind velocity after the passage through the turbine (downstream). It is possible to observe that $V_1 > V_2$, which means that there was a loss of wind speed, due to the transformation of kinetic energy into mechanical energy.

The decrease of the wind speed between the upstream position and the downstream position of the rotor is defined as the axial induction factor (a) and it is characterized by the following equation:

$$a = \frac{V_1 - V}{V_1} \quad (3.48)$$

At this point, we are able to define the wind force load on the rotor during operation mode as:

$$F_{Thrust} = \frac{1}{2} \times A \times \rho \times V^2 \times 4 \times a \times (1 - a) \quad [\text{N}] \quad (3.49)$$

With:

A: Rotor area [m^2]

ρ : Air density [kg/m^3]

V: Acting wind speed on the turbine rotor [m/s]

a: Axial induction factor [-]

Normally, in case of safety, the aero-generator stop wind speed is at 25 m/s, so when the wind speeds exceed this velocity, an existing security system acts, and through quick electromagnetic impulses, the rotation of the rotor is stopped slowly in order to do not induce significant stresses on the structure [14]. After this, the wind turbine remains stopped and is therefore necessary to know, which is the most unfavourable rotor position, because it has an influence on the tower design. So, according to studies already done, one of the most unfavourable positions is presented in Figure 3-20, and it will be this rotor position that is going to be considered along this study [32].

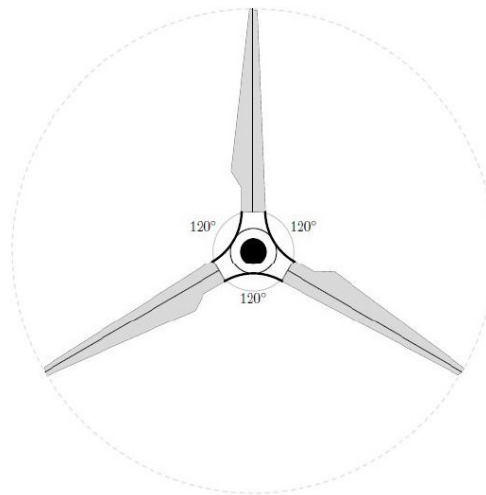


Figure 3- 20: Most unfavourable position of the rotor, [32]

3.2.3.3. AERODYNAMIC LOADS ON THE BLADES

The geometry of the rotor blades determines the amount of energy that is extracted from the wind at a given speed. Figure 3-21 and Figure 3-22 show the forces present in the cross-section area of the rotor blade.

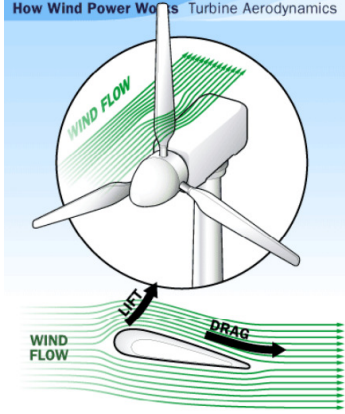


Figure 3- 21: Wind flow through a turbine blades, [71]

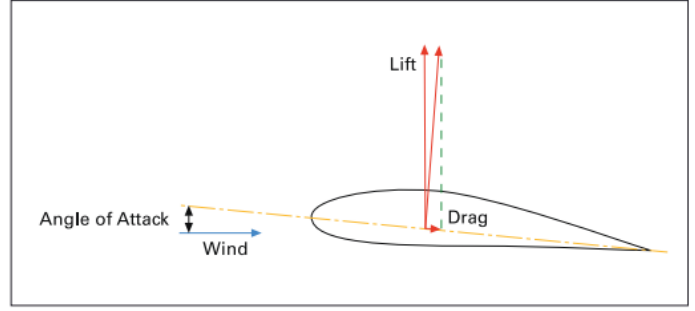


Figure 3- 22: Forces on a stationary rotor blade [73]

The aerodynamic forces presented on the blades consist of the lift and drag forces. The drag force should be minimal to a turbine with a horizontal axis, because it reduces the rotation velocity of the rotor, converting most of the kinetic energy of the wind pressure on the blade surface in the direction of the wind flow [72].

In order to obtain a higher rotational velocity of the rotor, it is necessary that the blades generate more lift force, which depends on the attacking angle, but a very large angle of attack the blade “stalls” and the lift decreases. This is explained by the wind flow passing over the blade at the top, since it will travel a bigger distance than at the bottom for the same period of time, so with an increase of the wind velocity, the dynamic pressure on top the blade is lower, showed this aspects in Figure 3-24.

At Figure 3-23 is represented the system forces and respective angles:

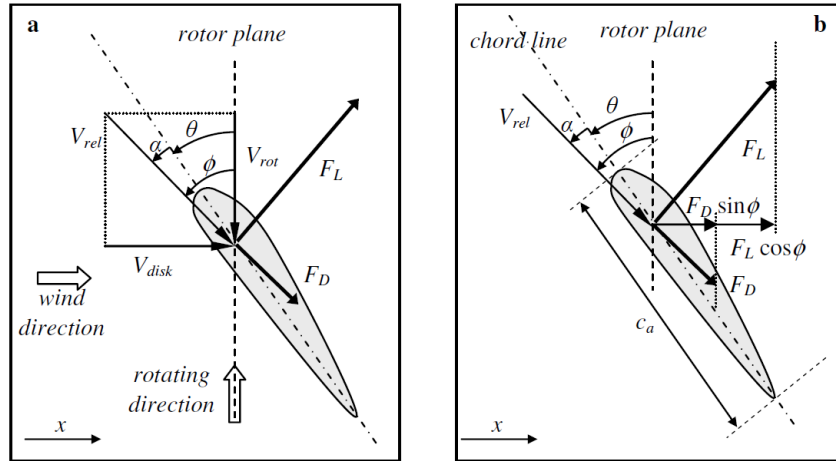


Figure 3- 23: a) System of forces acting on the blade; b) Resulting lift and drag loads in the x-axis direction, [60]

Aerodynamic lift force:

$$F_L = \frac{1}{2} \times C_L \times \rho \times V_{Rel}^2 \times c_a \times \Delta_r \quad [N] \quad (3. 50)$$

Aerodynamic drag force:

$$F_D = \frac{1}{2} \times C_D \times \rho \times V_{Rel}^2 \times c_a \times \Delta_r \quad [N] \quad (3. 51)$$

Where:

C_L : Aerodynamic lift coefficient [-]

C_D : Aerodynamic drag coefficient [-]

ρ : Air density [kg/m^3]

c_a : Airfoil chord length [m]

Δ_r : Radial length of the blade [m]

α : Attack angle [deg]

Θ : Pitch angle [deg]

$\emptyset$: Inflow angle [deg]

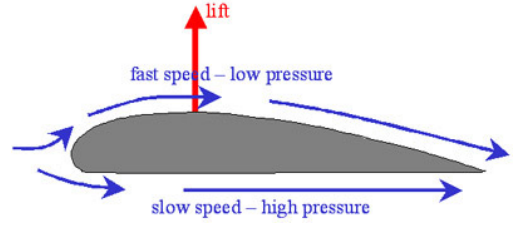


Figure 3- 24: Bernoulli Effect on the cross-section area of the rotor blade [73]

The lift force is perpendicular to the air stream caused by the Bernoulli Effect which reduces the pressure on top of the airfoil compared to the pressure at its bottom. The curvature above originates a higher stream velocity than at the bottom and therefore a lower pressure. The lift and the drag forces vary with the angle that the rotor blade makes with the direction of the air stream denominated as the attack angle [73].

The aerodynamic lift and drag coefficient, are obtained through tests on wind tunnels which provide a graphic representation that depends on the Reynolds number and the attack angle. These graphic representations must be given by the manufacturer [4].

The loads on the blade are generated by the lift and drag forces, which are induced by the wind speed, and the rotation speed that results from the relative wind speed over the blade, like it shows in Figure 3-23. The relative wind speed at the blade section is calculated as:

$$V_{Rel} = \sqrt{V_{disk}^2 + V_{Rot}^2} \quad (3. 52)$$

$$V_{Rot} = \Omega \times r \quad (3. 53)$$

$$\Omega = \frac{W}{2\pi} \quad (3. 54)$$

$$W = 2\pi f$$

$$f = \frac{\text{Rotor speed}}{\text{Time}} \quad (3. 55)$$

With:

V_{disk} : Wind velocity at airfoil [m/s]

V_{Rot} : Linear rotation speed at a blade section [m/s]

Ω : Angular rotation speed of the rotor [rad/s]

r : Radius of the rotor [m]

f : Rotor frequency [Hz]

The behaviour of the blades as a function of the attack angle can be divided into three operating zones, as shown in Table 3-1:

Table 3 - 1: Operation zones of the blades, [4]

Attack Angle	Zone
$30^\circ < \alpha < 90^\circ$	Linear
$-15^\circ < \alpha < 15^\circ$	Stall
$15^\circ < \alpha < 30^\circ$	Stopper

The angle of attack is chosen according to the drag and lift coefficients, in order to extract the maximum possible power from the wind. In turbines with an horizontal axis, the blade must be designed to operate with an angle of attack such the relation between the drag and lift force should be minimal, so that the optimum angle of attack is around 10 degrees [72].

To obtain the flow angle (ϕ), the tip speed ratio (λ) is needed, as well as the rotational induction factor (a') that describes the wind turbine flow field, which is due to the rotation flow in the wake zone. The equations are expressed as [74]:

$$\lambda = \frac{\Omega \times R}{V_1} \quad (3.56)$$

$$a' = -\frac{1}{2} + \frac{1}{2} \times \sqrt{1 + \frac{4}{\lambda^2} \times a \times (1 - a)} \quad (3.57)$$

$$\tan \phi = \frac{V_1 \times (1 - a)}{(1 + a') \times w \times r} \quad (3.58)$$

The load in x -axis direction per blade element is:

$$F_x = F_L \times \cos(\phi) + F_D \times \sin(\phi) \quad (3.59)$$

When the turbine is not in operation mode, it is necessary to determine the most unfavourable position of the rotor. Figure 3-23 shows the aerodynamic lift and drag forces required in order to calculate the load per blade element through the equation 3.59.

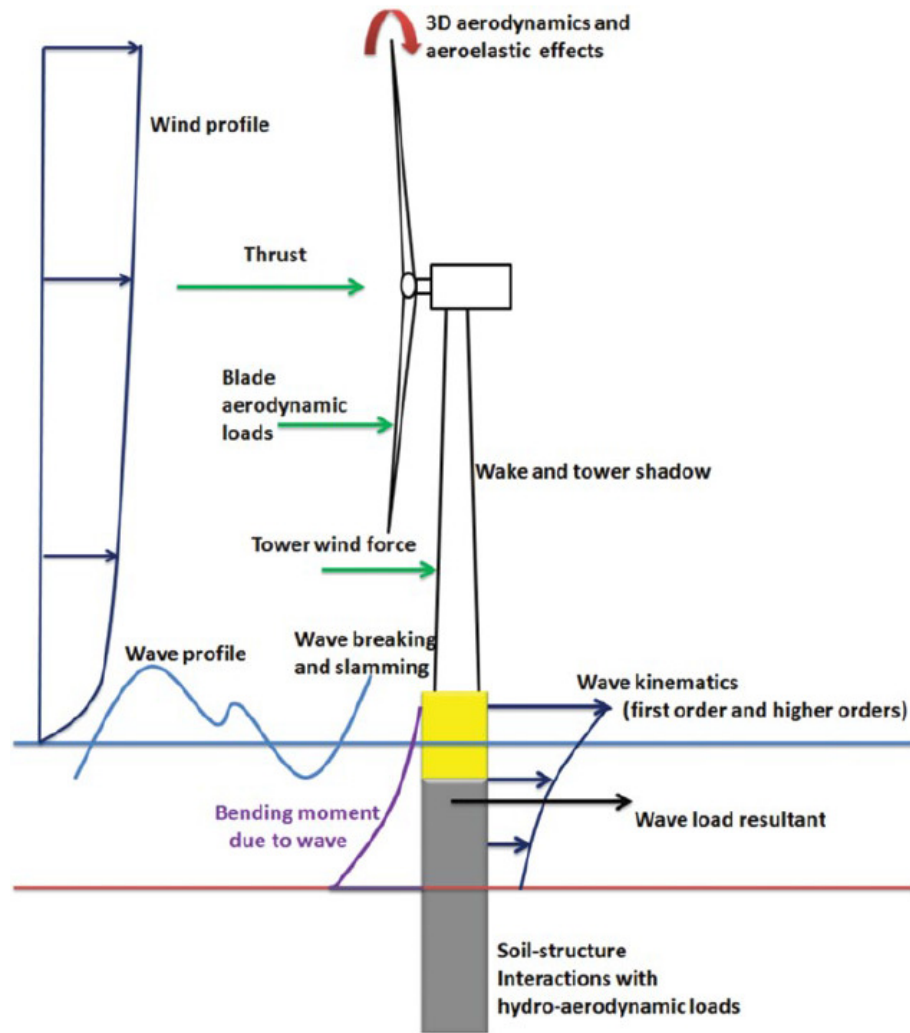


Figure 3- 25: Aero-hydro-dynamic, wind and wave loads for an offshore wind turbine structure with a monopile foundation, [14]

3.2.3.4. TURBINE POWER PRODUCTION

Betz theory states that only less than 59% of the wind kinetic energy can be converted into mechanical energy using a wind turbine. This is explained through the maximum power coefficient (C_p), which is obtained for an axial induction factor equal to $1/3$, so applying equation 3.61, $C_p = 16/27 = 0.593$ [63]. In equation 3.60, C_p is defined as the mechanical power extracted from the wind (P), divided by the total power available in the wind flow that goes through the rotor plane.

$$C_p = \frac{P}{\frac{1}{2} \times \rho \times A \times V_1^3} \quad (3. 60)$$

$$C_p = 4 \times a \times (1 - a)^2 \quad (3. 61)$$

The power coefficient (C_p) can be also presented graphically as a function of the ratio between the upstream and downstream speed, which has a reduction of velocity in $1/3$, due to the passage of the wind through the actuator disk. This can be observed below in Figure 3-26.

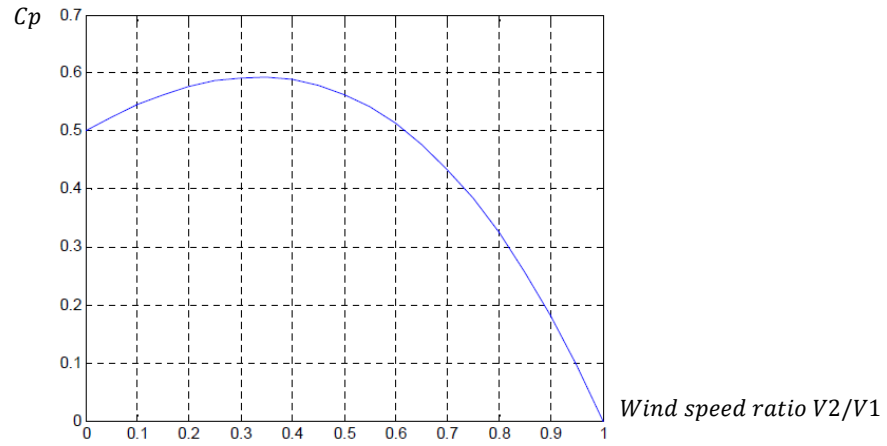


Figure 3- 26: Relation between the power coefficient (C_p) and the wind speed before (V_1) and after (V_2) its passage through the rotor, [72]

The mechanical power captured by the blades of the wind turbine can be calculated by the two following expressions:

$$P = 2 \times \rho \times A_{disk} \times V_1^3 \times a \times (1 - a)^2 \quad (3.62)$$

$$P = \frac{1}{2} \times \rho \times C_p \times A_{disk} \times V_1^3 \quad (3.63)$$

With:

ρ : Air density [kg/m^3]

A_{disk} : Swept area of the wind turbine rotor [m^2]

V_1 : Incident wind speed at the turbine rotor high [m/s]

a : Axial induction factor [-]

3.2.3.5. TURBINE CONTROL SYSTEMS

The blades' shape interfere in the energy produced by the wind turbine, which usually has a set of three blades per turbine, being possible to have an active and passive control of them, in order to operate for a given rotation. The wind turbines have a sophisticated system of control that allows the optimization of energy gains, such as [4]:

- **Cut-in wind speed** – is the minimum wind velocity value for which the wind turbine will start producing energy;
- **Rated wind speed** – is the wind speed value starting from which the wind turbine will produce the nominal power known as the rated power.
- **Cut-out wind speed** - the maximum wind speed which the wind turbine can operate in safety, normally to wind speed values above 25 m/s. The turbine has a system that will brake slowly for security reasons. Otherwise, it can lead to the total collapse of the structure, when the excitation frequency gets close to the natural frequency of the structure, entering in resonance until its total collapse.

After these topics, it is possible to have a better understanding of the power curves for different models of turbines given by the manufactures. Below is presented a possible power curve:

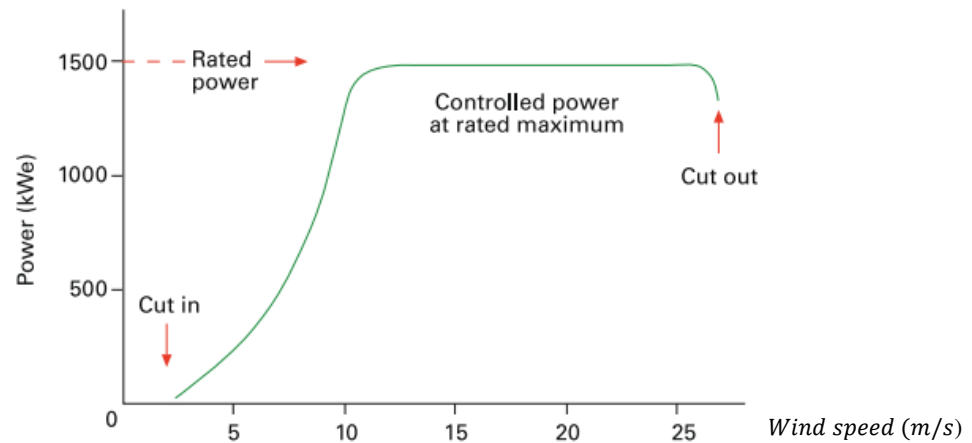


Figure 3- 27: Turbine power curve [14]

If the blades' angle is kept constant, it is impossible for the turbine to respond to changes of the wind speed and wind direction, being unable to maintain an optimum angle of attack to generate the maximum power. So, another type of control systems for wind turbines will be presented below:

- **Stall:** The stall control is a passive system in which the aerodynamic loss of the blades is controlled through the design of the shape blades, generating vortices and thereby increasing the drag force which decreases the angular velocity of the turbine. This passive control only has effect with high speeds, so it is not necessary to vary the pitch angle of the rotor [4].

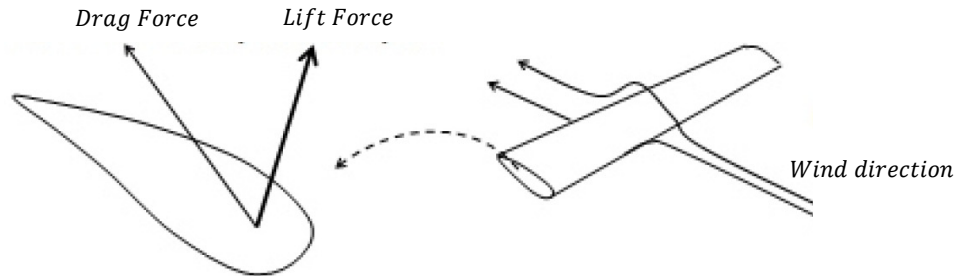


Figure 3- 28: Drag force is higher than the lift force braking the blade, [4]

- **Pitch:** The pitch angle of the rotor can vary through the information given by the system control, positioning the blades perpendicularly to the wind, reducing the aerodynamic lift force and thus the rotor rotation, this being an active way to proceed [30].
- **Yaw orientation system:** The yaw system is a component that is needed to keep the rotor in alignment with the wind, so the use of motors is required in order to make this rotation. Usually, a sensor is mounted above the nacelle with the objective to control whether the rotor is aligned with the wind or not, making the correct adjustment of the rotor [4].

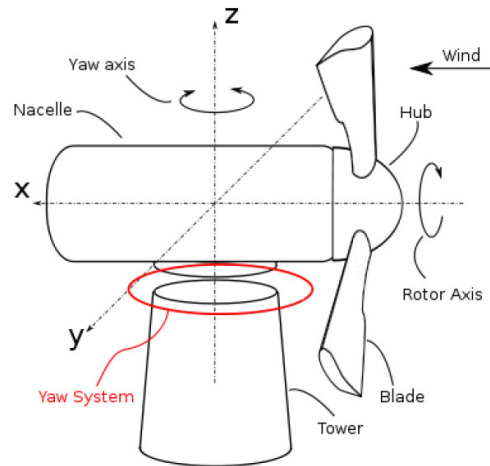


Figure 3- 29: Example of a yaw orientation system, [4]

3.3 WAVE AND WIND LOADS COMBINATIONS

The combinations of wave and wind loads have to be done separately because these forces do not vary together, they have different behaviours over time. This assumption can be made regardless of the intensities and directions of the load processes, and regardless of possible correlations between intensities and between directions [16].

The DNV standard refers two methods for the combination of these two loads:

- Linear combination of wind load and wave load, or of wind load effects and wave load effects
- Combination of wind load and wave load by simulation. This method is based on structural analysis in time domain, for simultaneously applied simulated time series of the wave and wind load.

Applying the linear process, the combined load effect on the structure is determined by combining the individually calculated wave and wind load effects shown in the previous subchapters. This method can be applied to conceptual evaluations, as well as for load calculations for final design. The design combined load is expressed as:

$$S_d = \gamma_{f_1} \times S_{wind,k} + \gamma_{f_2} \times S_{wave,k} \quad (3.64)$$

Where:

γ_{f_1} and γ_{f_2} : Load coefficients for the ULS.

$S_{wind,k}$: Characteristic wind load effect.

$S_{wave,k}$: Characteristic wave load effect.

The two most important limit states that must be verified to ensure the safety and functionality of the structure is the ultimate limit state (ULS) and the serviceability limit state (SLS). The accidental limit state (ALS) and the fatigue limit state (FLS) aren't always verified, depending on the requirements of the structure which is being designed [16].

The present research aimed for the loads' calculation and a pre-assessment of the monopile's design conditions. Therefore, the design of this offshore wind turbine is going to consider the ULS alone, due to the fact that serviceability conditions are lower relatively to the rupture conditions, in ultimate limit

state. Since, the scope of this work was more focused on the first approach to the design methodology, the more refined evaluations and verifications of serviceability, fatigue and accidental limit states are not developed or addressed in this study. However, it is important to note that they represent major aspects of the actual design work that should be performed for a real project situation. Another key aspect that should also be considered is the corrosion phenomena, which becomes particularly evident and important in the monopile's pieces which are alternately under and above the seawater level. For instance, both the transition piece and the metallic joints must have a proper coating in order to resist to the harsh environment caused by the corrosion. This aspect wasn't the purpose of this research, but it is important to take into account in design situations, as stated in DNV standards (2014).

The DNV standard also mentions that for SLS the load factor is equal to 1,0 for all load categories, for temporary and operational design conditions. For the ULS the load factor is higher, being presented in table 3-2. Therefore, the ULS is the worst scenario.

For safety reasons, and to ensure the stability of the structure without reaching the rupture, safety factors are defined for different types of loads, as:

Table 3 - 2: Load factors for the Ultimate Limite State, [16]

Load Factor Set	Limit State	Load Categories			
		Permanent Load (G)	Variable Functional Load (Q)	Environmental Load €	Deformation Load (D)
(a)	ULS	1,25	1,25	0,70	1,00
(b)	ULS	1,00	1,00	1,35	1,00
(c)	ULS for abnormal wind load cases	1,00	1,00	1,10	1,00

3.4 DYNAMIC ACTIONS

Offshore wind farms are quite sensitive to dynamic loading conditions, due to the combination of the slender structural nature of the turbine and the wide range of cyclic loads to which the turbine is subjected [75]. Therefore, the designer has to choose for the project a limit for the global frequency of the entire wind turbine, including the foundation, in order to avoid resonance and the potential fatigue damage. For all dynamic systems, these aspects occur when an excitation frequency gets close to the fundamental natural frequency of the structure. This leads to higher stress in the support structure and foundation, so it is important to ensure that the excitation frequencies (with high energy levels) do not coincide with the natural frequency of the entire structure. For the case of offshore wind turbine structures, the excitation may occur due to wave and wind loads, as well as the frequency generated by the rotor during operation mode.

It is essential to evaluate the first fundamental natural frequency of the offshore wind turbine support structure, since it is an important parameter which conditions its dynamic behaviour.

In a first approximation, it is possible to determine the fundamental natural frequency of the support structure by considering a simplified geometry, as is shown in Figure 3-30. This simplified geometry is similar to a cantilevered vertical column, equivalent to a steel pipe that represents the support structure, with a turbine mass concentrated at the top [35]. So, the first fundamental natural frequency of the complete structure can be estimated as [60]:

$$f_{nat}^2 = \frac{3,04}{4 \times \pi^2} \times \frac{EI}{(0,227 \times \mu \times L + m_{top}) \times L^3} \quad (3.65)$$

With:

f_{nat} : First natural frequency [Hz]

m_{top} : Turbine mass [Kg]

μ : Tower mass per meter [Kg/m]

L : Tower length [m]

EI : Tower bending stiffness [N.m²]

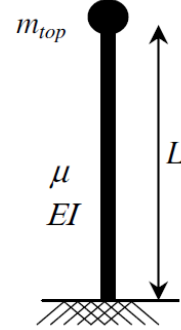


Figure 3- 30: Simplified geometry structural model of a flexible wind turbine system, [60]

Another conservative method to determine the first fundamental natural frequency of the structure is through the equation below:

$$f = \frac{1}{2\pi} \times \sqrt{\frac{K}{M}} \quad (3.66)$$

Where:

f : First natural frequency [Hz]

M : Equivalent mass of the structure (Turbine + Tower + Monopile) [KN]

K : Stiffness of the structure [KN/m]

For a cantilevered vertical column, the stiffness can be calculated as: $K = \frac{3EI}{L^3}$ (3.67)

The primary objective of the analysis in terms of frequencies is to ensure the natural frequency of the structure moves away as much as possible from the frequencies of the external actions that will be subject. The two determinant factors of the analysis of the wave and wind are characterized by having energy peaks at very low frequencies, as show in Figure 3-31:

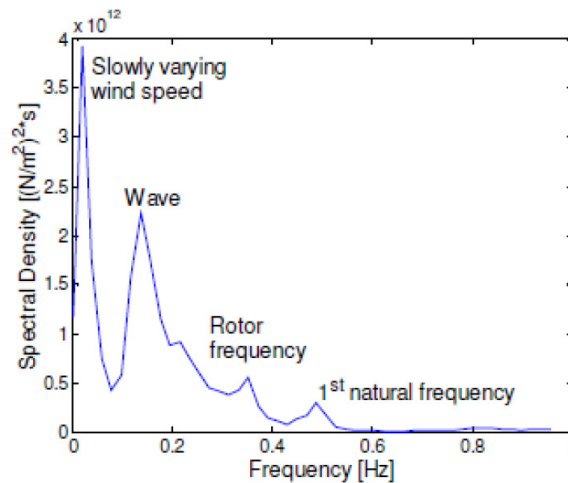


Figure 3- 31: Characteristic values of frequencies for a generic case, [4]

Figure 3-32 shows the typical frequencies of the actions applied to a specific wind turbine system. The “1P” frequency defines the rotational frequency of the turbine during the operation and the “3P” frequency defines the blade-passing frequency. The “3P” frequency results from shadowing effects of the individual blades passing in front of the tower, caused by a drop in the upstream wind velocity. The dynamic wind and wave loading are also shown, where the wind load is characterized by the Froya wind spectrum and the wave load is described by the Pierson-Moskowitz spectrum [75].

In order to avoid resonance effects for the safety and stability of the wind turbines, the support system must be properly designed, so that the magnitude of the dynamic load applied to the structure can be reduced [75].

To put this into practice, three types of design methods can be implemented, which are based on vibration frequency (natural frequency) of the system (structure and foundation). They are explained below [76]:

- Soft-Soft design: where the first natural frequency is placed below the “1P” frequency range, being a very flexible structure and almost impossible to design for a grounded system;
- Soft-Stiff design: where the first natural frequency is between “1P” and “3P” frequency intervals, being the most common in the current offshore construction projects;
- Stiff-Stiff design: where the first natural frequency has a higher natural frequency than the upper limit of the “3P” band, being necessary a very stiff support structure.

The DNV standard mentions that the first fundamental natural frequency should not be within 10% of the “1P” and “3P” intervals, so the desired frequency of the structure should be in the “soft-stiff” range. For this reason, wind turbines with a monopile foundation are designed to be “soft-stiff”, being possible to have a greater flexibility of the support structure (foundation and transition piece) while minimizing the costs associated with the additional material required [75].

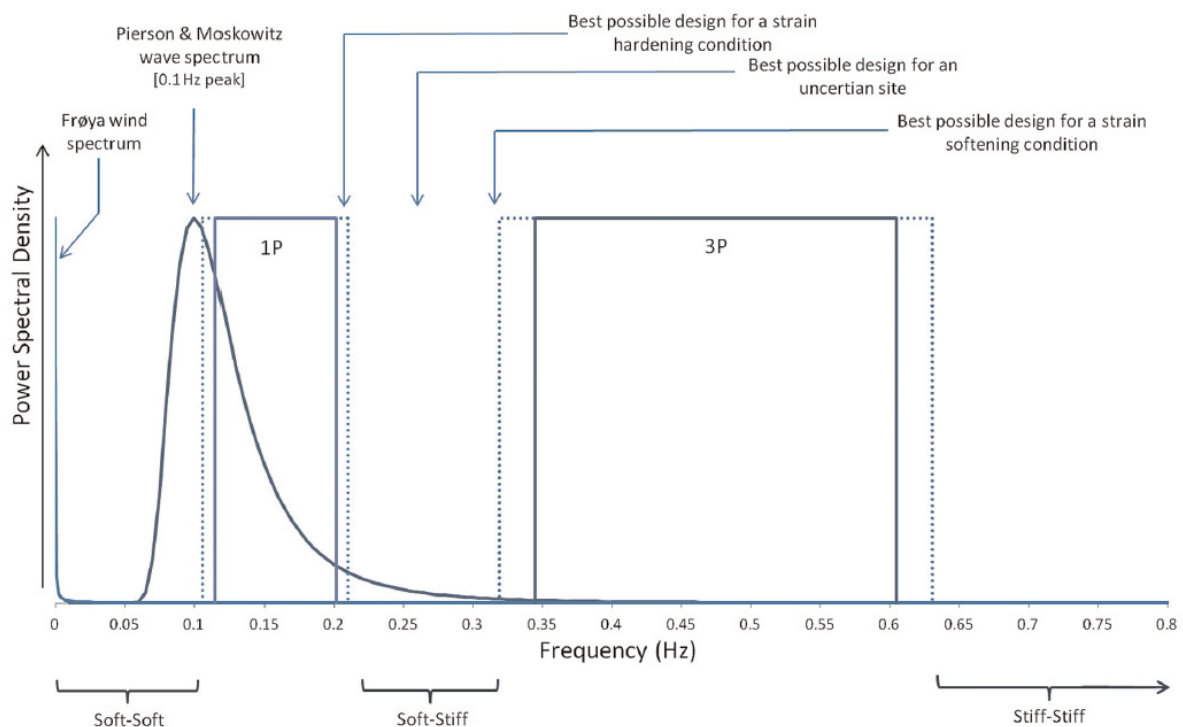


Figure 3- 32: Frequency spectrum of the dynamic loads exhibiting the three design options, [76]

Dynamic amplification and high excitation forces affect monopile foundation supports drastically. Hence, since the rotational speeds of the turbine rotor typically vary between 10 and 20 rpm, the first excitation frequency for one blade “1P” occurs in the interval of 0,17 – 0,33 Hz, and for a turbine with three blades, the blade passing frequency “3P” typically varies between 0,5 – 1,0 Hz [35].

3.5 TOWER TOP DISPLACEMENT AND ROTATION

For the study of the fundamental natural frequency, it was assumed that the structure would be treated as a cantilevered element, so the tower top displacement and rotation are going to be evaluated based on cantilever expressions [16]. Despite this simplified model of the structure and its simplistic calculations, common approaches of finite elements for example can also be applied to the analysis [77].

The wind and wave loads are applied laterally along the height of the tower, then the maximum displacement at the top of the tower can be obtained by [78]:

$$\delta_{max} = \frac{PL^4}{8EI} \quad (3.68)$$

With:

P: Lateral load [KN]

L: Tower length [m]

E: Modulus of elasticity [Kpa]

I: Moment of inertia [m⁴]

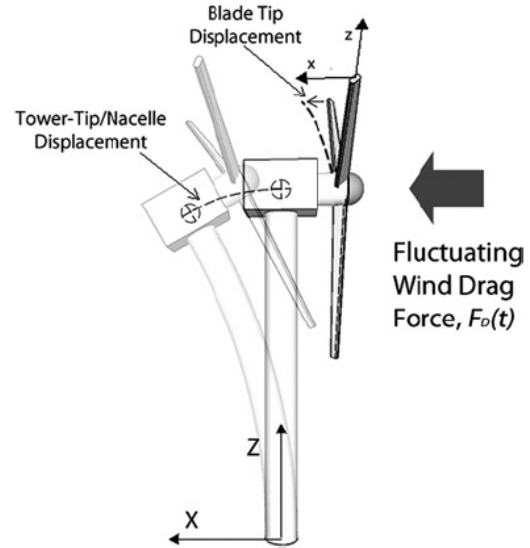


Figure 3- 33: Tower and blade displacement, [79]

According to DNV standard, for cantilevered element the maximum displacement must be less than 1% of the total length of the tower or it can also be determinate with equation (3.68).

$$\delta_{max} < \frac{2L}{200} = 0,01L \quad (3.69)$$

The tower top rotation must be also quantified. During operation mode the blades can collide with the tower, leading to damage in both components, as well as the collapse of the wind turbine. So, the maximum rotation may be calculated as [78]:

$$\theta_{max} = \frac{PL^3}{6EI} \quad (3.70)$$

3.6 CROSS SECTION DESIGN VERIFICATIONS

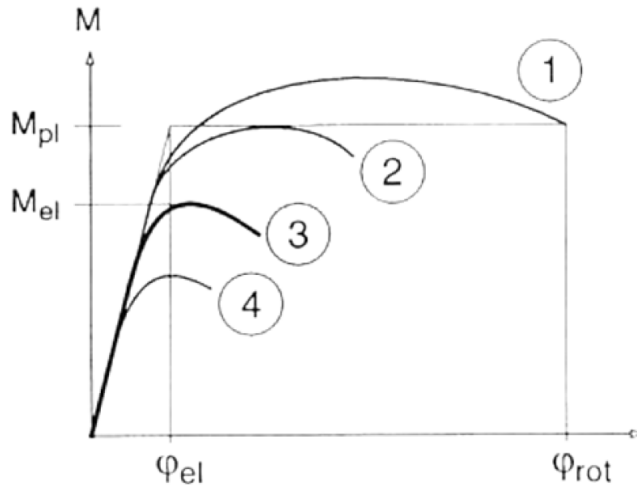
3.6.1. STRUCTURE STABILITY

The design of steel structures in Europe is often performed with Eurocode 3 [23]. It includes rules and regulations applicable to buildings and other steel structures [16].

3.6.1.1. CLASSIFICATION OF CROSS SECTIONS

The classification of the cross sections reflects how the resistance and the ability to deform in bending of one section are influenced by local buckling phenomena. On a slender section the compressed areas may not have capacity to support the compression stress caused by the loads cases and the section can yield completely, but before this happens it may suffer from local buckling phenomena. The EN1993-1-1 [23] classifies the sections through their rotational capacity and ability to form a plastic hinge, this classification is exposed in the items below [23]:

- Class 1 cross-sections are those which can form a plastic hinge with a rotating capacity to the minimum required for the use of plastic analysis methods;
- Class 2 cross-sections are those that due to the occurrence of local buckling, which is possible to achieve the plastic moment, but have a limited rotation capacity because of local buckling;
- Class 3 cross-section are those in which the stress in the extreme compression fiber of the steel element, assuming an elastic distribution of stress, can reach the yield strength value, but the plastic moment may not be achieved due to local buckling;
- Class 4 cross-section are those where the local buckling prevents the the yield strength to be reached on the most compressed areas of the section;



Subtitle:

1 – Class 1 cross-section

2 - Class 2 cross-section

3 - Class 3 cross-section

4 - Class 4 cross-section

M_{el} – Elastic moment

M_{pl} – Plastic moment

Figure 3- 34: Behaviour of the subject sections to flexion, [32]

According to EC3, the classification of a tubular cross section is made through Figure 3-35, considering the diameter and the thickness of the tubular member, as well as the steel yield strength (f_y).

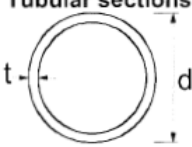
Tubular sections						
						
Class	Section in bending and/or compression					
1	$d/t \leq 50\epsilon^2$					
2	$d/t \leq 70\epsilon^2$					
3	$d/t \leq 90\epsilon^2$					
NOTE For $d/t > 90\epsilon^2$ see EN 1993-1-6.						
$\epsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ϵ	1,00	0,92	0,81	0,75	0,71
	ϵ^2	1,00	0,85	0,66	0,56	0,51

Figure 3- 35: Maximum width-to-thickness ratios for compression components of a tubular section, [23]

In order, to determine the class of the cross section, it is needed to apply the expression 3.71 presented below, from EC3:

$$\frac{d}{t} \quad (3.71)$$

3.6.1.2. BUCKLING

Due, to the slenderness of the monopile and tower when this elements are subjected to a large compressive stresses, it generates instabilities, being characterized by the occurrence of large cross deformations. This phenomena is called buckling, consisting in the appearance of flexural stress due to axial compressive loads alone.

The theoretical critical load, namely the load to which the element starts to develop deformations is defined by expression 3.72. This equation is based in some conditions like, material with linear elastic behavior, structure without geometric imperfections and residual stresses, as well as the applied loads which must be perfectly centered.

$$N_{cr} = \frac{\pi^2 \times E \times I}{(2 \times L)^2} \quad (3.72)$$

Where:

E: Elasticity modulus of the material [KPa]

I: Moment of inertia [m⁴]

L: Pile length [m]

The buckling resistance of an element depends on the flexural stiffness (EI) of the cross section, its length and also the support conditions. According to the EC3, the design of the elements subjected to pure compression is based on the application of buckling curves. This curves allow the reproduction of the imperfections effect of the real structure elements as, non-linearity, loads eccentricity, residual stress, etc..

3.6.1.3. DESIGN CHECKS

3.6.1.3.1. COMPRESSION DESIGN

According to EN1993-1-1 [23], the resistance of the cross-sections of axial compressed elements is given by the condition (4.3).

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1,0 \quad (3.73)$$

With:

N_{Ed} : Axial actuating compression effort [KN]

$N_{c,Rd}$: Resistance of the cross-section for uniform compression KN]

The value of the design resistance of the cross-section for uniform compression is obtained by:

- Class 1, 2 and 3 cross-sections:

$$N_{c,Rd} = \frac{A \times f_{yd}}{\gamma_{M0}} \quad (3.74)$$

- Class 4 cross-sections:

$$N_{c,Rd} = \frac{A_{eff} \times f_{yd}}{\gamma_{M0}} \quad (3.75)$$

Where:

A: Area of the cross-section [m²]

A_{eff} : Effective area of the cross-section [m²]

f_{yd} : Yield strength of the steel [KPa]

γ_{M0} : Partial safety coefficient equal to 1,0, defined by EC3 [-]

For compressed elements we must verify the condition:

$$N_{Ed} \leq N_{b,Rd} \quad (3.76)$$

The resistance to buckling per flexion, $N_{b,Rd}$, is determinate as:

- Class 1, 2 and 3 cross-sections:

$$N_{b,Rd} = \frac{x \times A \times f_{yd}}{\gamma_{M1}} \quad (3.77)$$

- Class 4 cross-sections:

$$N_{b,Rd} = \frac{x \times A_{eff} \times f_{yd}}{\gamma_{M1}} \quad (3.78)$$

With:

x : Reduction factor for the relevant buckling mode [-]

γ_{M1} : Partial safety coefficient equal to 1,0, defined by EC3 [-]

$$x = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \leq 1,0 \quad (3.79)$$

$$\Phi = 0,5 \times [1 + \alpha \times (\bar{\lambda} - 0,2) + \bar{\lambda}^2] \quad (3.80)$$

The dimensionless coefficient of slenderness ($\bar{\lambda}$) is calculated by:

- Class 1, 2 and 3 cross-sections:

$$\bar{\lambda} = \sqrt{\frac{A \times f_{yd}}{N_{cr}}} \quad (3.81)$$

- Class 4 cross-sections:

$$\bar{\lambda} = \sqrt{\frac{A_{eff} \times f_{yd}}{N_{cr}}} \quad (3.82)$$

Where:

α : Imperfection factor [-]

N_{cr} : Critical load [KN]

3.6.1.3.2. COMBINED COMPRESSION WITH BENDING DESIGN

According to EC3, the sectional verification for the applied internal forces is given by:

$$\sigma \leq \sigma_y \rightarrow \frac{N_{Ed}}{A} + \frac{M_{Ed}}{I} \times y \leq \sigma_y \quad (3.83)$$

And:

$\sigma_y = f_{yd}$: Yield strength of the steel [KPa]

A: Area of the cross-section [m²]

I: Moment of inertia [m⁴]

N_{Ed} : Actuating axial force [KN]

M_{Ed} : Actuating bending moment [KN.m]

y: Equal to the radius of the cross-section [m]

3.6.1.3.3. MEMBER DESIGN CHECK

According to EC3, the member verifications must be carried out in order, to analyse the buckling safety of the element, so it is described as:

$$\frac{N_{Ed}}{x \times N_{pl,Rd}} + \frac{k \times M_{Ed}}{M_{el,Rd}} \leq 1,0 \quad (3.84)$$

$$N_{pl,Rd} = A \times f_{yd} \quad (3.85)$$

$$M_{el,Rd} = \frac{W_{el} \times f_{yd}}{\gamma_{M0}} \quad (3.86)$$

$$w_{el} = \frac{\pi \times (D^4 - d^4)}{32 \times D} \quad (3.87)$$

With:

N_{Ed} : Actuating axial force [KN]

M_{Ed} : Actuating bending moment [KN.m]

χ : Reduction factor for the relevant buckling mode, calculated by equation (4.8) [-]

A : Area of the cross-section [m²]

f_{yd} : Yield strength of the steel [KPa]

γ_{M0} : Partial safety coefficient equal to 1,0, defined by EC3-1-1 [-]

D : Cross-section external diameter [m]

d : Cross-section internal diameter [m]

k : interaction factor [-]

3.6.2. DYNAMIC EFFECTS – VORTEX SHEDDING

When a given fluid or gas for example the wind and waves flows past a certain tubular structure like an offshore wind turbine, the longitudinal movement of the fluid around the tubular section with a given critical velocity can cause the formation of the phenomenon known as vortex shedding, a transversal movement generated by the designated whirls of Von Kármán [80].

If, the frequency shedding vortices (f_{vortex}) around a cylindrical pile as the same magnitude of the fundamental frequency of the structure, lateral vibrations may occur in highly harmful resonance for the stability of the structure, leading to the total collapse.

This frequency phenomenon is variable with the number of Reynolds and a critical velocity, as it is showed in the expression 3.88 [81]:

$$f_{vortex} = \frac{1}{\phi \times (R_e)} \times \frac{U}{D} \approx \frac{U}{5 \times D} \quad (3.88)$$

For the design it is applied the following equation:

$$D = \frac{U_{cr}}{5 \times f} \quad (3.89)$$

The critical wind velocity is calculated by:

$$U_{cr} = \frac{f \times D}{S_t} \quad (3.90)$$

For design reasons, it is important to determine the static force equivalent to the generated forces during the fluctuations in resonance, because it is capable of causing vibrations on the structure. This force is given per unit length, being calculated as [81]:

$$F_L = \frac{1}{2 \times \xi} \times C_L \times D \times q_{cr} \quad (3.91)$$

The critical dynamic overload due to wind is given by:

$$q_{cr} = 0,613 \times U_{cr}^2 \quad (3.92)$$

Therefore, it is needed to verify if the $U_{cr} \leq 0,2 \times U_{project}$ in order, to observe if at the base of the tower the tensions are not controlled by the sharing vortices phenomenon.

Where:

$\emptyset \times (R_e)$: The value of the dimensionless function for tubular sections is about 5,0 [-]

U: Wind velocity [m/s]

U_{cr} : Critical wind velocity [m/s]

D: Pile diameter [m]

f: Natural frequency of the structure [Hz]

S_t : Strouhal number with an approximate value of 0,2 [-]

C_L : Von Kármán lift coefficient which for cylinders has a value of 0,2 [-]

ξ : Damping viscous coefficient for steel structures is about 2 %

$U_{project}$: Project wind velocity [m/s]

3.6.3. DYNAMIC EFFECTS - OVALIZATION OF THE SECTIONS

The wind force passing through the tubular structure is not uniform. On the direction of the flow it compresses the section and from the back side decompresses the section, generating flexion efforts which can originate the roundness of the section. This instability phenomenon is more dangerous in slender towers.

The ovalization frequency is calculated as [81]:

$$f_{round} = 175,4 \times e \times \frac{\sqrt{E}}{D^2} \quad (3.93)$$

The resonance can also occur, when the roundness fundamental frequency of the sections is the double of the sharing vortices frequency, like it is shows below:

$$f_{round} \approx 2f_{vortex} \rightarrow U_{cr,round} \approx 438,5 \times e \times \frac{\sqrt{E}}{D} \quad (3.94)$$

With:

e: Pile thickness [m]

D: Pile diameter [m]

E: Wind elasticity modulus [KPa]

When $D/e < 250$ it is not necessary to make the above calculations because this phenomenon is negligible. For values higher than 250 are necessary the use stiffening rings.

3.7 FOUNDATION

Like it was already mentioned in chapter 2, the increase of the turbine generator capacity during the years, requires a large rotor, higher towers which induce larger vertical and lateral forces acting on the foundation.

Therefore, for the foundation design it is going to be study the pile-soil interaction behaviour for a monopile which can be subjected to static, dynamic and cyclic lateral loads in medium dense sand.

For designing laterally load piles the p-y curves method is used, which is provided in current design regulations, such as, the Det Norske Veritas (DNV), the American Petroleum Institute (API) or the Germanischer Lloyd (GL).

The soil-pile behaviour is affected by the flexibility of the pile. Based on the recommendations of some researchers, e.g. Poulos and Hull, the piles used for the development of the p-y curves behave as flexible piles [82]. Generally, the monopiles used for offshore wind structures behave as rigid piles due to the large diameter related to the respective pile length, being usually called as short piles. They only rotate when subjected to large horizontal loads and if founded in soft soil. Figure 3-36 shows the two respective behaviours of the monopiles.

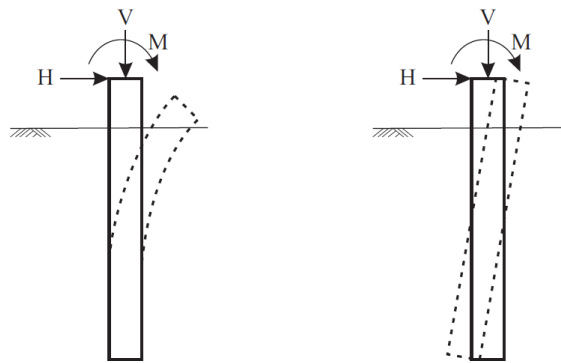


Figure 3- 36: Flexible versus rigid pile behaviour, [82]

Then, for larger diameter piles the soil stiffness response of the soil-pile interaction will be higher compared to flexible, slender piles and the p-y curves do not consider explicitly the pile stiffness. Consequently, it is necessary to check the validity of the p-y curves method for this kind of design structures [83].

For this study, in order to evaluate the pile-soil interaction it was assumed that the monopile has a flexible behaviour, applying directly the Winkler approach.

3.7.1. P-Y CURVES – WINKLER MODEL

Laterally loaded monopiles used for the offshore wind turbines industry are normally designed based on the Winkler approach, considering a beam supported by nonlinear elastic springs which represents the soil lateral stiffness. This is based on semi-empirical relations between the soil pressure (p) acting against the pile wall and the lateral displacement (y) of the pile, like is showed in Figure 3-37 [82].

The p-y method describes the non-linear relation between pile deflection and soil resistance. With the increase of the depth the soil response becomes stiffer. According to the standards, the soil resistance depends on the type of soil and its properties such as the angle of internal friction (Φ) and density.

According to the Winkler model, that describes the horizontal deflection of a pile subjected to the horizontal load and bending moment through the following equation [82]:

$$E_p I_p \times \frac{d^4 y}{dx^4} - E_{py} \times y = 0 \quad (3.95)$$

Where y is the lateral displacement of the pile at a given point along the pile, E_{py} is the spring stiffness, $E_p I_p$ is the bending stiffness of the pile with the Young's modulus E_p and the second moment of area I_p .

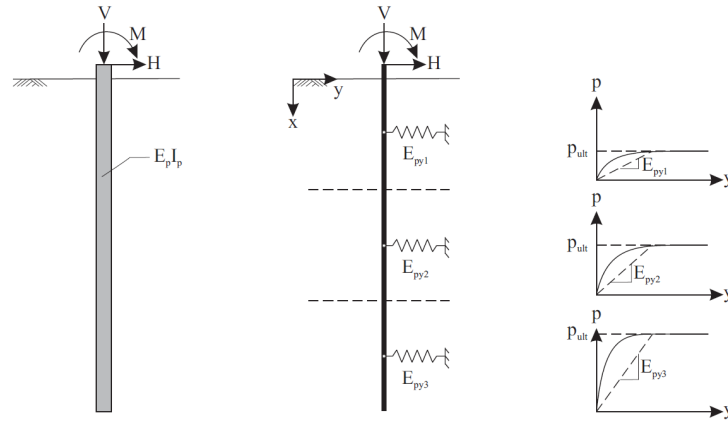


Figure 3- 37: Winkler model and definition of the p-y curves, [82]

Generally, the E_{py} increases with depth and decreases with increasing displacements, which depends on the soil conditions and in the static loading case. The lateral capacity of the soil p_{ult} is obtained through the p-y curves when a horizontal asymptote is reached, being possible to obtain the stiffness of each soil stratum.

3.7.2. P-Y CURVES FOR PILES IN SAND

Based on the API standard, the p-y method for piles in sand is given by [82]:

$$p = A \times p_{ult} \times \tanh\left(\frac{K_{sand} \times z \times y}{A \times p_{ult}}\right) \quad (3.96)$$

The initial modulus of subgrade reaction K_{sand} depends on the angle of internal friction Φ (degrees), being expressed in [KPa/m] as:

$$K_{sand} = (0.008085 \times \Phi^{2.45} - 26,09) \times 10^3 \quad (3.97)$$

This equation is applied for internal friction angles between $29^\circ \leq \Phi \leq 45^\circ$

The ultimate lateral capacity p_{ult} at certain depth below the seabed for shallow depths waters is determinate by:

$$p_{ult,shallow} = (C_1 \times z + C_2 \times D) \times \sigma_v \quad (3.98)$$

For deep depths waters, the ultimate lateral resistance p_{ult} at given depth below the seabed is calculate as:

$$p_{ult,deep} = C_3 \times D \times \sigma_v \quad (3.99)$$

The equation considered for the calculation of the lateral capacity p_{ult} is the one, which gives a smaller value for a certain depth.

The coefficients to determine the ultimate lateral capacity are function of the angle of internal friction Φ (degrees), and can be described by the following expressions:

$$C_1 = 0,115 \times 10^{0,0405 \times \Phi} \quad (3.100)$$

$$C_2 = 0,571 \times 10^{0,022 \times \Phi} \quad (3.101)$$

$$C_3 = 0,646 \times 10^{0,0555 \times \Phi} \quad (3.102)$$

With:

z: Distance below seabed [m]

y: Lateral displacement [m]

D: Pile diameter [m]

A: Factor for cyclic or static loading conditions:

- For cyclic loading $A = 0,9$
- For static loading $A = \left(3,0 - 0,8 \times \frac{z}{D}\right) \geq 0,9$

σ_v : Effective vertical stress at the considered depth [KPa]

The present chapter provided a summary of the theoretic basis regarding the structural concepts and criteria for the pre-assessment of the design of an offshore wind turbine with monopile foundation. The theory for loads quantification and the structural dynamic behaviour were presented and the verifications and safety criteria were reviewed. The following chapter 4 will provide the case study of the offshore wind farm park Horns Rev I, located in Denmark, which is analysed according to the contents in chapter 3. Therefore, the conclusions presented below were obtained, according to the calculations performed for the Horns Rev I:

- The limit states that must be analysed for an offshore wind turbine are the ULS, SLS, FLS, ALS.
- The present study was dedicated to the worst case scenario conditions which referred to the ultimate limit state (ULS). However the other three are essential in a real life design project, in order to obtain a suitable structural analysis. For a review study as this one, it is very difficult to obtain trustful data on accidental loads, particularly the ones that lead to the partial or total collapse. This fact is linked to the high competitive environment that rules in offshore engineering companies.
- Besides the limit states, there are phenomena which might also be very important when designing such type of infrastructures, e.g. corrosion, for instance plays a major role in the stability of the transition piece and the metallic/steel joints;
- The main loads considered were the ones induced by waves, currents and wind. For this agents the theories used can achieve a considerable level of complexity;

- In this case study the linear Airy theory provided a reasonable approach for the waves' loads. It was concluded that this simplification, instead of the Navier-Stokes integration, leads to quicker and simpler calculations, but it is also on the conservative side, which is guaranteed by the safety coefficients applied;
- The low velocity that characterised the current's actions led to a higher importance of the loads caused by the hydrodynamic behaviour of the waves;
- Both currents and waves effects are later on combined in order to consider the hydrodynamic final loads;
- The calculations for both variables were performed successfully according to Morrison formulas which are both applied DNV and API standards. In this research the DNV was the followed standard. However it was concluded that the theoretical concepts behind them are in several aspects similar
- DNV and API standards differ in the calculations of the drag and inertia coefficients, which leads to different values in the hydrodynamic forces. Due to the lack of time, in the present dissertation, the computations according to API weren't performed. Nevertheless DNV provided suitable results for safety verification.
- The normal wind conditions calculation, according to DNV, required the statistics regarding the 1 hour and 10 minutes wind speed at 10 meters height. This information wasn't available for the Horns Rev I location. Therefore, field information was used from the nearest point available (Ekofisk Oilfield), which was used to apply the extreme wind conditions method, also given in DNV standards. This method only requires the maximum wind speed and provided reasonable results and avoided the problem of lack of information.
- The general wind speed and force relationship was used to achieve the wind drag force and the obtained load was further used for design purposes. This calculations were based on the expressions which are stated in API standards.
- More specific conditions, as the aerodynamic loads on the blades and rotor, were discussed and computed for the case study.
- The structural analysis software used doesn't include a package to compute the hydrodynamic loads and the aerodynamic ones.
- The loads combinations were performed according to the Eurocode and DNV standards. Four combinations were used, two of them from each reference.
- The axial forces' worst case scenario was obtained through DNV, in both cases, with the rotor stopped or working. This values were also consistent with the approach of the cantilever beam and the approach with the soils' interactions.
- On the other side, critical section of the monopile, which was the base, also had a more severe value according to DNV. This occurred as well in both cases, with the cantilever beam and the approach with the soils' interaction.
- The combination of Eurocode I was the most severe one for the hub tower, i.e. the non-submersed part of the structure. This was induced by the coefficients factors which increased the winds' load.
- The vibration frequencies were also studied, in case one (cantilever element) and two (soils' interaction/stiffness).
- For case one the vibration is within the design limits. However in the second case it isn't. In order to correct this the diameter should be increased, taking into consideration the thickness changes and the new axial weights.
- A preliminary check on the safety verifications was performed successfully and it is discriminated and compared along the dissertation (Chapter 4).

4

Case Study

4.1 GENERAL CONSIDERATIONS

This chapter will present all the considerations and processes performed, in order to obtain the design project of a specific wind turbine. Some important design aspects like, the geometry, sections, material, and especially the actions that the structure is subjected.

Other point to refer, is the finite element program that was used with the objective to verify as real as possible the natural frequency of the structure, the displacement and the values of the internal forces, analyzing also the dynamic soil structure interaction. The program used was Autodesk Robot Structural Analysis Professional 2015.

4.2 INITIAL CONSIDERATIONS OF THE PROJECT

The objective of this study is the design of a monopile foundation for the support of a wind turbine.

In the design of an offshore wind turbine project the efficiency and economy are two of the most important points, so it is necessary to achieve an optimal combination of these two criteria. Therefore, it is fundamental to define a structural scheme in accordance with the regulations and based on other previous studies and only after this it is possible to make an economic and efficient study, based on the changes of the geometry of the structure.

The study is based on data from the offshore wind farm park Horns Rev I, which is located in (Denmark) the coastline of the North Sea and it was commissioned in December of 2002. The area of this park is 20 km² and contains 80 wind turbines, the distance from shore is about 14-20 km and water depth between 6-14 m. The installed capacity is 160 MW and the annual estimated production is 600GWh/year [84].

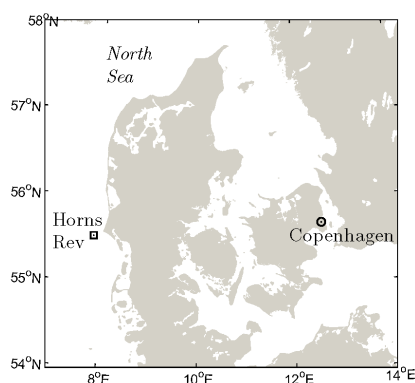


Figure 4 - 1: Horns Rev 1 wind farm park location, [85]

4.3. WIND TURBINE MODEL

The wind turbine, tower and substructure specifications from Horns Rev I are presented in the next table:

Table 4 - 1: Horns Rev 1 data, [86]

Turbine	Turbine Manufacturer:	Vestas Wind Systems
	Turbine Model:	Vestas V80-2.0 MW
Operational	Cut-in Wind Speed:	4 m/s
	Rated Wind Speed:	16 m/s
	Cut-out Wind Speed:	25 m/s
Rotor & Hub:	Rotor Type:	3-bladed, 86 horizontal axis
	Rotor Position:	Upwind
	Rotor Diameter:	80 m
	Rotor Area:	5027 m ²
	Rotor Speed (min):	10.8 rpm
	Rotor Speed (rated):	16.7 rpm
	Rotor Speed (max):	19.1 rpm
	Rotor Weight (incl. hub):	20 t
	Hub Height (above MSL):	70 m
Blades	Blade Tip Height (above MSL):	110 m
	Blade Length:	39 m
	Blade Max Chord (max width):	3.5 m
	Weight pr. Blade:	6.5 t
Nacelle	Nacelle Weight:	79 t
Tower	Structure Type:	Tubular Steel Tower
	Height:	61 m
	Weight:	160 t
Substructure:	Structure Type:	Steel monopile
	Transition Piece Structure Description:	Grouted; Diameter: 4,2 m; Length: 18 m
	Support Structure Description:	Diameter: 4 m; Thickness: 5 cm; Weight: 180-230 t
	Foundation Type:	Piled
	Foundation Structure Description:	The monopiles are driven 25 m into seabed

The next figures shows the geometry of the wind turbine and presents the power curves of the respective turbine, the V80-2.0 MW:

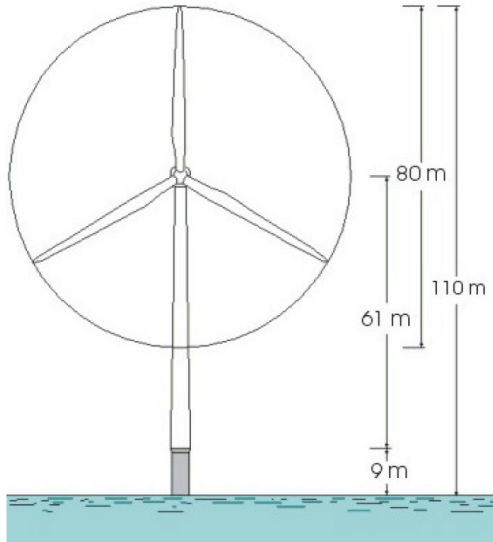


Figure 4 - 2: Dimensions of the wind turbine, [87]

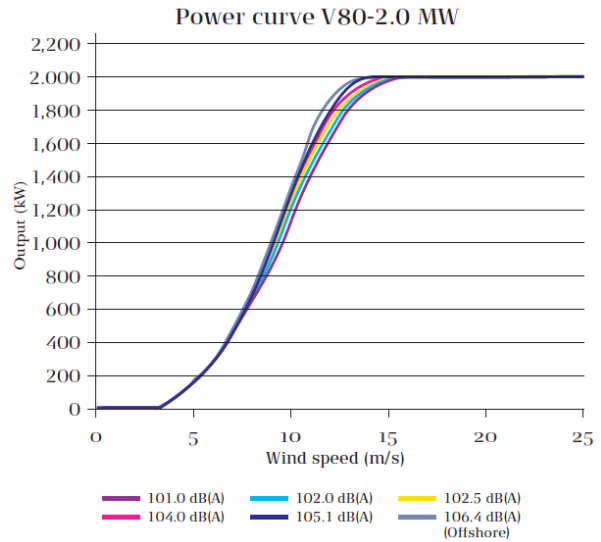


Figure 4 - 3: Power curves at different sound levels for the V80-2.0 MW turbine, [88]

4.3 SIMPLIFICATIONS AND ASSUMPTIONS FOR THE DESIGN

In general, simplifying a model basically corresponds to reducing the number of variables and parameters of the real system. However, every simplification must be well thought and argued in order for the model which is going to be analysed, adequately represent the reality.

Taking into account the complete structure of the wind turbine, the following simplifications and assumptions are described in the present topics:

- All external equipment, such as work platform, cables and ladders, welded or bolted to the structure, are not considered as a structural element, so this aspects are omitted;
- About the transition piece with a grouted connection is not going to be studied, so the stresses on this area will not be quantified. This is a delicate area and therefore it should be studied in a finite element program. So, in this dissertation the grout is considered as a linear elastic and isotropic continuous material, with a compressive strength, steel density, Poisson's ratio and modulus of elasticity. Note that this piece has important phenomena as corrosion for example;
- Due, the lack of information, it was assumed that the tower is variable with height, having at the base an outside diameter of 4,0 m and 0,05 m of thickness, at the top an outside diameter of 3,0 m and 0.035 m of thickness.

During this chapter more simplifications and assumptions will be mentioned. However the most important ones have been already presented above and will be putted into practice, on the calculation model.

4.4 STRUCTURAL MODELLING

The idealization of the calculation model was done in order to analyse the structure behaviour, according to the following aspects of the project:

- Dynamic response of the structure;
- Soil-structure iteration;
- Safety verification of the ultimate limit state (ULS);
- Safety verification to the serviceability limit state (SLS).

Therefore, a model was created which ensures a better approximation of reality as much as possible, contemplating the above aspects. Due, to the present structural analysis it was used a linear model with beam elements with a variable section for the characterization of the tower and a uniform section for the monopile foundation. The sections were defined through the data provided in Table 4-1. The structure is going to be analysed for two different situations:

- On a first design approach of the structure, the study it is going to be done on a simplified model, considering that the monopile is fixed at the seabed level;
- On a second design approach, it is considered that the structure has three degrees of freedom at the seabed level, which more accurately represents the reality and with this process it is made a study to the dynamic soil-structure iteration.

After the implementation of this two designing processes, a comparison of the results obtained will be made.

4.5 LOADS DETERMINATION

The structure must resist to all the loads provided from the turbine, the tower, its self-weight and the environmental loads. These will interfere on the substructure weight, due to the internal forces that these loads induce to the structure, so they are very relevant in order to determine the substructure thickness. Along this subchapter the values of the permanent and environmental loads will be presented. The results will determine the strength that the monopile has to resist to and will allow to obtain the size and thickness of the same.

The expressions used for the loads calculation were shown already in Chapter 3. Thus, in this Chapter the results from the loads will be given in table formats for a better visualization.

4.5.1. PERMANENT LOADS (G)

As mentioned in Chapter 3, the permanent loads refer to the weight of the complete structure. From table 4-1, it was possible to obtain the weight of the structure components and the total weight of the structure, which is shown in Table 4-2:

Table 4 - 2: Permanent loads

Rotor & Hub Weight (KN):	196,13
3 Blades Weight (KN):	191,23
Nacelle Weight (KN):	774,72
Tower Weight (KN):	2189,08
Monopile Weight until seabed level (KN):	1076,07
Total weight of the structure from the top to the seabed level (KN):	4427,24

4.5.2. VARIABLE FUNCTIONAL LOADS (Q)

In Chapter 3 it was seen that the variable loads are described as the collision forces caused from the ship impacts against the structure. These loads will not be considered on this study as previously justified. The loads from the installation operations and maintenance of the wind turbine will also not be considered due to the lack of information.

After the characterization of all the loads, they will be combined and multiplied each one by a safety coefficient.

4.5.3. ENVIRONMENTAL LOADS (E)

4.5.3.1. WAVES AND CURRENT QUANTIFICATION

For the present study is not necessary to obtain the real wave spectra from the zone because this study is based on an existing wind farm park, so it is possible to search for available meta-ocean data from that location. Table 4-3 shows this information:

Table 4 - 3: Metocean data from Horns Rev 1, [64]

Maximum wave height – H_{max} (m)	Wave period – T (s)	Water depth – d (m)
8,1	12	13,5

In terms of design, some engineers consider the significant wave height (H_s) of the time series and not the maximum wave height, due to the combination coefficient that would increase the wave force.

The significant wave height is defined as the mean of the 1/3 highest waves in the time series, which is equal to 4 times the standard deviation (σ) of the time series [60]. Therefore, this wave has a relation between the maximum wave height and a coefficient, like it is shown [89]:

$$H_s = \frac{H_{max}}{1,86} = \frac{8,1}{1,86} = 4,36 \text{ m}$$

In this study it will be considered for the design the maximum wave height (H_{max}) because it is the worst scenario. In Table 4-4 shows the results of the wave parameters, which are obtained through the expressions in Chapter 3.2.1.3.

Table 4 - 4: Wave Parameters

Wave	Amplitude [m]	Wavelength [m]	Celerity [m/s]
	4,05	129,40	10,78
	Angular Frequency [rad/s]	Frequency [Hz]	Wave number [-]
	0,52	0,08	0,05

The next step is to verify in Figure 3-4 which wave theory can be used, so the theory obtained according to the figure was Stokes nonlinear wave theory, but in order to simplify the calculation process it is assumed the linear Airy theory.

This assumption can be taken, due to the safety factor used on the wave combination that covers this simplification and also when we determine the total hydrodynamic force with the Morison equation it considers that the drag force and the inertia force occur at the same time, which is not true. Therefore

the consideration of the linear Airy theory is reasonable and allows us to be on the conservative approach of design, for the present study. If more complex theories are used, the design processes will require the application of simulations methods and computational fluids methods in order to model the wave's behaviour.

To proceed with the calculations of the water particle velocity, acceleration and pressure through Airy linear theory, it is necessary to verify which phase angle originates a higher internal forces. So, the calculation of the water particle parameters was made for the intermediate depth waters formulas presented in subchapter 3.2.1.2, due to the relation between the water depth and the wavelength. The formulas were applied to all the following phase angles, which were explained in subchapter 3.2.1.4:

Table 4 - 5: Phase Angles

Phase Angles				
0	$\pi/12$	$\pi/6$	$\pi/4$	$\pi/3$
$5\pi/12$	$\pi/2$	$7\pi/12$	$2\pi/3$	$3\pi/4$
$5\pi/6$	$11\pi/12$	π	$13\pi/12$	$7\pi/6$
$5\pi/4$	$4\pi/3$	$17\pi/12$	$3\pi/2$	$19\pi/12$
$5\pi/3$	$7\pi/4$	$11\pi/6$	$23\pi/12$	2π

To determinate the hydrodynamic drag and inertia forces through the Morison equation it were used the expressions shown in subchapter 3.2.1.5.

The formulas used for the calculation of the total hydrodynamic force and bending moment are presented in subchapter 3.2.1.8. The integration was made from the sea surface elevation until the seabed.

After the calculations of the forces for all phase angles, it was observed that the $\pi/4$ phase angle has the higher forces, being the worst situation.

If the interval of the phase angles were more refined, maybe it would be slightly different. This could be solved by means of a computational algorithm. Nevertheless, since this case was a simple application case, the values hereby presented seem reasonable enough.

The current is added to the hydrodynamic drag force as shown in equation 3.31. Therefore, to determine the current velocity the value of the tidal current at still water level is needed, which is **0,5 m/s** [90] as well as the 1-hour mean wind speed at 10 meters height, which was assumed to be **28,8 m/s** due to the lack of information about the measured wind speed at the respective location. This wind velocity has a return period of one year and it was obtained from the Ekofisk oil field located also in the North Sea [69] which is the nearest location to the Horns Rev I site.

Table 4-6 are exposed the values of the current from the still water level until the seabed, calculated with the expressions in subchapter 3.2.1.6:

Table 4 - 6: Current velocity values for a water depth interval of 0,5 meters

Z (m)	$v_{wind}(z)$ [m/s]	$v_{tide}(z)$ [m/s]	$v(z)$ [m/s]
0,00	0,58	0,50	1,08
-0,50	0,57	0,50	1,07
-1,00	0,56	0,49	1,06
-1,50	0,56	0,49	1,05
-2,00	0,55	0,49	1,04
-2,50	0,55	0,49	1,03
-3,00	0,54	0,48	1,02
-3,50	0,54	0,48	1,01
-4,00	0,53	0,48	1,01
-4,50	0,52	0,47	1,00
-5,00	0,52	0,47	0,99
-5,50	0,51	0,46	0,98
-6,00	0,51	0,46	0,97
-6,50	0,50	0,46	0,96
-7,00	0,50	0,45	0,95
-7,50	0,49	0,45	0,93
-8,00	0,48	0,44	0,92
-8,50	0,48	0,43	0,91
-9,00	0,47	0,43	0,90
-9,50	0,47	0,42	0,89
-10,00	0,46	0,41	0,87
-10,50	0,46	0,40	0,86
-11,00	0,45	0,39	0,84
-11,50	0,44	0,38	0,82
-12,00	0,44	0,37	0,80
-12,50	0,43	0,34	0,78
-13,00	0,43	0,31	0,74
-13,50	0,42	0,00	0,42

With the values of the current velocity obtained in Table 4-6 it was drawn the current diagram, which has the following form:

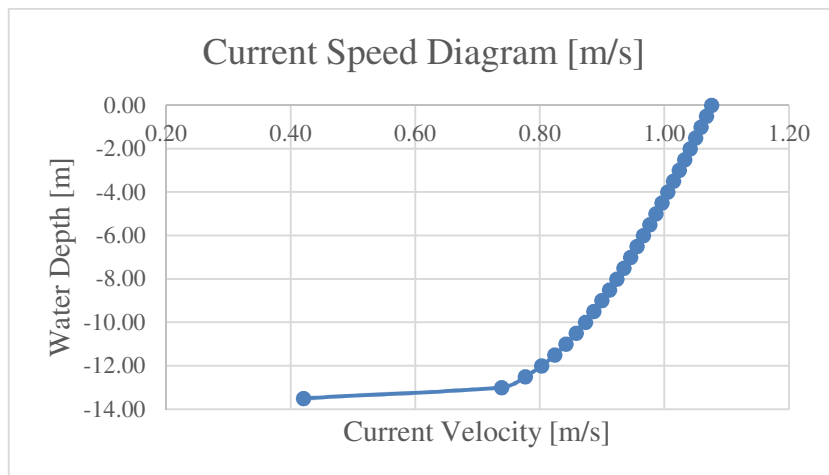


Figure 4 - 4: Current Speed Diagram

Finally, it is presented in Table 4-7 all the values for the horizontal water particle velocity and acceleration, pressure, the hydrodynamic drag and inertia force, with the current velocity added to the velocity component of the drag force and the total hydrodynamic force, as follows:

Table 4 - 7: Final efforts results for a water depth interval of 0,5 meters

$\Theta = \pi/4$	Horizontal		Hydrodynamic Forces			
Z (m)	Velocity [m/s]	Acceleration [m/s ²]	Drag Force [kN]	Inertia Force [kN]	Total Force [kN]	Pressure [Kpa]
4,00	2,95	1,54	9,30	16,36	25,66	-7,64
3,50	2,90	1,52	8,99	16,09	25,08	-3,15
3,00	2,85	1,49	8,70	15,83	24,54	1,36
2,50	2,81	1,47	8,43	15,58	24,01	5,89
2,00	2,76	1,45	8,17	15,34	23,51	10,43
1,50	2,72	1,43	7,93	15,11	23,03	15,00
1,00	2,68	1,40	7,69	14,88	22,57	19,58
0,50	2,64	1,38	7,47	14,67	22,14	24,18
0,00	2,61	1,36	14,50	14,46	28,96	28,80
-0,50	2,57	1,35	14,15	14,26	28,42	33,43
-1,00	2,54	1,33	13,82	14,07	27,90	38,08
-1,50	2,50	1,31	13,51	13,89	27,40	42,75
-2,00	2,47	1,29	13,21	13,72	26,93	47,43
-2,50	2,44	1,28	12,92	13,56	26,47	52,13
-3,00	2,41	1,26	12,64	13,40	26,04	56,85
-3,50	2,39	1,25	12,38	13,25	25,63	61,58
-4,00	2,36	1,24	12,13	13,11	25,24	66,32
-4,50	2,34	1,22	11,89	12,98	24,86	71,09
-5,00	2,32	1,21	11,66	12,85	24,51	75,86
-5,50	2,29	1,20	11,44	12,73	24,17	80,66
-6,00	2,27	1,19	11,23	12,62	23,85	85,46
-6,50	2,26	1,18	11,03	12,52	23,55	90,29
-7,00	2,24	1,17	10,84	12,42	23,27	95,12
-7,50	2,22	1,16	10,66	12,33	23,00	99,97
-8,00	2,21	1,16	10,49	12,25	22,74	104,84
-8,50	2,19	1,15	10,32	12,18	22,50	109,72
-9,00	2,18	1,14	10,16	12,11	22,27	114,61
-9,50	2,17	1,14	10,00	12,05	22,06	119,52
-10,00	2,16	1,13	9,85	12,00	21,85	124,45
-10,50	2,15	1,13	9,71	11,95	21,66	129,38
-11,00	2,15	1,12	9,56	11,92	21,47	134,33
-11,50	2,14	1,12	9,41	11,88	21,29	139,30
-12,00	2,14	1,12	9,25	11,86	21,10	144,28
-12,50	2,13	1,12	9,06	11,84	20,90	149,27
-13,00	2,13	1,12	8,81	11,83	20,64	154,28
-13,50	2,13	1,12	6,96	11,83	18,79	159,30
$\Sigma =$					858,03	

The hydrodynamic load is obtained from the seabed until the sea surface elevation, being the surface elevation calculated by expression 3.5, as it shows below:

$$h = a \cdot \cos(\Theta) = 4,05 \cdot \cos(0) = 4,05 \text{ m}$$

$\Theta = 0$ degrees, because the sea surface elevation is higher considering the zero degrees phase angle, being the worst scenario. Applying the $\pi/4$ phase angle originates a higher drag and inertia force.

The calculations presented in Table 4-7 were made for a depth water interval of 0,5 meters resulting the exposed forces. After this, the depth water interval was refined to 0,1 meters obtaining a bigger precision for the total hydrodynamic force, which are presented in Table 4-8. This total force result is at the seabed level, where the internal forces have a maximum value, due to be the monopile critical area.

Table 4 - 8: Final efforts results for a water depth interval of 0,1 meters

Hydrodynamic Forces	
Total Force [KN]	Pressure [Kpa]
837,88	159,30

With the total hydrodynamic force values obtained for a water depth interval of 0,1 meters it was possible to draw the shear force diagram:

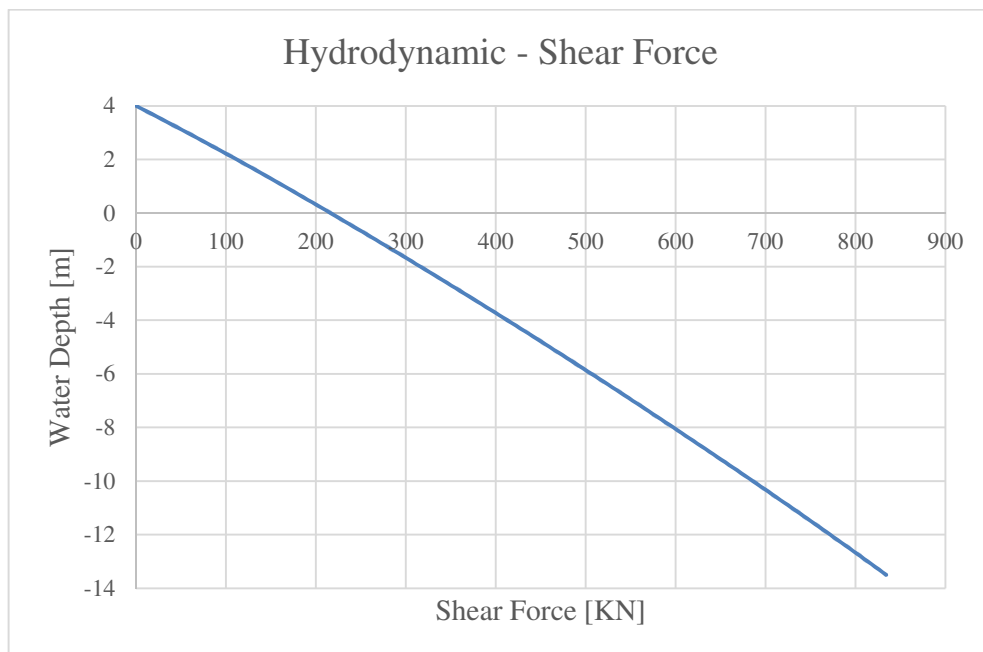


Figure 4 - 5: Total hydrodynamic shear force diagram

After integrating the shear force diagram the bending moment diagram is obtained as follows:

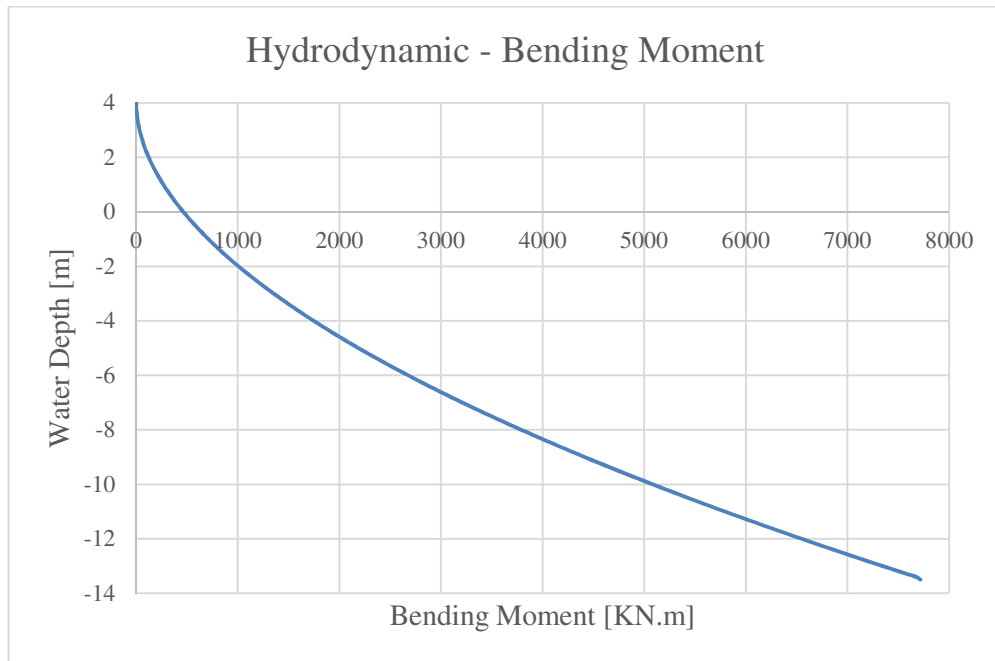


Figure 4 - 6: Total hydrodynamic bending moment diagram

The breaking waves, will not be considered in this study because it was assumed that the waves break near the coast line and not within the offshore wind farm.

Breaking waves criterion is given by:

$$H = 0,78 \times d = 0,78 \times 13,5 = 10,53 \text{ m}$$

The wave height is 8,1 m less than 10,53 m, so the breaking waves occurs near to the coast and not within the wind turbine.

4.5.3.2. WIND QUANTIFICATION

For the characterization of the wind loads, the DNV code makes a distinction between normal wind conditions and extreme wind conditions, like it was exposed in subchapter 3.2.2. For, offshore foundation design it is recommended to use the worst scenario, so in this study we are going to proceed with the extreme wind conditions method [69].

To implement this method according to DNV code it is necessary the 1-hour mean wind speed at 10 meters height, this value is the same that was used to determinate the current speed on subchapter 4.5.3.1., so the wind velocity assumed is **28,8 m/s** for a return period of one year.

4.5.3.2.1. WIND FORCE ON TOWER

The extreme wind conditions method takes into account the variation of the wind with height and duration. Therefore, after the implementation of the expressions it is obtained the wind speed values per meter of the wind turbine tower. To calculate the wind force load for each meter of tower, it is applied equation 3.47, obtaining the following values presented in Table 4-9:

Table 4 - 9: Wind force values per meter of tower

Height [m]	Wind Turbulence - IU [m/s]	Wind Velocity - U(T,z) [m/s]	Tower Diameter - D [m]	Wind Force - Fw [KN]
70,00	0,09	46,85	3,00	2,06
69,00	0,09	46,81	3,02	2,07
68,00	0,09	46,77	3,03	2,07
67,00	0,09	46,73	3,05	2,08
66,00	0,09	46,70	3,07	2,09
65,00	0,09	46,66	3,08	2,10
64,00	0,09	46,62	3,10	2,10
63,00	0,09	46,57	3,11	2,11
62,00	0,09	46,53	3,13	2,12
61,00	0,09	46,49	3,15	2,13
60,00	0,09	46,45	3,16	2,13
59,00	0,09	46,41	3,18	2,14
58,00	0,09	46,36	3,20	2,15
57,00	0,09	46,32	3,21	2,15
56,00	0,09	46,27	3,23	2,16
55,00	0,09	46,22	3,25	2,17
54,00	0,09	46,18	3,26	2,17
53,00	0,09	46,13	3,28	2,18
52,00	0,09	46,08	3,30	2,19
51,00	0,09	46,03	3,31	2,19
50,00	0,09	45,98	3,33	2,20
49,00	0,09	45,93	3,34	2,20
48,00	0,10	45,87	3,36	2,21
47,00	0,10	45,82	3,38	2,22
46,00	0,10	45,76	3,39	2,22
45,00	0,10	45,71	3,41	2,23
44,00	0,10	45,65	3,43	2,23
43,00	0,10	45,59	3,44	2,24
42,00	0,10	45,53	3,46	2,24
41,00	0,10	45,47	3,48	2,25
40,00	0,10	45,40	3,49	2,25
39,00	0,10	45,34	3,51	2,25
38,00	0,10	45,27	3,52	2,26
37,00	0,10	45,20	3,54	2,26
36,00	0,10	45,13	3,56	2,26
35,00	0,10	45,06	3,57	2,27
34,00	0,10	44,98	3,59	2,27
33,00	0,10	44,91	3,61	2,27
32,00	0,10	44,83	3,62	2,28
31,00	0,10	44,75	3,64	2,28
30,00	0,11	44,66	3,66	2,28
29,00	0,11	44,57	3,67	2,28
28,00	0,11	44,48	3,69	2,28
27,00	0,11	44,39	3,70	2,28

26,00	0,11	44,29	3,72	2,28
25,00	0,11	44,19	3,74	2,28
24,00	0,11	44,08	3,75	2,28
23,00	0,11	43,97	3,77	2,28
22,00	0,11	43,86	3,79	2,28
21,00	0,11	43,74	3,80	2,27
20,00	0,12	43,61	3,82	2,27
19,00	0,12	43,48	3,84	2,27
18,00	0,12	43,33	3,85	2,26
17,00	0,12	43,19	3,87	2,25
16,00	0,12	43,03	3,89	2,25
15,00	0,12	42,86	3,90	2,24
14,00	0,12	42,68	3,92	2,23
13,00	0,13	42,48	3,93	2,22
12,00	0,13	42,27	3,95	2,21
11,00	0,13	42,04	3,97	2,19
10,00	0,13	41,79	3,98	2,17
9,00	0,14	41,50	4,00	2,15
8,00	0,14	41,19	4,00	2,12
7,00	0,15	40,83	4,00	2,08
6,00	0,15	40,41	4,00	2,04
5,00	0,16	39,90	4,00	1,99
4,00	0,16	39,27	4,00	1,93
3,00	0,18	38,45	4,00	1,85
2,00	0,19	37,24	4,00	1,73
1,00	0,22	35,03	4,00	1,53
$\Sigma =$				152,21

With the wind forces values obtained along the tower per meter, it was possible to obtain the shear force diagram, presented below:

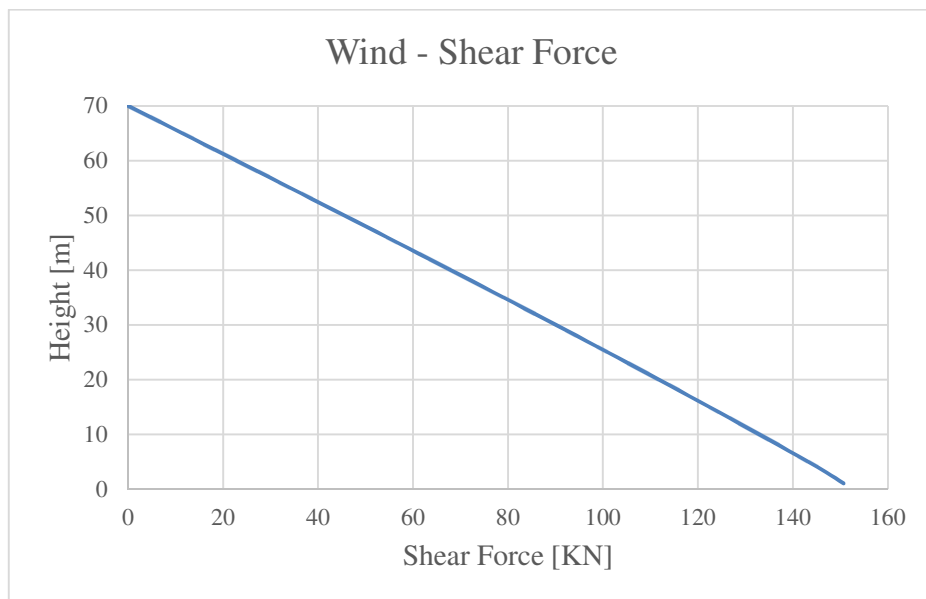


Figure 4 - 7: Wind shear force diagram

Integrating the shear force diagram, the following bending moment diagram is obtained, as:

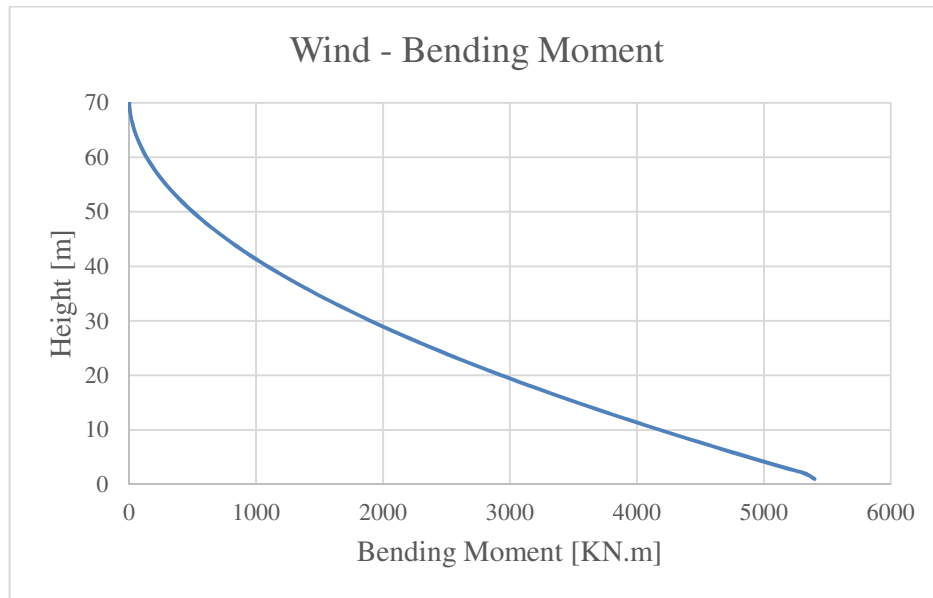


Figure 4 - 8: Wind bending moment diagram

4.5.3.2.2. AERODYNAMIC LOADS ON THE ROTOR AND BLADES

For the design of the wind turbine it is going to be considered two situations. The first one, the rotor in operation mode and the second one the rotor completely stopped and the wind acting directly on the blades of the rotor.

4.5.3.2.2.1. TURBINE IN OPERATION MODE

For the first situation, to determinate the wind force on the rotor during operation mode it was used the equation 3.49. At hub height the wind velocity is 46,85 m/s, but the wind speed considered is 25 m/s due to the safety system installed on the wind turbine, which stops the rotor for velocities above this value. The obtained wind force on the rotor during operation mode is 142,19 KN.

4.5.3.2.2.1. TURBINE STOPPED

For the second situation it is needed to calculate the aerodynamic drag and lift force on the blades. The wind velocity at the center of mass of the two blades turned down is 46,49 m/s and at the center of gravity of the blade turned up is 47,53 m/s. With these wind speeds is possible to obtain the relative velocity through expression 3.52 for each case.

Due to the lack of information, the blades type were assumed as NACA N63-212 airfoil and the drag (C_D) and lift (C_L) coefficient can be taken by the graph in Figure 4-9. This graph must be provided by the manufacturer and is shown below:

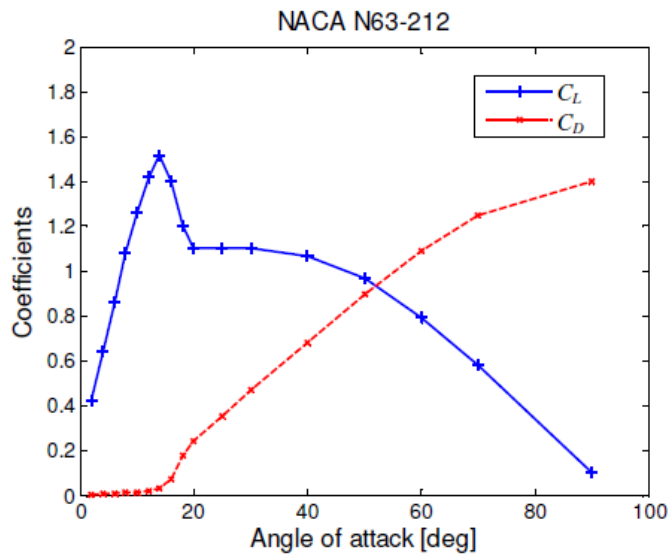


Figure 4 - 9: Drag and lift coefficient curve per angle of attack for the NACA N63-212 airfoil, [60]

For an attack angle of 10 degrees the drag coefficient is about 0,03 and the lift coefficient is 1,1 degrees. Now, with this coefficient values, as well as the relative wind speed it is possible to calculate the aerodynamic drag and lift force through the expressions 3.50 and 3.51. After this, applying the equation 3.59 is obtained the load per blade element, which for the two blades turned down the value is 90,44 KN and for the blade turned up is 91,37 KN.

The total force and bending moment applied at the top of the tower due, to the generated wind forces on the rotor, are:

$$F = 91,37 + 90,44 \times 2 = 272,25 \text{ KN}$$

$$M = -91,37 \times 19,5 + (90,44 \times 9,75) \times 2 = -18,14 \text{ KN.m}$$

All the loads values that are considered in this study according to Chapter 3, were already presented, so the next step is to calculate the structure internal forces generated by the load effects.

4.7. FIRST DESIGN APPROACH OF THE STRUCTURE

On a first design approach of the structure, the analysis is going to be performed on a simplified model, considering that the monopile is fixed at the seabed level. As stated in previous Chapter, that the model corresponds to a cantilever element according the DNV standards recommendations (2014).

4.7.1. STRUCTURAL INTERNAL FORCES

Considering the structure as a cantilever beam, the most unfavourable section is the base at the seabed level. Using the Robot Structural Analysis program it is going to be analysed which of the two situations above develops a higher internal forces due to the applied loads. After this observation, a safety verification of the structure for the obtained internal forces is going to be done.

Beyond the combinations considered from DNV standard in subchapter 3.3 it is going to be added two more combinations from Eurocode 1 (EN1991), combination 3 and 4. Therefore, the combinations contemplated in this study are listed below:

✓ DNV combinations:

- Combination 1: $S_d = 1,25G + 0,7(S_{\text{wave}} + S_{\text{wind}})$
- Combination 2: $S_d = 1,00G + 1,35(S_{\text{wave}} + S_{\text{wind}})$

✓ Eurocode combinations:

- Combination 3: $S_d = 1,00G + 1,5(S_{\text{wave}} + \Psi_{0,S_{\text{wind}}} \times S_{\text{wind}})$
- Combination 4: $S_d = 1,00G + 1,5(\Psi_{0,S_{\text{wave}}} \times S_{\text{wave}} + S_{\text{wind}})$

Where, $\Psi_{0,S_{\text{wave}}} = \Psi_{0,S_{\text{wind}}} = 0,4$ and G represents the permanent loads, S_{wave} the wave action and S_{wind} the wind action.

Due, to the two different considerations of the aerodynamic loads on the rotor, it will be seen which one leads for the worst scenario, being presented the internal forces values as follows:

- Rotor in operation mode, considering a maximum wind speed of 25 m/s at hub height;

Table 4 - 10: Combinations results for rotor during operation mode

	Z (m)	N (KN)	V (KN)	M(KN.m)
Combination 1	83,50	1452,60	100,97	0,00
	22,50	4186,05	195,39	8957,13
	0,00	5529,92	793,55	19011,63
Combination 2	83,50	1162,08	194,73	0,00
	22,50	3348,84	376,82	17274,47
	0,00	4423,94	1530,41	36665,29
Combination 3	83,50	1162,08	86,55	0,00
	22,50	3348,84	167,48	7677,54
	0,00	4423,94	1441,83	23422,16
Combination 4	83,50	1162,08	216,37	0,00
	22,50	3348,84	418,69	19193,86
	0,00	4423,94	938,81	33612,73

The values in bold refer to the highest internal forces at each respective high. The 0,00 meters correspond to the seabed level, the 22,50 meters contemplates the monopile length and the transition piece which was assumed that has the same characteristics of the monopile, in terms of diameter, in order to simplify the design. Added to those 22,50 meters there are 61 meters which correspond to the tower's height, performing a total of 83,50 meters.

- Rotor completely stopped and the wind acting directly on the blades of the rotor;

Table 4 - 11: Combinations results for rotor completely stopped

	Z (m)	N (KN)	V (KN)	M(KN.m)
Combination 1	83,50	1452,60	192,02	12,70
	22,50	4186,05	286,43	14523,39
	0,00	5529,92	884,59	26626,34
Combination 2	83,50	1162,08	370,32	24,49
	22,50	3348,84	552,40	28009,40
	0,00	4423,94	1705,99	51350,79
Combination 3	83,50	1162,08	164,58	10,88
	22,50	3348,84	245,51	12448,62
	0,00	4423,94	1519,87	29949,05
Combination 4	83,50	1162,08	411,46	27,21
	22,50	3348,84	613,78	31121,56
	0,00	4423,94	1133,90	49929,96

Observing the internal forces presented in the tables, it is possible to conclude that the worst situation occurs when the wind turbine is completely stopped. For this case, the respective internal forces diagrams obtained from the Robot Structural analysis program, for the worst combinations, are presented below:

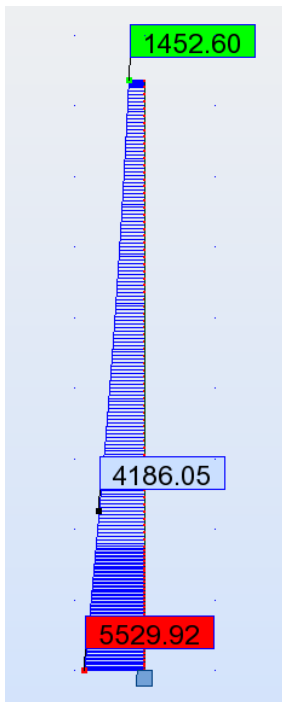


Figure 4 - 10 Combination 1, axial internal forces

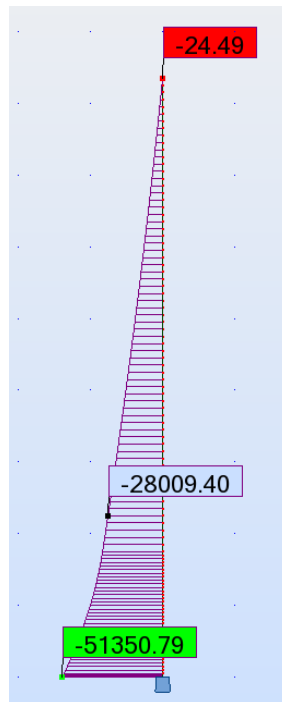


Figure 4 - 11 Combination 2, bending moment internal forces

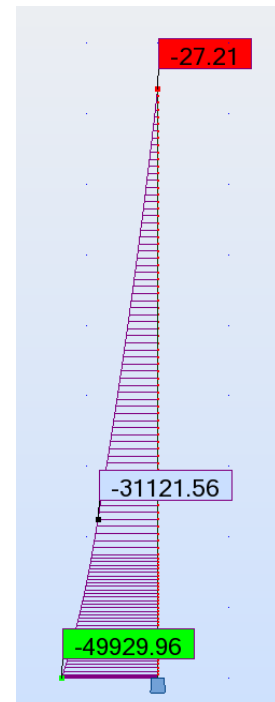


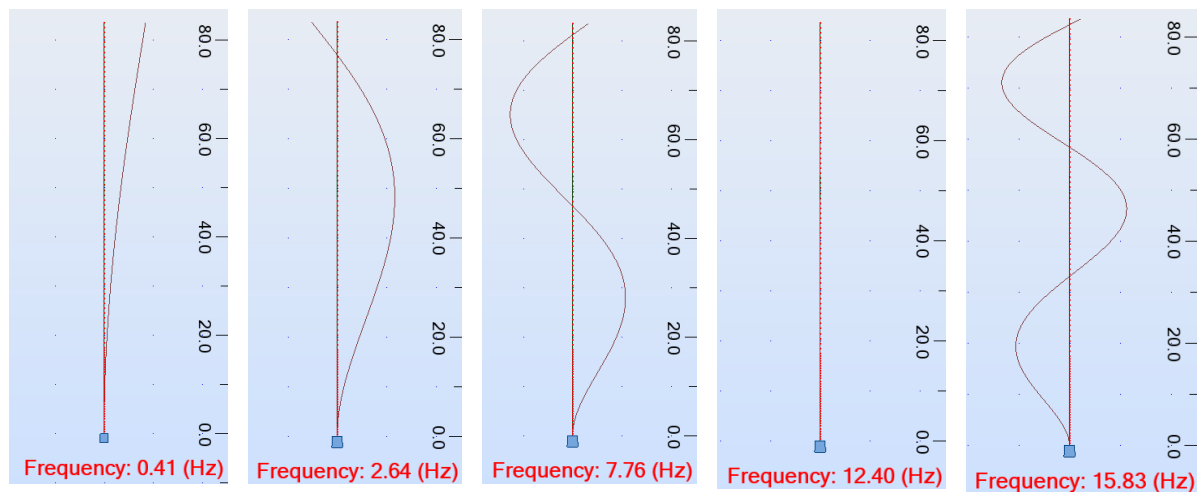
Figure 4 - 12 Combination 4, bending moment internal forces

The diagram concerning the axial forces for combination 1 has a higher values because of the load coefficient factor which is 1,25 for the permanent loads and for the others combinations this coefficient is 1,0.

Relatively to the bending moment diagrams, for combination 2, the wave and wind loads results are increased by the coefficient factor of 1,35. However for combination 4 the coefficient factors decrease to 0,6 for the wave loads and increase 1,5 for the wind load. This, justifies the higher results obtained at the base of the structure for combination 2 when compared to combination 4, as well as the values at the sea level and at the top tower, which are higher for combination 4 and lower for combination 2.

4.7.2. MODULES ANALYSIS

The fundamental natural frequency of the structure was determined by the Robot Structural Analysis program. The other nine vibrations modes that are presented below, correspond to the possible vibration modes of the structure, when it is subject to a dynamic force. The vibration modes are related to the mass of the structure and to the support conditions.



The most important frequency used for design, is the fundamental natural one, which corresponds to the first vibration mode of the structure. Like stated in Figure 4-13 this natural frequency must move as much as possible away from the wind, wave and rotor frequency in order to avoid resonance. Note that this was already explained in subchapter 3.4.

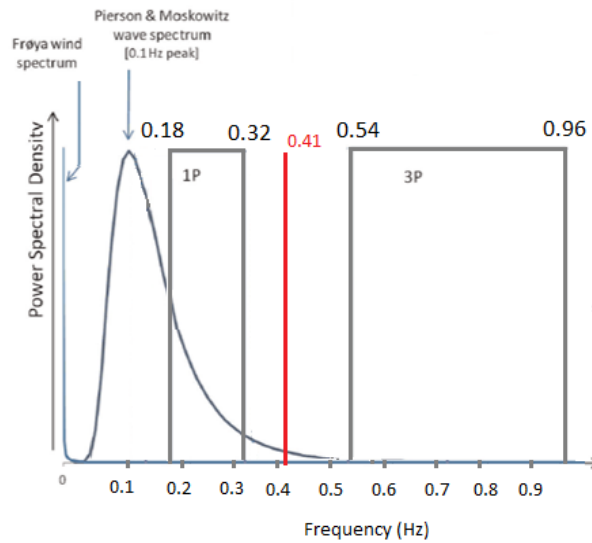
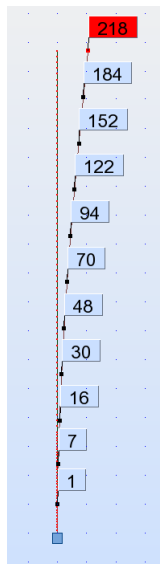
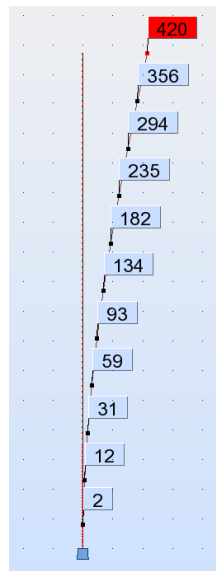
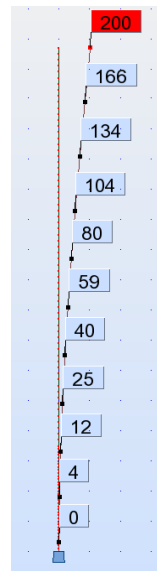
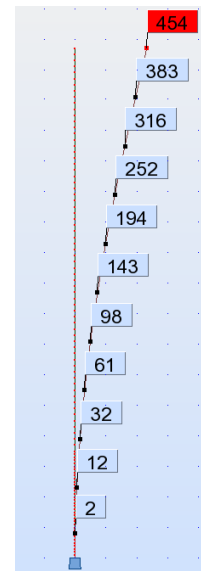


Figure 4 - 18: Diagram showing natural frequency and excitation frequencies

In figure 4-23 it is represented the first natural frequency of the structure, which is on the desired range that was between 0,32 and 0,54 Hz, this corresponds to the rotor frequencies, as it was explained also in subchapter 3.4.

4.7.3. STATIC BEHAVIOUR

The following figure provides the results of the displacements (in mm) computation, regarding the four loads combinations. The results vary according to the combination considered:

Figure 4 - 19:
Combination 1 –
Structure displacementsFigure 4 - 20:
Combination 2 -
Structure displacementsFigure 4 - 21:
Combination 3 -
Structure displacementsFigure 4 - 22:
Combination 4 -
Structure displacements

The maximum displacement was obtained for the combination 4 with 0,454 m and the maximum displacement allowed is determined through equation 3.69, which is approximated to 1% of the structure length, so 0,835 m. Thereby, the structure is within limits of the maximum deflection allowed.

4.7.4. CROSS SECTION DESIGN VERIFICATIONS

4.7.4.1. STRUCTURE STABILITY

The type of steel assumed for the monopile was the S235.

Section classification: $\frac{d}{t} \leq 90 \rightarrow$ Class 3 cross-section

Table 4 - 12: Cross-section classification

Z [m]	D [m]	t [m]	D/t	Section Classification
83,50	3,00	0,035	85,71	class 3 cross-section
22,50	4,00	0,050	80,00	class 3 cross-section
0,00	4,00	0,050	80,00	class 3 cross-section

After the classification of the cross-section it was possible to obtain the buckling values in Table 4-13, through the correct application of the expressions presented in subchapter 3.6 for the class 3 cross-section.

Table 4 - 13: Buckling results

Z [m]	N _{cr} [KN]	$\bar{\lambda}$	N _{c,Rd} [KN]	Φ	x	N _{b,Rd} [KN]
83,50	26628,62	1,70	76614,41	2,10	0,30	23032,16
22,50	89944,96	1,27	145809,17	1,42	0,49	70807,23
0,00	89944,96	1,27	145809,17	1,42	0,49	70807,23

The following axial and bending moment internal forces values for the rotor stopped scenario, are taken from the internal forces diagrams presented above, being considered the higher internal forces values in order, to make a design for the worst scenario. In terms of economical design this is not the best way to act, we should considered only one combination, for example, combination 2 due to the higher bending moment value at the base of the structure which is the critical area.

Table 4 - 14: Internal forces results

Z [m]	N _{ed} [KN]	M _{ed} [KN.m]
83,5	1452,6	27,21
22,5	4186,05	31121,56
0,00	5529,92	51350,79

In order, to analyse the resistance of the cross-section due to axial compressed elements, the equation 3.73 must be verified:

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1,0$$

Table 4 - 15: Resistance of the cross-section

Z [m]	N_{ed}/N_{c,Rd}	Failure
83,50	0,02	No
22,50	0,03	No
0,00	0,04	No

For compressed elements it is also to be verified the equation 3.76:

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

Table 4 - 16: Compressed elements condition

Z [m]	Failure
83,50	No
22,50	No
0,00	No

To evaluate the sectional stability it is verified the equation condition 3.83:

$$\frac{N_{Ed}}{A} + \frac{M_{Ed}}{I} \times y \leq \sigma_y$$

Table 4 - 17: Sectional stability verification

Z [m]	σ [KPa]	σ_y [KPa]	Failure
83,50	4569,480377	235000	No
22,50	58174,75217		No
0,00	93769,29241		No

For the member stability it is verified the equation condition 3.84 expressed below:

$$\frac{N_{Ed}}{x \times N_{pl,Rd}} + \frac{k \times M_{Ed}}{M_{el,Rd}} \leq 1,0$$

Table 4 - 18: Member stability verification

Z [m]	N_{pl,Rd} [kN]	W_{el} [m³]	M_{el,Rd} [kN.m]	Equation 4.13	Failure
83,50	76614,41	0,24	56135,70	0,06	No
22,50	145809,17	0,61	142209,51	0,28	No
0,00	145809,17	0,61	142209,51	0,44	No

At the end of the safety verification calculations, it was observed that due to the given internal forces, applied in the structure, the respective elements are able to resist and fulfil the presented safety criteria.

If they did not fulfil such criteria, the alternative would be to increase the diameter of the pile or its own thickness in order to verify the condition of equation 3.84. However, since an increasing of the diameter or thickness leads to higher weights, it should be noted that the axial forces would increase as well. Besides that, an equilibrium must be reached in terms of the optimum design that resists to the loads applied and the costs generated by the new materials that might have better resistances but higher prices.

4.7.3.2. SHARING VORTICES

To verify the sharing vortices phenomenon it is necessary to determine the critical velocity through expression 3.90 in order, to verify the following condition $U_{cr} \leq 0,2 \times U_{project}$.

Table 4 - 19: Sharing vortices verification

Z [m]	f [Hz]	D [m]	St	U _{cr} [m/s]	U _{project} [m/s]	Failure
83,50	0,41	3,00	0,20	6,15	46,85	No
22,50		4,00		8,20	41,50	No

4.7.3.3. SECTION ROUNDNESS

If the condition $D/e < 250$ is verified, it is not necessary to use stiffening rings and the roundness is controlled, as it is seen in Table 4-20:

Table 4 - 20: Roundness verification

Z [m]	D [m]	e [m]	D/e	Failure
83,50	3,00	0,035	85,71	No
22,50	4,00	0,050	80,00	No
0,00	4,00	0,050	80,00	No

4.8. SECOND DESIGN APPROACH OF THE STRUCTURE

On a second design approach, it is considered that the structure is founded 25 m into the seabed and has three degrees of freedom at the seabed level. This analysis, represents the reality in a more accurate way, as it considers the dynamic soil-structure iteration.

4.8.1. DYNAMIC SOIL-STRUCTURE ITERATION

For this study it is represented in Table 4-21 the soil characteristics in order to determine the stiffness of each stratum.

Table 4 - 21: Soil profile, including the parameters of each stratum (A. Augustesen, 2009)

Stratum	Type of Soil	Depth [m]	Es [Mpa]	γ [KN/m ³]	Φ [°]	ν [-]
1	Sand	0 - 4,5	130	20	45,4	0,28
2	Sand	4,5 - 6,5	114,3	20	40,7	0,28
3	Sand to silty sand	6,5 - 11,9	100	20	38	0,28
4	Sand to silty sand	11,9 - 14	104,5	20	36,6	0,28
5	Sand	18,2 →	168,8	20	38,7	0,28

The values presented on the below table, were obtained through the application of the expressions in subchapter 3.7.2, for shallow depths waters:

Table 4 - 22: Parameters for shallow depths waters

Stratum	C1 [-]	C2 [-]	x [m]	σ_v [Kpa]	D [m]	Pult [KN/m]	Ksand [Kpa/m]
1	7,93	5,69	2,25	22,95	4,00	932,33	66692,67
2	5,12	4,49	5,50	56,10	4,00	2586,04	44898,23
3	3,98	3,91	9,20	93,84	4,00	4903,79	33909,80
4	3,49	3,65	15,05	153,51	4,00	10305,13	28637,89
5	4,25	4,06	21,60	220,32	4,00	23783,46	36653,95

Through the implementation of the equations for deep depths waters exposed also in subchapter 3.7.2, it gave the following results, in Table 4-23:

Table 4 - 23: Parameters for deep water depths

Stratum	C3 [-]	x [m]	σ_v [Kpa]	D [m]	Pult [KN/m]	Ksand [Kpa/m]
1	66692,67	2,25	22,95	4,00	19623,44	66692,67
2	44898,23	5,50	56,10	4,00	26309,06	44898,23
3	33909,80	9,20	93,84	4,00	31165,96	33909,80
4	28637,89	15,05	153,51	4,00	42631,36	28637,89
5	36653,95	21,60	220,32	4,00	80019,61	36653,95

Observing the two tables it is possible to see that the lower results for the ultimate lateral capacity p_{ult} were obtained for shallow depths waters, so this is the case that is going to be considered in order, to determinate the soil stiffness. Acting this way, we are on the safe side because it is the worst case scenario.

The p-y curves of each stratum were drawn by applying the equation 3.96. An approximation of results was made in order to obtain the relation between the soil lateral resistance and the soil displacement, like it is presented through the below graphs:

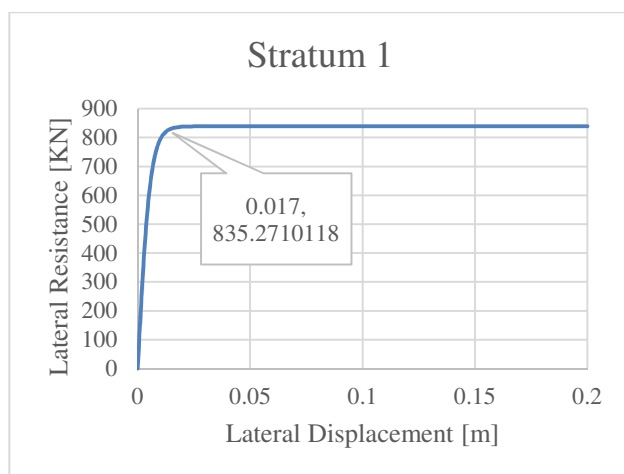


Figure 4 - 23: P-Y curves for stratum 1

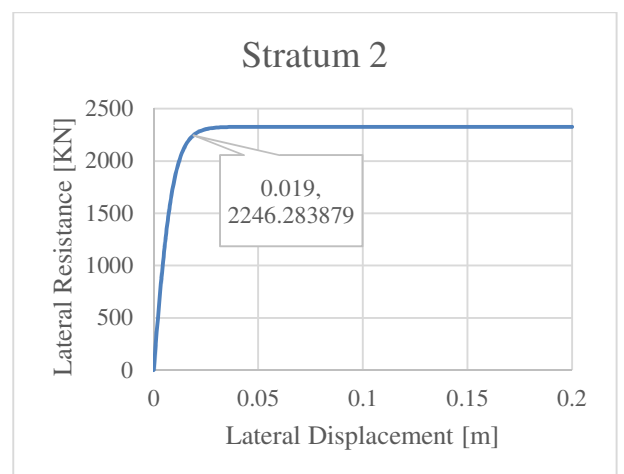


Figure 4 - 24: P-Y curves for stratum 2

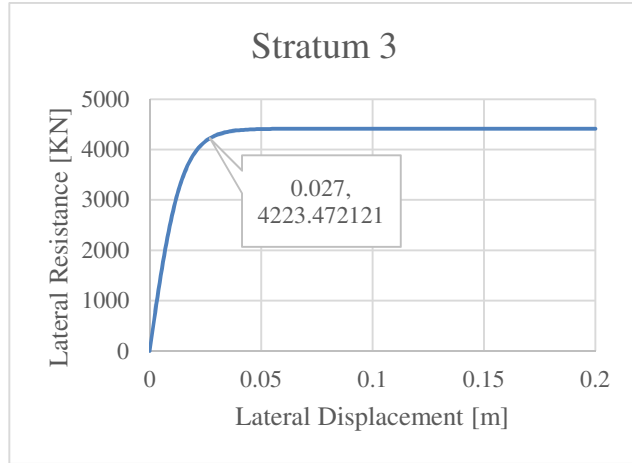


Figure 4 - 25: P-Y curves for stratum 3

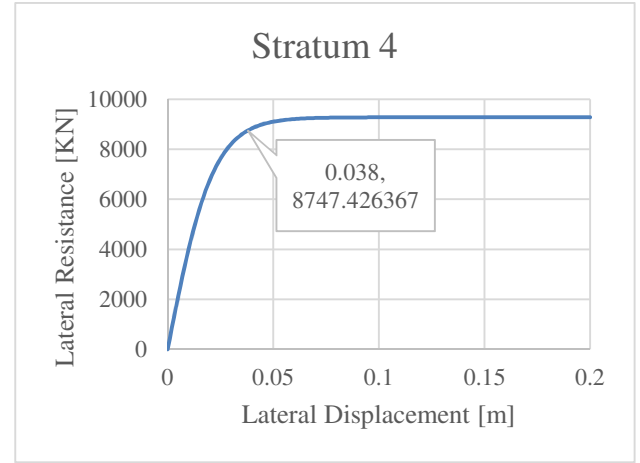


Figure 4 - 26: P-Y curves for stratum 4

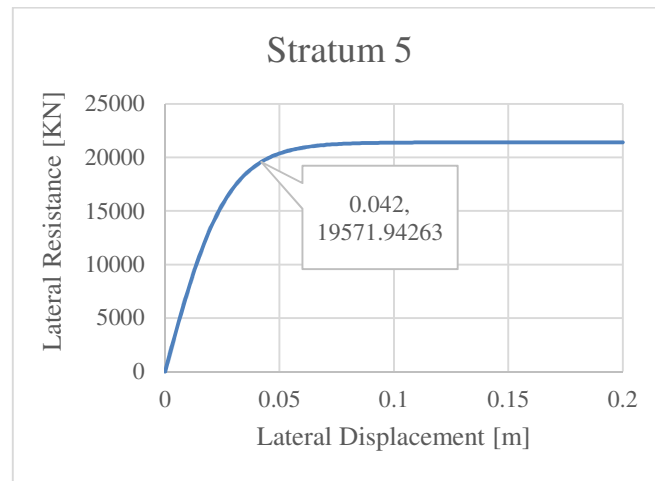


Figure 4 - 27: P-Y curves for stratum 5

Through the values of the graphs it was obtained the stiffness of each stratum, as shown in table 4-24:

Table 4 - 24: Stratum stiffness

Stratum	y [m]	p [kN]	K [kN/m]
1	0,017	835,27101	49133,59
2	0,019	2246,28388	118225,47
3	0,027	4223,47212	156424,89
4	0,038	8747,42637	230195,43
5	0,042	19571,94263	465998,63

4.8.2. STRUCTURAL INTERNAL FORCES

Using once more, the Robot Structural Analysis program it is going to be checked which of the two situations are the worst scenario, the same as it was done for the first situation above. On this study it is considered the “real structure”, where the analysis of the soil-structure interaction is done through springs defined with the respective stiffness of each stratum.

The combination introduced in the program are the same presented in subchapter 4.71, being exposed below:

✓ DNV combinations:

- Combination 1: $S_d = 1,25G + 0,7(S_{\text{wave}} + S_{\text{wind}})$
- Combination 2: $S_d = 1,00G + 1,35(S_{\text{wave}} + S_{\text{wind}})$

✓ Eurocode combinations:

- Combination 3: $S_d = 1,00G + 1,5(S_{\text{wave}} + \Psi_{0,S_{\text{wind}}} \times S_{\text{wind}})$
- Combination 4: $S_d = 1,00G + 1,5(\Psi_{0,S_{\text{wave}}} \times S_{\text{wave}} + S_{\text{wind}})$

Where, $\Psi_{0,S_{\text{wave}}} = \Psi_{0,S_{\text{wind}}} = 0,4$ and G represents the permanent loads, S_{wave} the wave action and S_{wind} the wind action.

The values of the internal forces for the two aerodynamic situations that are being study, for the new support conditions are represented in the tables below:

- Rotor in operation mode, considering a maximum wind speed of 25 m/s at hub height;
-

Table 4 - 25: Combinations results for rotor during operation mode

	Z (m)	N (KN)	V (KN)	M(KN.m)
Combination 1	108,50	1452,60	100,97	0,00
	47,50	4448,13	220,46	9948,64
	25,00	5493,36	806,70	21428,69
	0,00	6986,55	-1500,01	0,00
Combination 2	108,50	1162,08	194,73	0,00
	47,50	3558,50	425,18	19186,66
	25,00	4394,69	1555,78	41326,76
	0,00	5589,24	-2892,88	0,00
Combination 3	108,50	1162,08	86,55	0,00
	47,50	3558,50	213,80	8527,40
	25,00	4394,69	1470,02	28544,75
	0,00	5589,24	-2084,93	0,00
Combination 4	108,50	1162,08	216,37	0,00
	47,50	3558,50	447,59	21318,51
	25,00	4394,69	950,08	35750,42
	0,00	5589,24	-2471,11	0,00

Like it was mentioned on the first case, the values in bold refer to the higher internal forces at each respective high. Once again the 0,00 meters corresponds to beginning of the monopile foundation, the 25 meters are at the seabed level. After that, the top of the monopile and the transition piece are included until the 47,5 meters. Finally, with the tower's height (61 meters) being added, the final height is 108,5 meters.

- Rotor completely stopped and the wind acting directly on the blades of the rotor;

Table 4 - 26: Combinations results for rotor completely stopped

	Z (m)	N (KN)	V (KN)	M(KN.m)
Combination 1	108,50	1452,60	192,02	12,70
	47,50	4448,13	311,50	15970,11
	25,00	5493,36	897,74	28966,22
	0,00	6986,55	-2002,22	0,00
Combination 2	108,50	1162,08	370,32	24,49
	47,50	3558,50	600,76	30799,50
	25,00	4394,69	1731,36	55863,43
	0,00	5589,24	-3861,43	0,00
Combination 3	108,50	1162,08	164,58	10,88
	47,50	3558,50	291,83	13688,66
	25,00	4394,69	1548,06	35005,49
	0,00	5589,24	-2500,93	0,00
Combination 4	108,50	1162,08	411,46	27,21
	47,50	3558,50	642,68	34221,66
	25,00	4394,69	1145,17	52506,60
	0,00	5589,24	-3588,67	0,00

Observing the internal forces presented in the tables, it is possible to conclude that the worst situation occurs when the wind turbine is completely stopped.

For this case the respective internal forces diagrams obtained from the Robot Structural analysis program are exposed below:

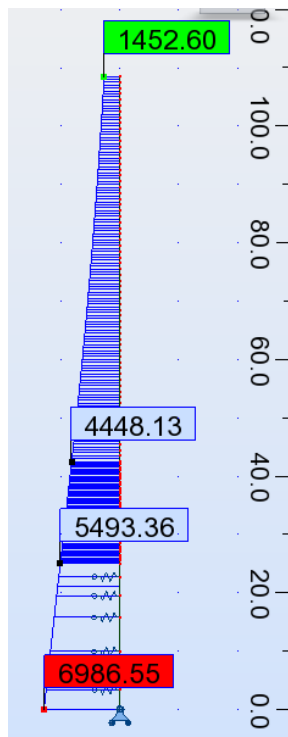


Figure 4 - 28: Combination 1, axial internal forces

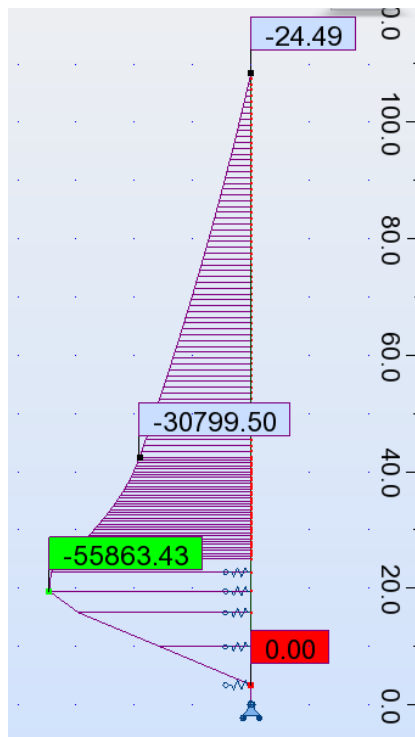


Figure 4 - 29: Combination 2, bending moment internal forces

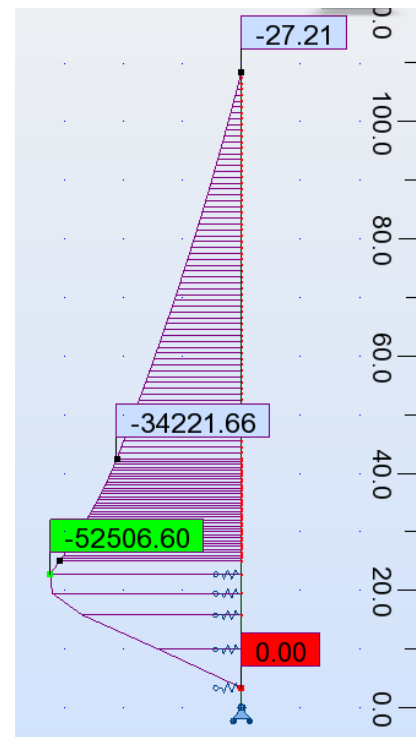


Figure 4 - 30: Combination 4, bending moment internal forces

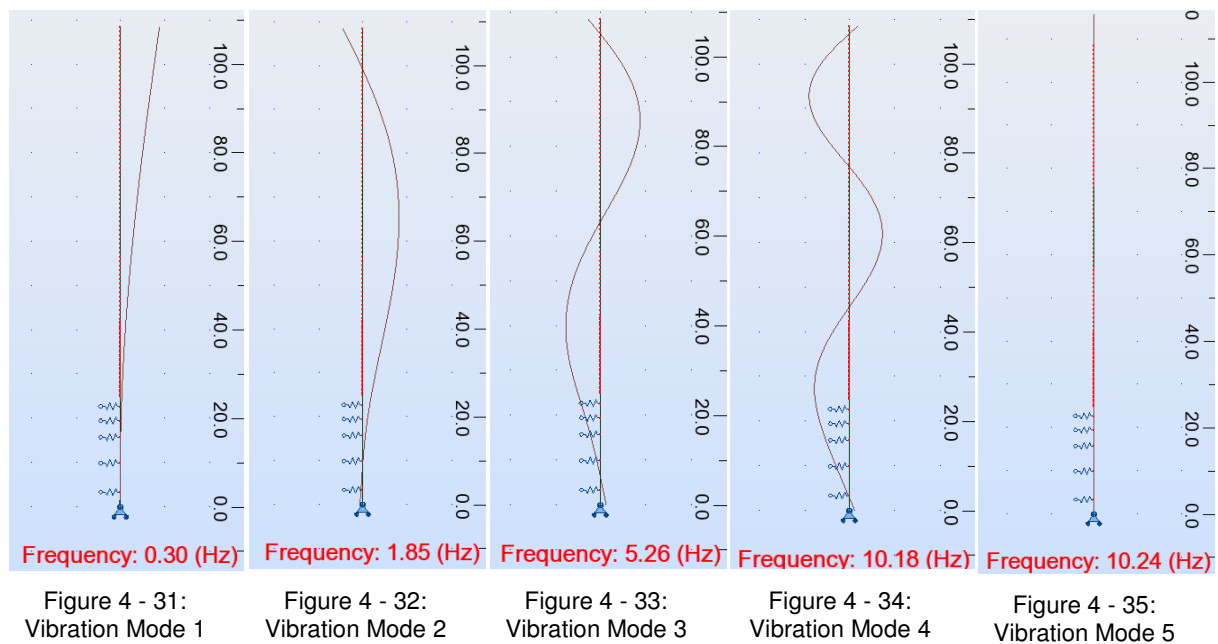
On the bending moment diagrams, the internal forces' values next to the vertical simple support are zero due, to the lattice of the vertical support.

Like it was explained for the first case, the axial internal forces diagram for combination 1 has a higher values because of the load coefficient factor which is 1,25 for the permanent loads and for the others combinations this coefficient is 1,0.

Relatively to the bending moment diagrams, it also happens the same as case one (cantilever element), due to the combinations that remain the same for the two analysed cases. For combination 2 the wave and wind loads results are increased by the coefficient factor of 1,35. However for combination 4 the coefficient factors decreases to 0,6 for the wave loads and increases to 1,5 in the wind load. This, justifies the higher forces results obtained at the base of the structure for combination 2, when compared to the combination 4, as well as the values at the sea level and at the top tower which are higher for combination 4 and lower for combination 2.

4.8.3. MODULES ANALYSIS

For the new support conditions it was obtained the fundamental natural frequency and the other respective vibration modes of the structure, through the same way as it was determined for the first case, being presented below:



The first fundamental natural frequency of the structure for this second case is lower than the first one due, to the higher flexibility of the structure compared to the first case. This, can be proved through equation 3.66, the bigger is the stiffness, the higher is the frequency of the structure.

Like it was explained above on case one (cantilever element), the relevant frequency for the design of the structure is the first vibration mode, which corresponds to the natural frequency. The natural frequency must move away from the wind, wave and rotor frequency in order to avoid resonance, but like it shows in Figure 4-46 this does not happened. The natural frequency is coincident with the rotor frequency, so, in order to avoid future problems with the stability of the structure, it is needed to increase the stiffness of the structure, by increasing the monopile thickness or diameter.

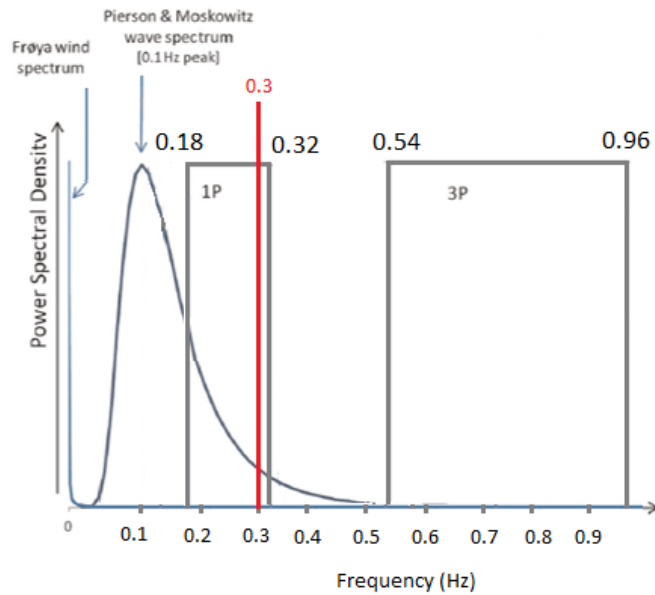


Figure 4 - 36: Diagram showing natural frequency and excitation frequencies

Figure 4-46 shows the desired frequency range where the natural frequency must be, which is between 0,32 and 0,54 Hz, corresponding to the rotor frequencies.

4.8.4. STATIC BEHAVIOUR

To the four loads combinations and for the new support conditions, it is represented in the following figures the displacements values (in mm), for each combination:

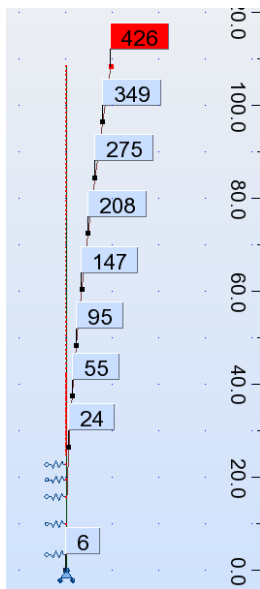


Figure 4 - 37:
Combination 1 –
Structure displacements

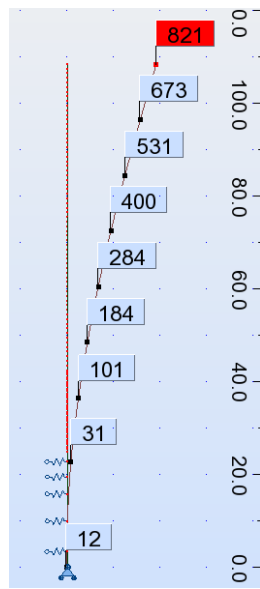


Figure 4 - 38:
Combination 2 - Structure
displacements

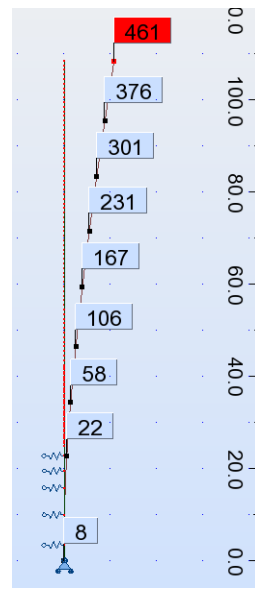


Figure 4 - 39:
Combination 3 - Structure
displacements

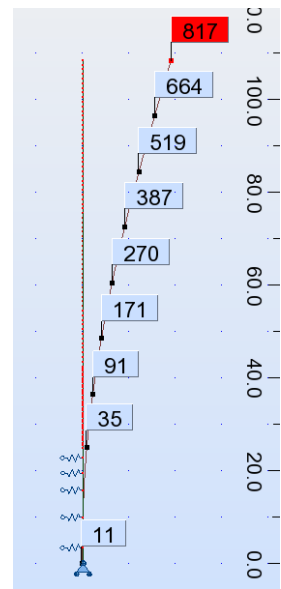


Figure 4 - 40:
Combination 4 - Structure
displacements

The maximum displacement was obtained for the combination 2 with 0,821 m and the maximum displacement allowed is determinate through equation 3.69, which is approximated to 1% of the structure length, so 1,085 m. Thereby, the structure is within limits of the maximum deflection allowed.

For this second case, when compared with the first one, the displacements results are higher due, to the shear force values which are also higher than the shear force of case one.

4.8.5. CROSS SECTION DESIGN VERIFICATIONS

4.8.5.1. STRUCTURE STABILITY

The type of steel assumed for the monopile was the S235.

Section classification: $\frac{d}{t} \leq 90 \rightarrow$ Class 3 cross-section

Table 4 - 27: Cross-section classification

Z [m]	D [m]	t [m]	D/t	Section Classification
108,50	3,00	0,035	85,714	class 3 cross-section
47,50	4,00	0,050	80,000	class 3 cross-section
25,00	4,00	0,050	80,000	class 3 cross-section
0,00	4,00	0,050	80,000	class 3 cross-section

After the classification of the cross-section it was possible to obtain the buckling values in table 4-28, through the correct application of the expressions presented in subchapter 3.6 for the class 3 cross-section.

Table 4 - 28: Buckling values

Z [m]	N _{cr} [KN]	$\bar{\lambda}$	N _{c,Rd} [KN]	Φ	x	N _{b,Rd} [KN]
108,50	26628,62	1,70	76614,41	2,10	0,30	23032,16
47,50	89944,96	1,27	145809,17	1,42	0,49	70807,23
25,00	89944,96	1,27	145809,17	1,42	0,49	70807,23
0,00	89944,96	1,27	145809,17	1,42	0,49	70807,23

The following axial and bending moment efforts values for the rotor stopped scenario, are taken from the internal forces diagrams presented above:

Table 4 - 29: Internal forces results

Z [m]	N _{ed} [KN]	M _{ed} [KN.m]
108,50	1452,60	27,21
47,50	4448,13	34221,66
25,00	5493,36	55863,43
0,00	6986,55	0,00

In order, to analyse the resistance of the cross-section due to axial compressed elements, the equation 3.73 must be verified:

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1,0$$

Table 4 - 30: Resistance of the cross-section

Z [m]	Ned/Nc,Rd	Failure
108,50	0,02	No
47,50	0,03	No
25,00	0,04	No
0,00	0,05	No

For compressed elements we must also verify equation 3.76:

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

Table 4 - 31: Compressed elements condition

Z (m)	Failure
108,50	No
47,50	No
25,00	No
0,00	No

To evaluate the sectional stability it is verified the equation condition 3.83:

$$\frac{N_{Ed}}{A} + \frac{M_{Ed}}{I} \times y \leq \sigma_y$$

Table 4 - 32: Sectional stability verification

Z [m]	σ [KPa]	σ_y [KPa]	Failure
108,50	4569,48	235000	No
47,50	63720,03		No
25,00	101167,47		No
0,00	11260,19		No

For the member stability it is verified the equation condition 3.84 expressed below:

$$\frac{N_{Ed}}{x \times N_{pl,Rd}} + \frac{k \times M_{Ed}}{M_{el,Rd}} \leq 1,0$$

Table 4 - 33: Member stability verification

Z [m]	Npl,Rd [KN]	Wel [m3]	Mel,Rd [KN.m]	Equation 4.13	Failure
108,50	76614,41	0,24	56135,70	0,06	No
47,50	145809,17	0,61	142209,51	0,30	No
25,00	145809,17	0,61	142209,51	0,47	No
0,00	145809,17	0,61	142209,51	0,10	No

4.8.5.2. SHARING VORTICES

To verify the sharing vortices phenomenon it is necessary to determine the critical velocity through expression 3.90 in order, to verify the following condition $U_{cr} \leq 0,2 \times U_{project}$.

Table 4 - 34: Sharing vortices verification

Z [m]	f [Hz]	D [m]	St [-]	U_{cr} [m/s]	U_{project} [m/s]	Failure
108,50	0,30	3,00	0,20	4,50	46,85	No
47,50		4,00		6,00	41,50	No

4.8.5.3. SECTION ROUNDNESS

If the condition $D/e < 250$ is verified, it is not necessary to use stiffening rings and the roundness is controlled, like it is seen in Table 4-35:

Table 4 - 35: Roundness verification

Z [m]	D [m]	e [m]	D/e	Failure
108,50	3,00	0,035	85,71	No
47,50	4,00	0,050	80,00	No
25,00	4,00	0,050	80,00	No
0,00	4,00	0,050	80,00	No

After the end of the safety verifications it is observed that do not exists failure of the structure elements, the same, like it was viewed in the previous scenario. If the structure does not resist to the effects that is subjected, so it is necessary to increase the thickness or diameter of the pile, as it was said on the case before.

5

Conclusions and Future Developments

5.1 CONCLUSIONS

The present thesis was developed in order to provide a preliminary analysis of an offshore wind turbine, located at the North Sea. A case study concerning to the Danish offshore wind farm Horns Rev I was used.

The present research work enabled the opportunity of contributing for a deeper knowledge in offshore structures design and allowed for a significant technological review to be performed.

This dissertation provided a preliminary design study of an offshore wind turbine with a monopile foundation, which is the most usual substructure type for offshore wind projects, located at shallow and intermediate water depths. Besides the structural analysis and design, this research included the fundamentals and computations for the hydrodynamic loads, which are usually performed within the hydraulics project. Such calculations have required the investigation and study of several concepts linked to offshore and marine engineering, which aren't usually approached by non-specialised engineers. A review of the methodologies applied to calculate such loads was successfully concluded and a starting point on that matter is provided for future structural research in this area.

The presentation and summary of the several types of foundations existent was done, allowing for a better and solid description, of the main components used in an offshore wind turbine structure.

A comprehensive description of the methodologies used to compute the environmental loads calculations was also provided.

The offshore engineering, particularly the offshore wind market is gaining a considerable and growing importance in our society, both in terms of development and resources management. Besides that, the economic factors are playing a major role in the energy market demand. Therefore new challenges arise requesting innovative approaches and deeper knowledge, to provide proper structural designs. In this sense, this research intended to provide an integrated contribute for an integrated approach where the both hydraulics and structural specifications are successfully combined towards a common end.

5.2 FUTURE WORKS

This investigation led to interesting conclusions, but it also pointed out the important aspects that should be considered as starting points for future research.

Some of these technical aspects are listed below as possible issues to be considered in future works, as a development of the present dissertation:

- Extend and establish a more detailed comparison between the several guidelines used to design these type of structures;
- Consider a more complex model of the offshore structures, where the critical nodes as the transition piece, joints and other elements are further analysed;

- Develop the verifications and design requirements, for the present case study, in order to consider the corrosion phenomena and the FLS, SLS and ALS limit states;
- Provide a detailed study on the critical section, where the monopile penetrates the seabed and if possible with proper scour protection analysis, combining the structural criteria given by hydraulics and structural concepts;
- Verify the possibilities of diameter's optimisation according to the calculations and verifications hereby developed.

In terms of other general fields of research, the consideration of complex approaches using finite elements (e.g. USFOS, FAST), reliability analysis and risk analysis also seem to be general issues that can be coupled with the present research, aiming for an improvement of the work already performed.

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