corewind

D1.2 DESIGN BASIS

INNOSEA / COBRA / DTU / JDR / UPC / FIHAC / UL INT GMBH / USTUTT

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1 ACRONYMS

Abbreviation	Description
ALS	Accidental Limit State
CA	Consortium Agreement
CFS	Certificate on Financial Statement (audit report)
DLC	Design Load Case
DoF	Degree of Freedom
EC	European Commission
EC - GA	(European Commission)-Grant Agreement
EIB	Exploitation and Innovation Board
ESS	Extreme Sea State
ETA	European Technical Approval
ETM	Extreme Turbulence Model
EU	European Union
FLS	Fatigue Limit State
FOWT	Floating Offshore Wind Turbine
GAGA	General Assembly / Grant Agreement
GC	Gran Canaria
GM	Metacentric height
IAB	International Advisory Board
IPR	Intellectual Property Right
LCOE	Levelized Cost of Energy
LODMAT	Lowest observed daily mean air temperature
LRFD	Load and resistance factor design
MB	Morro Bay Site
MSL	Mean sea level
NOAA	National Oceanic and Atmospheric Administration
NSS	Normal Sea State
NTM	Normal Turbulence Model
OEM	Original Equipment Manufacturer
PC	Project Coordinator
PMO	Project Management Office
PR	Periodic Report
SCADA	Supervisory Control And Data Acquisition
SLS	Service Limit State
SSS	Severe Sea State
T&I	Transport and Installation
ULS	Ultimate Limit State
VIM	Vortex Induce Movements

WoB	West of Barra
WoB	West of Barra Island
WP	Work package
WTG	Wind turbine generator

2 INTRODUCTION AND OBJECTIVE

2.1 INTRODUCTION

Floating offshore wind is still a nascent technology and its LCOE is substantially higher than onshore and bottom-fixed offshore wind, and thus requires to be drastically reduced.

The COREWIND project aims to achieve significant cost reductions and enhance performance of floating wind technology through the research and optimization of mooring and anchoring systems and dynamic cables. These enhancements arisen within the project will be validated by means of simulations and experimental testing both in the wave basin tanks and the wind tunnel by taking as reference two concrete-based floater concepts (semi-submersible and spar) supporting large wind turbines (15 MW), installed at water depths greater than 100 m and 200 m for the semi-submersible and spar concept, respectively. Special focus is given to develop and validate innovative solutions to improve installation techniques and operation and maintenance (O&M) activities. They will prove the benefits of concrete structures to substantially reduce the LCOE by at least 15% compared to the baseline case of bottom-fixed offshore wind, both in terms of CAPEX and OPEX. Additionally, the project will provide guidelines and best design practices, as well as open data models to accelerate the further development of concrete-based semi-submersible and spar FOWTs, based on findings from innovative cost-effective and reliable solutions for the aforementioned key aspects. It is aimed that the resulting recommendations will contribute to the cost-competitiveness of floating offshore wind energy, reducing risks and uncertainties and contributing to lower LCOE estimates.

COREWIND aims to strengthen the European leadership on wind power technology (and specially floating offshore wind turbines). To do so, the project consortium has been designed to ensure proper collaboration between all stakeholders (users, developers, suppliers, academia, etc.) which is essential to accelerate commercialization of the innovations carried out in the project.

2.2 OBJECTIVE

The purpose of this document is to define the basis for the analysis and design of the FOWT configurations to be developed during this project. The basis shall define the framework for covering the design of both floater types (spar and semis-submersible) and all their subsystems:

- Wind turbine definition
- Coupled analyses – Global performance
- Structural design
- Mooring design
- Export cable design

It will define the principle design parameters including the operational requirements and applicable codes. The information provided in this document forms the basis for developing a design and guidelines to produce the functional and technical specifications that meets the project requirements.

3 PROJECT DESCRIPTION

The project focuses on the cost reduction of the floating offshore wind power turbines through the optimization of mooring and dynamic cable design. In order to establish a reasonable framework for the analyses two floater types are selected, spar and semi-submersible. The project is structured in ten work packages:

- WP1: Efficient design tools for FOWTs
- WP2: Design and optimization of station keeping systems
- WP3: Dynamic cable design optimization
- WP4: Optimization of O&M strategies and installation techniques
- WP5: Experimental testing
- WP6: LCOE analysis & Life Cycle Assessment
- WP7: Standardizatoin, commercialization and exploitation actions
- WP8: Dissemination and communication
- WP9: Project management
- WP10: Ethics requirements

All analyses are designed to hold a 15MW turbine. The following sections introduces the two different floating technologies to be used.

3.1 WindCrete Floater Description

3.1.1 General Description

WindCrete is a monolithic concrete spar platform including both the tower and the floater in a unique concrete member. The monolithic characteristic means that joints between the tower and the floater are avoided, thus the fatigue resistance is increased since weak points are driven out. The whole structure is in compression state by the use of active reinforcement, and it is designed to avoid traction at any point during the life span of the platform. An overview of the WindCrete is shown in Figure 3.1-1.

The WindCrete can be divided into the following parts:

- **Wind turbine generator (WTG):**

  The substructure is post-tensioned concrete made. Taking advantage of the post-tensioning anchors placed at the top of the tower, a specific steel plate has been designed for the connection between the rotor and nacelle assembly (RNA) and the structure. This round steel plate presents a U-shaped cross section, which acts as a stiff baseplate for the steel tendons, partially or completely closed by an additional upper steel plate, where the connection bolts are placed which connects with the yaw bearing.

- **Concrete Tower:**

  The tower is a truncated cone piece. The width of the tower is set to resist the bending moment during the service life of the structure, with a minimum dimension that allows the placing of the post-tensioning tendons and enough concrete cover to ensure the durability of the active and passive reinforcement.

- **Substructure:**

The substructure is divided in three parts listed in the following:

- Bottom hemi-sphere: The base of the floater presents a hemispheric shape, with the same diameter as the cylinder. This shape is completely favorable in structural terms, distributing the hydrostatic pressures in a compression field around the base, while the post-tensioning steel tendons have continuity along the whole structure. In terms of hydrodynamic properties, the hemispheric shape presents a smaller damping than a flat base.
- Cylinder buoy: The cylinder is the main part that ensures the buoyancy needed as well as allows the placing of the ballast in its base to achieve the needed pitch and roll stiffness.
- Tappered transition piece: The transition between the tower and the floater connects the tower and the floater. This transition is designed to minimize the curvature of the geometry changes, where the losses and deviation forces of post-tensioning are more significant.

- **Station keeping system**

The station keeping system is designed with three mooring lines distributed each 120º with delta arrangement. The fairlead points should be installed close to the center of gravity of the structure to avoid pitch coupling motions that will increase the tension range of the mooring lines, and thus reducing its lifespan.

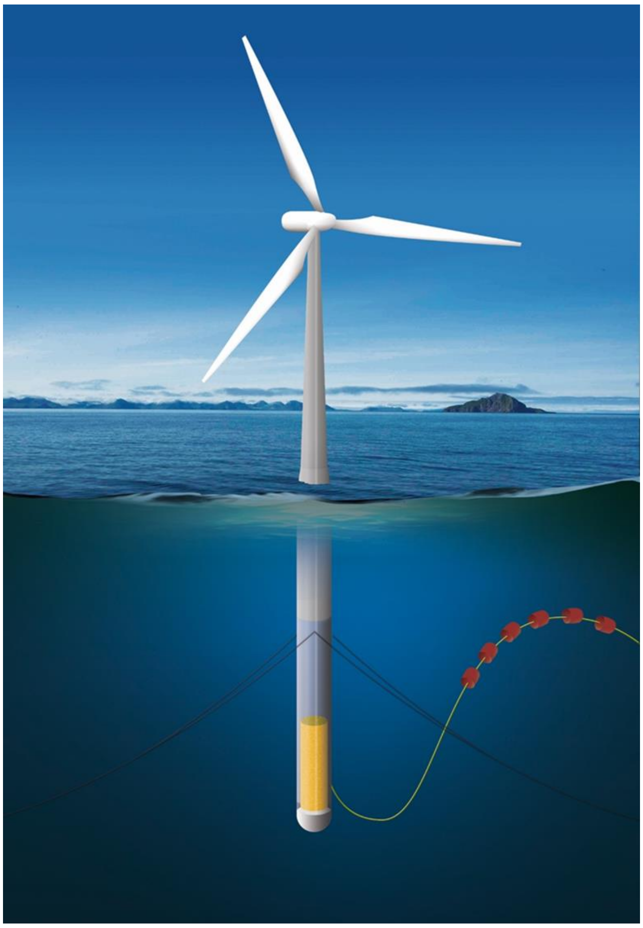

Figure 3.1-1: WindCrete overview

3.1.2 Transport and Installation

The WindCrete is planned to be built in a horizontal position, launched to the sea and towed close to the final location in horizontal position. There or in a place close to it, the erection process, the WTG installation, the ballasting and emergence process are performed. More details about the construction, transport and installation process are presented in the following.

- **Construction:**

Considering the inherent constraints of monolithic structures, such as heavy weight and large dimensions, construction is expected to be done in a dry dock or similar facility, from where it is possible to launch the structure directly to the sea. The structure is designed to be built in a horizontal position by using a slipform. A comparison of different construction process and alternatives for WindCrete were studied and compared in an article, see Ref.[OP4].

- **Launching and Towing:**

Depending on the construction facility, the sea launching can be done in two different ways. One method is by the flotation of the structure in the case of using a dry dock. The second method is by the use of sliding guides or wheel skates to slip the structure into the sea. In both cases, the construction site has to be located a few meters from the coast.

- **Erection:**

The erection process is performed by flooding the structure in a controlled manner. The erection process has to ensure that the maximum bending moment on the tower is not surpassed by ensuring that only a portion of the tower protrudes above MSL when it becomes vertical. The process is schematically shown in Figure 3.1-2. During flooding of the structure a vertical cable to restrain the dynamic motion of the structure during this phase is needed because the transition between horizontal and vertical position is unstable around 5-10º of tilt.

During this process, it is important to keep around 90% of the structure submerged, which offers some advantages. The maximum tower bending moment is reduced and the installation of the wind turbine can be done without heavy floating cranes, as described in detailed in the next section.

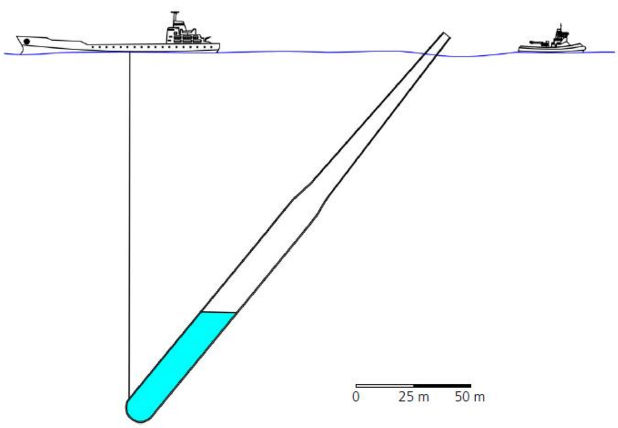

Figure 3.1-2: WindCrete erection process (See Ref.[OP5])

- **WTG installation:**

The WTG installation is performed using a catamaran or an equivalent equipped with a small crane as shown in Figure 3.1-3. The process take advantage of the low height of the tower due to the structure being submerged around 90%. With this procedure, there is no need for the use of heavy floating cranes, reducing the complexity of the offshore tasks and the installation costs.

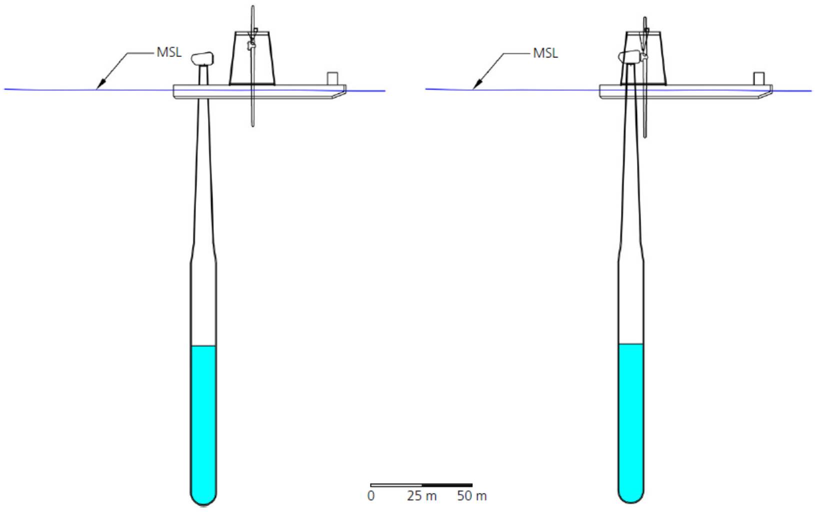

Figure 3.1-3: Wind Turbine installation (See Ref.[OP5])

- **Ballasting and Emergence:**

The structure is taken out by pumping out water (on the left side of Figure 3.1-4), maintaining stability with the water ballast. Then, the aggregates can be introduced inside the floater using a side opening, which after can be the designated maintenance door, using conveyor belts from an outside ship moored to the structure (right side of Figure 3.1-4).

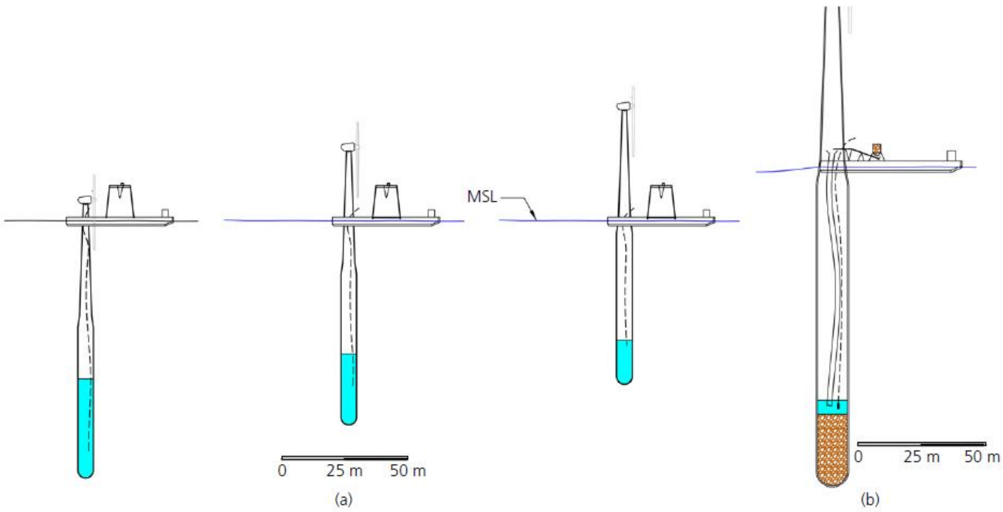

Figure 3.1-4 - Emerging and ballasting WindCrete (See Ref.[OP5])

3.1.3 Operating and maintenance philosophy

The WindCrete structure is designed to ensure a lifespan of over 60 years without intensive maintenance. This means that the structure does not have to be moved onshore for inspection and maintenance, which reduces the cost of these operations. The maintenance of the wind turbine, the dynamic cable and the mooring system, can be performed at the offshore location, and the substitution of the WTG components will be performed like the installation process.

3.2 ACTIVEFLOAT Floater Description

3.2.1 General Description

ACTIVEFLOAT is based on a semisubmersible-type configuration, which means that it has enough waterplane area inertia to face tilting angles with large righting moment. This is reached thanks to three separated columns piercing the water surface, which are the main contribution to the platform stability. A central column supports the WTG tower while three prismatic pontoons link all the system together below the sea level.

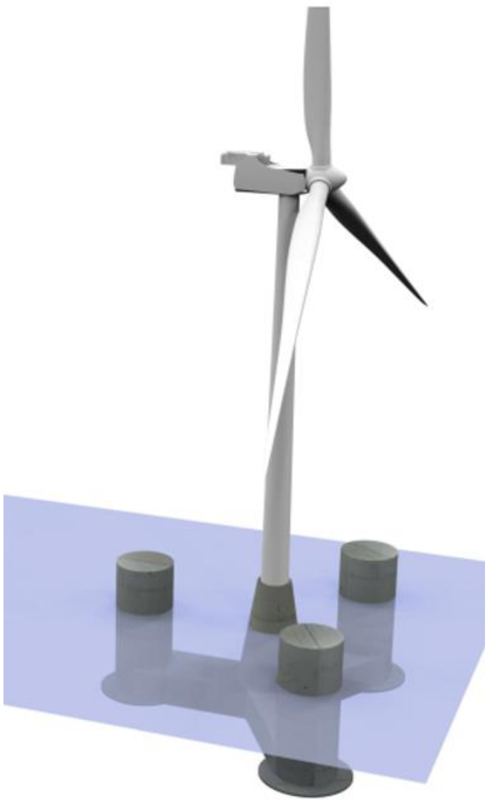

Figure 3.2-1 – ACTIVEFLOAT. General overview

The ACTIVEFLOAT platform is divided in the following main parts:

- **Wind turbine generator (WTG):** It is worth noting that only the nacelle, hub and blades are part of the WTG.
- **Steel tower:** A steel tower is fitted on top of the lower concrete tower piece.
- **Foundation:** ACTIVEFLOAT is a unique body which comprises the following parts:

- **External columns:** Three cylindrical towers positioned in the perimeter of the foundation each 120º. These columns emerge above the sea level which provides the stability to the platform. They shall be also water filled to ballast the platform.
- **Pontoons:** The prismatic beams joining the central column with the external ones. They are beam-like rectangular-cross-section elements which provide resistance to bending moments and act as water ballast tanks.
- **Central column:** Conical tower which acts as the tower foundation. Its height is the same as that of the outer columns and holds the access platform and tower flange on top. It is a dry space to allocate the ballast system, HVAC, etc.

- **Mooring system**: the mooring system consists of anchors, mooring lines, connectors and links. It is composed of several catenary mooring lines (chain, fibers or mixed systems). The final arrangement will be defined later in detail phases.

3.2.2 Transport and Installation

The ACTIVEFLOAT is a semisubmersible, which allows for the transportation of the entire platform from the port and installing it without the assistance of Heavy Lift Vessels. Several installation alternatives are possible depending on project site specific requirements. Therefore, the final installation procedure will be written down on case by case basis taking into account the available installation aids, harbour facilities, etc.

I. Deployment at sea

The deployment at sea is dependent on the facilities available at the fabrication yard such as dry-dock, slipway and craneage. The base case assumes that the foundations are floated out from a dry-dock, launched through a slipway or loaded-out onto a barge. After the launching, the steel tower and WTG are fitted on top.

II. Transport

Transport is carried out as a simple towing operation. Pulling padeyes or bitts are fitted so that a simple tugboat can be engaged to perform the operation. Additional pulling points shall be foreseen for a second tugboat to act as escort.

III. Offshore installation

Once offshore, the foundation shall be ballasted down till its final operation draught. Then, the mooring lines are connected, and the turbine can be commissioned.

A comprehensive high-level list of operations follows:

- Mooring pre-installation
- Tugs and platform positioning to initiate the operation
- Water ballasted until targeted draught
- Mooring connection and pretension of lines
- Cable pull-in
- WTG commissioning

Note that the installation process need to be designed in further detail phases.

3.2.3 Operating and Maintenance Philosophy

The Activefloat platform is a made of concrete in order to have a good durability level of the main bearing structure members. Concrete is a material known for its low degradation rate in hard environmental conditions compared to steel if the quality and design requirements are met in terms of cracking, cover of the reinformcements and concrete mix composition.

Although, concrete structure is not going to drive the maintenance campaigns, other components such as the turbine, the mooring or the dynamic cable required a inspection or corrective plan during the life time of the platform that shall be elaborated during the further phases of the project.

Specific tasks are planned within the COREWIND project that shall adress the O&M phase.

3.3 SITES

Three sites are selected for designing the solutions. The environmental conditions and depths are different so the project conclusions will include sensitivity aspects to this criteria.

Sites info are obtained from previous projects or provided by FIHAC, in order to save time in COREWIND project. A single design depth is selected for each side.

Project locations and information source are indicated below:

- West of Barra Island, Scotland (UK). A depth of 100 meters is the design depth of this location. Information source is the LIFES50+ project.
- Gran Canaria Island (Spain). A depth of 200 meters is the design depth of this location. The information for this site is mainly obtained from the ELICAN project.
- Morro Bay (USA). A depth of 870 meters is the design depth of the site. The information was provided by the FIHAC.

3.3.1 Site A. West of Barra Island (SCOTLAND) Depth 100 m

The selected site A, West of Barra (WoB), is located 19 km West of Barra Island, Scotland, within the 12 nm zone. This site has been identified by a previous project funded by the European Commission under the Horizon 2020 programme, the LIFES50+, as a potential area where test sites for deep water floating technology could be located.

Coordinates	Sexagesimal System		Standard UTM System (m)		
	W	N	E	N	Zone
COREWIND PLATFORM	56°53'09.60"	7°56'52.84"	564100.60	6305189.01	29 V

Table 3.3-1 – West of Bara Island Site. Coordinates

For the characterization of the oceanographic and meteorological conditions of the selected site in West of Barra, the information provided in the public deliverable D1.1 of the above-mentioned project LIFES50+ is used.

A 100 meters depth is assigned to this location which is consistent with the actual depth of the site.
Information source is the LIFES50+ project.

Figure 3.3-1 – West of Barra Location

3.3.2 Site B. Gran Canaria Island (SPAIN). Depth 200 m

The second site selected for the COREWIND project is located off the southeast coast of Gran Canaria (GC) island, in the Canary Islands, Spain.

Design depth of the site is 200 meters.

Coordinates	Sexagesimal System		Standard UTM System (m)		
	W	N	E	N	Zone
COREWIND PLATFORM	15°19'48.00"	27°45'0.00"	467478.89	3069552.70	28 R

Table 3.3-2 – Gran Canaria Site. Coordinates

Figure 3.3-2 – Gran Canaria Site. Location

3.3.3 Site C. Morro Bay (United States of America). Depth 870 m

The third site selected for the COREWIND project is located in the west coast of the United States at California State. The site has moderated extreme conditions and depths that ranges 600 to 900 meters. The selected depth for design is 870 meters.

Coordinates	Sexagesimal System		Standard UTM System (m)		
	W	N	E	N	Zone
COREWIND PLATFORM	121°30'00.00"	35°5'0.00"	671538.71	3901342.14	10 S

Table 3.3-3 – Morro Bay Site Location

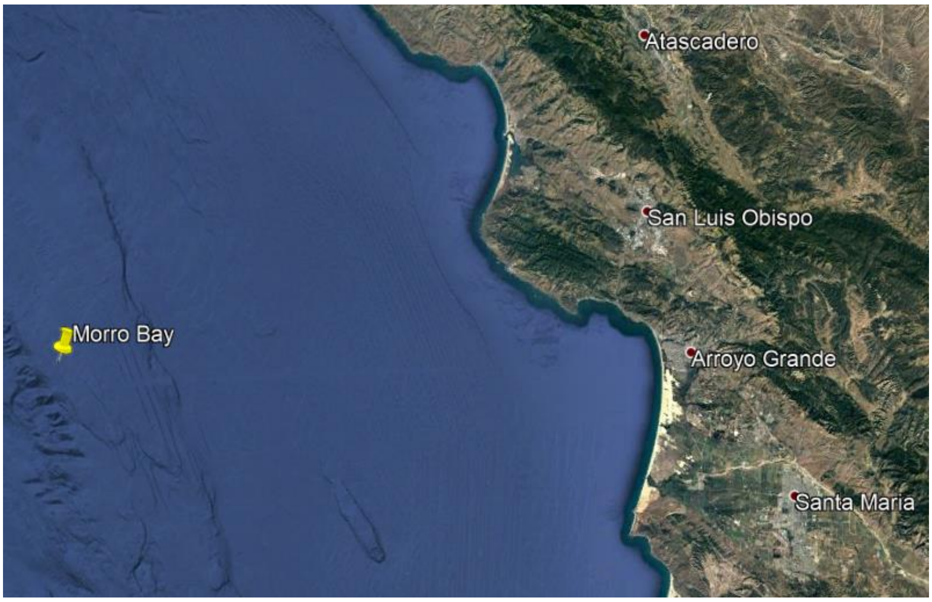

Figure 3.3-3 – Morro Bay Site. Location

4 CODES, STANDARDS AND REFERENCE DOCUMENTS

4.1 CODES AND STANDARDS

Generally for a commercial project tha local rules of each site shall be at the top of the hierarchy. This project will set a fixed set of rules for all sites.

There has been two improvements in the regulatory framework of the floating wind turbines made by DNVGL and the IEC organisms. The first issued in July of 2018 and the second in April of 2019 issued offshore standard specific for the floating wind turbines design.

These two standards share scopes and are established as the main standards to be followed in the COREWIND project with preference to the DNVGL that has additional rules that cover all systems of the platform.

- General leading codes:
  - DNVGL-ST-0119
  - IEC TS 61400-3-2
- Loads and dynamic analyses:

- DNVGL-ST-0437
    - DNVGL-RP-0286
- Structural Design:
    - EN 1992
    - EN 1993
    - Model Code 2010
- Mooring Design
    - DNVGL-OS-E301
- Dynamic cable
    - DNVGL-ST-0119
    - DNV-OS-J103

Following sections depicts the main principles extracted for the standards and the recommended practices (all listed in table below).

REF	Document	Document Title
S1	DNVGL-ST-0119	Design of floating wind turbine structures
S2	IEC TS 61400-3-2	Design requirements for floating offshore wind turbines
S3	DNVGL-ST-0126	Support structures for wind turbines
S4	DNVGL-ST-0437	Loads and site conditions for wind turbines
S5	DNVGL-RP-0286	Coupled analysis of floating wind turbines
S6	DNVGL-RP-C205	Environmental conditions and environmental loading
S7	IEC 61400-1	Wind energy generation systems - Design requirements
S8	IEC 61400-3-1	Design requirements for fixed offshore wind turbines
S9	EN 1992	Eurocode 2: Design of concrete structures
S10	EN 1993	Eurocode 3: Design of steel structures
S11	Model Code 2010	fib Model Code 2010 for Concrete Structures
S12	DNVGL-RP-C203	Fatigue design of offshore steel structures
S13	DNV-OS-C502	Offshore concrete structures
S14	DNV RP H103	Modelling and analysis of marine operations
S15	DNV RP H101	Risk management in marine and subsea operations
S16	DNV OS C301	Stability and watertight integrity
S17	DNV RP H104	Ballast, stability and watertight integrity - Planning and operating Guidance
S18	DNVGL-RP-C202	Buckling strength of shells
S19	DNV RP F205	Global performance analysis of deepwater floating structures
S20	DNVGL OS E301	Positioning mooring
S21	DNVGL OS E302	Offshore mooring chain
S22	DNVGL OS E303	Offshore mooring fibre ropes

S23	DNVGL OS E304	Offshore mooring steel wires ropes
S24	DNVGL OS E332	Offshore fibre ropes

5 LIST OF HOLDS

HOLD No.	Section	Description
1	**8.4.2**	Wind-Wave misalignment data at WoB
2	**9.4.2**	Wind-Wave misalignment data at GC
3	**10.5**	Current data at Morro Bay
4	**17**	Marine growth below 100 m
5	**16**	Wind farm description
6	**23.2.2**	Excursion limit for depths of 200 and 870 meters

6 COORDINATE SYSTEMS AND UNITS

6.1 COORDINATE SYSTEM AND SIGN CONVENTION

Definition of the coordinate system and sign convention might be specific to the different types of floaters. For example, for a barge floater, the surge axis is the longitudinal axis. For a spar, which is axisymmetric, the surge and sway axes of the floater are not predefined by the spar structure itself, but their orientation may be dictated by the station keeping system.

An example is shown in the next figure.

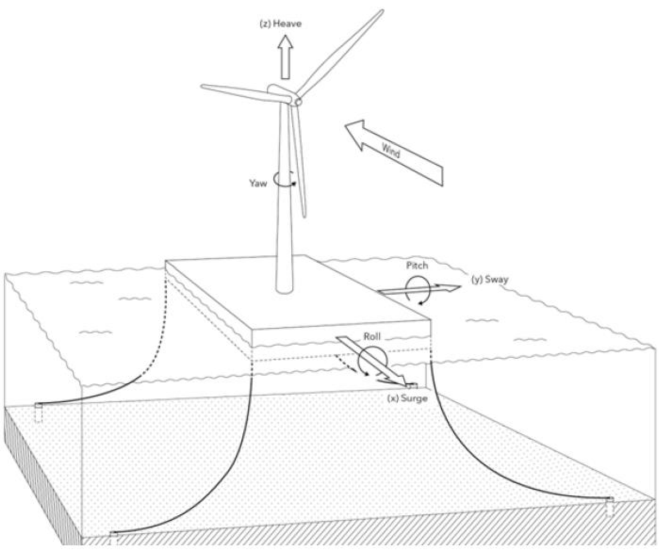

Figure 6.1-1 – Example of coordinate system

6.1.1 WindCrete Coordinate system

Following figure shows the axis system and sign convention. Summarizing:

- Center of coordinates is situated at the vertical axis of the platform at the a certain height to be decided in future phases of the design.
- +Z axis pointing upwards
- +X axis initially pointing between mooring lines 2 and 3.
- +Y axis initially pointing to port side, 270º off mooring line 1.

Motions shall be given as:

- Positive surge when movement goes parallel to X-axis towards its positive direction.
- Positive sway when movement goes parallel to Y-axis, towards its positive direction.
- Positive heave when movement goes upwards.
- Positive roll when the platform rotates around X-axis and the turbine goes towards negative Y-direction.
- Positive pitch when the platform rotates around Y-axis and the turbine goes towards positive X-direction.
- Positive yaw when platform rotates counterclockwise in plan view.

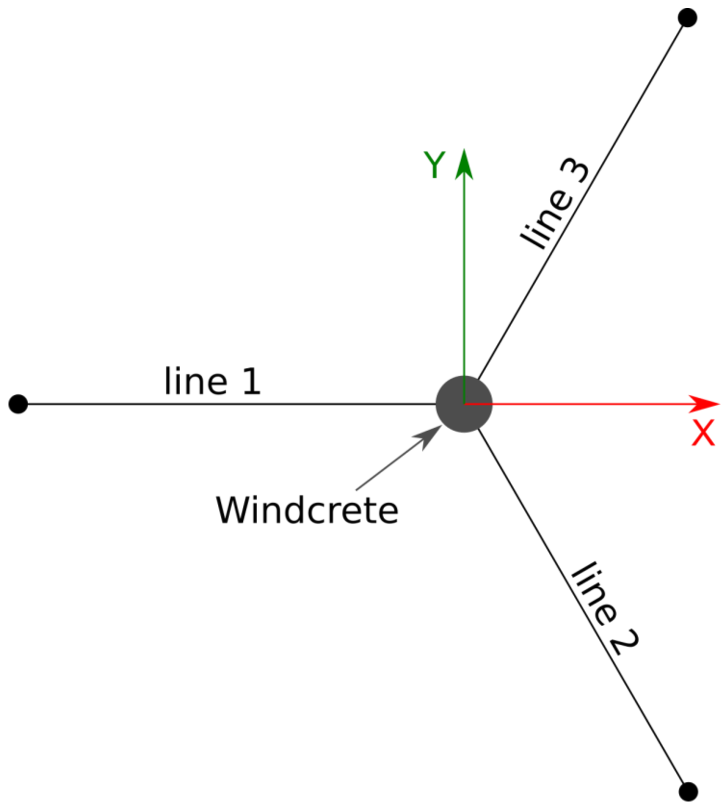

Figure 6.1-2 – WindCrete Coordinate System

6.1.2 ACTIVEFLOAT Coordinate System

Following figure shows the axis system and sign convention. Summarizing:

- Center of coordinates is situated at the geometric center of the lower slab, aligned with the tower vertical axis.
- +Z axis pointing upwards.
- +X axis pointing between two external columns, between mooring lines 1 and 2.
- +Y axis pointing to port side, 30º off mooring line 2 and 90º off mooring line 3.

Motions shall be given as:

- Positive surge when movement goes parallel to X-axis towards its positive direction.
- Positive sway when movement goes parallel to Y-axis, towards its positive direction.
- Positive heave when movement goes upwards.
- Positive roll when the platform rotates around X-axis and the turbine goes towards negative Y-direction.
- Positive pitch when the platform rotates around Y-axis and the turbine goes towards positive X-direction.
- Positive yaw when platform rotates counterclockwise in plan view.

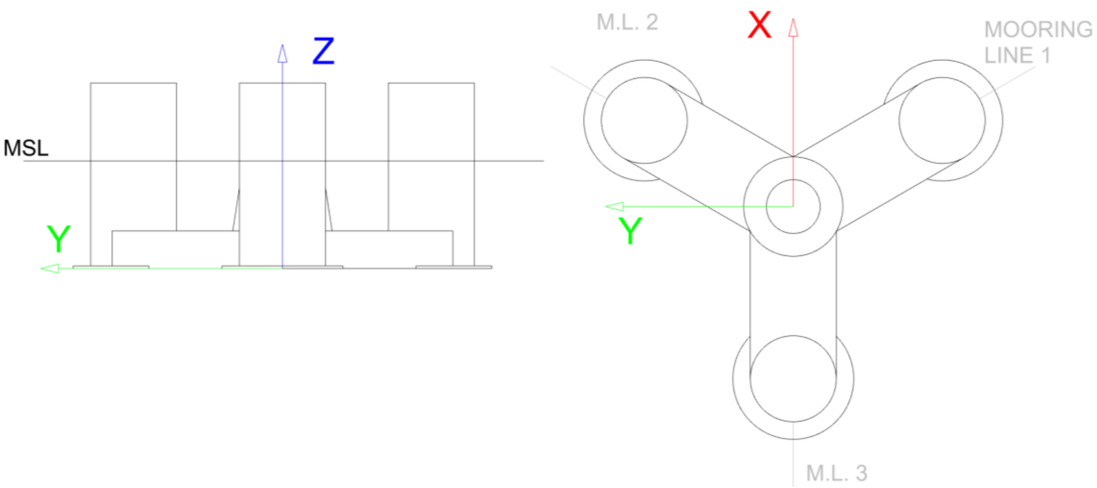

Figure 6.1-3 – ACTIVEFLOAT Coordinate System

6.2 UNITS SYSTEM

The ISO International System of units (SI) shall be used. Angles shall be referred to in the 360 degree system.

7 WIND TURBINE DATA

The wind turbine chosen is a preview of the IEA 15 MW reference turbine, upscaled from the 10 MW turbine from DTU. Its main features are shown in the following table which are extracted from Ref. [D1.1].

WIND TURBINE CHARACTERISTICS	
Output power	15 MW
Rotor diameter	240.00 m
Hub diameter	6.00 m
Hub height above sea level	150.00 m
Nacelle mass including rotor (RNA)	1446.00 t
Blade mass	65.70 t
Distance from Tower Top to Hub Height	5.00 m
Cut-in wind speed	3.00 m/s
Rated wind speed	10.56 m/s
Cut-out wind speed	25.00 m/s
Minimum rotor speed	4.60 rpm
Maximum rotor speed	7.60 rpm

Table 6.2-1 – Wind Turbine main parameters

Note that design of wind turbine in Ref. [D1.1] is based on a land-based WTG, therefore some adjustments may be implemented for some parameters shown in table above. Hub height, for example, would have to be reviewed in order to comply with the air gap requirements.

Following figures show the main performance of the 15 MW wind turbine.

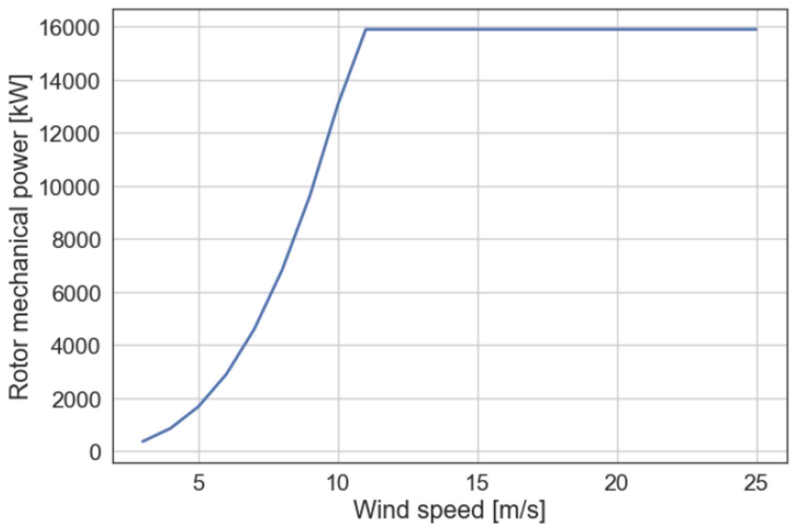

Figure 6.2-1 – 15 MW wind turbine. Power curve

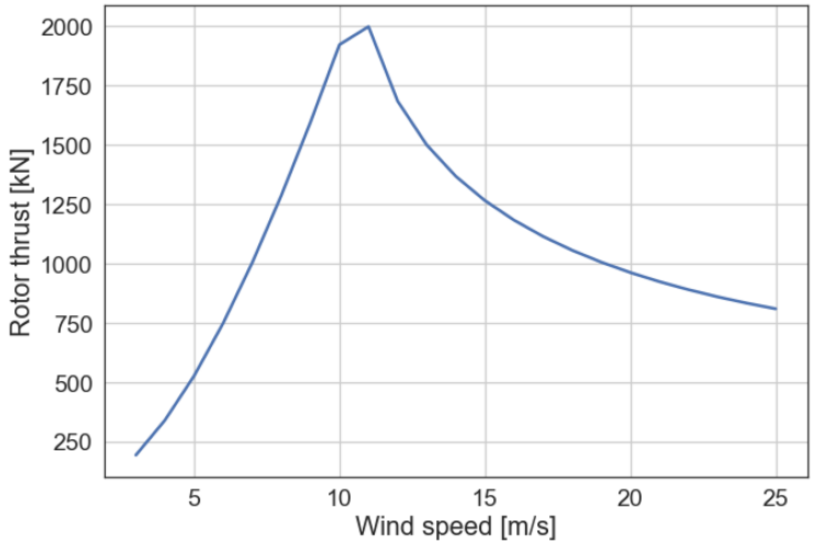

Figure 6.2-2 – 15 MW wind turbine. Thrust curve

8 ENVIRONMENTAL DATA.SITE A. WEST OF BARRA ISLAND

All data provided in this section are extracted from the document D1.1 of LIFES50+.

8.1 WATER DEPTHS AND LEVELS

The design water depth at West of Barra Island is set to 100 m. Summary of West of Barra´s water levels are given below.

WATER LEVELS FOR WEST OF BARRA, SCOTLAND	
Highest Still Water Level (HSWL)	4.16
Highest Astronomical Tide (HAT)	3.16
Mean Sea Level (MSL)	2.32
Lowest Astronomical Tide (LAT)	-1.48
Lowest Still Water Level (LSWL)	-2.48

Table 8.1-1 – WoB Water Levels

Positive and negative storm surges are the 50 year return period values extracted from LIFES50+ project data.

No data about the 1 year return period storm surges are available.

8.2 WIND

As expected, given the location of this site, the wind resource is high and reliable through the year, presenting an annual mean power density of around 1,3 kW/m^2.

The main reference considered when evaluating the wind conditions of West of Barra site is the report issued by European project LIFES50+. This document states that all the data available are 1-hour averaged wind speeds at 10 m above MSL (measurements over 31 years), so all the numbers will be generated by extrapolating to 10-minute averaged and to other heights.

8.2.1 Normal Wind Profile

The best fit for the wind speed profile in normal conditions has been found to be the logarithmic law:

$$V(z) = V_{hub} \cdot \frac{ln(z\,/z_0)}{ln(z_{hub}/z_0)}$$

The resulting 10-minute mean wind speed profile is the following:

Normal Wind Profile	
Height	Speed
[m]	[m/s]
10	9.50
20	10.16
50	10.97
100	11.58
119	11.74
150	11.95

Table 8.2-1 Normal wind speed profile for WoB

Note that assuming a logarithmic fit, as indicated in the LIFES50+. project, the 150 m wind speed has been inferred.

8.2.2 Extreme Wind Profile

LIFES50+ project used standard extreme reference wind speed of 50 m/s for the 50 years return period wind speed. (The LIFES50+ project used standard value of Vref of 50 m/s although the calculated 50-yr return period at 119 meters above sea level was 53.79 m/s).

The wind profile used is the recommended in DNVGL in Ref. [S5] and IEC in Ref. [S6] with a power law with a 0.12 exponent (α) as recommended in Ref. [S6]:

$$V(z) = V_{hub} \cdot (z/z_{hub})^{0.12}$$

The extreme wind speed profile for a return period of 50 years and 10 minutes mean, would be the following:

Extreme Wind Profile (50-yr)	
Height	Speed
[m]	[m/s]
10	36.13
20	39.26
50	43.82
100	47.63
119	48.63
150	50.00

Table 8.2-2 - Extreme conditions wind speed profile for WoB (Tr = 50 years)

Note that a revision of this profile may be required when hub height is established for the different floaters designers.

Following the same procedure as for the 50 years return period profile, the 1 year return period profile is provided.

Extreme Wind Profile (1-yr)	
Height	Speed
[m]	[m/s]
10	29.77
20	32.35
50	36.11
100	39.24
119	40.07
150	41.20

Table 8.2-3 - Extreme conditions wind speed profile for WoB (Tr = 1 years)

8.2.3 Wind Speed Histogram

Following table summarizes the exceedance probability for the 1-hour averaged wind speed.

Wind Speed [m/s]	$0,0 < u_{10} < 0,3$	100,00 %
	$0,3 < u_{10} < 1,6$	100,00 %
	$1,6 < u_{10} < 3,4$	99,97 %
	$3,4 < u_{10} < 5,5$	95,82 %
	$5,5 < u_{10} < 8,0$	82,67 %
	$8,0 < u_{10} < 10,8$	60,08 %
	$10,8 < u_{10} < 13,9$	35,00 %
	$13,9 < u_{10} < 17,2$	14,79 %
	$17,2 < u_{10} < 20,8$	4,20 %
	$20,8 < u_{10} < 24,5$	0,73 %
	$24,5 < u_{10} < 28,5$	0,11 %
	$28,5 < u_{10} < 32,7$	0,00 %

Table 8.2-4 Wind speed exceedance probability for WoB

8.2.4 Wind Speed Rose

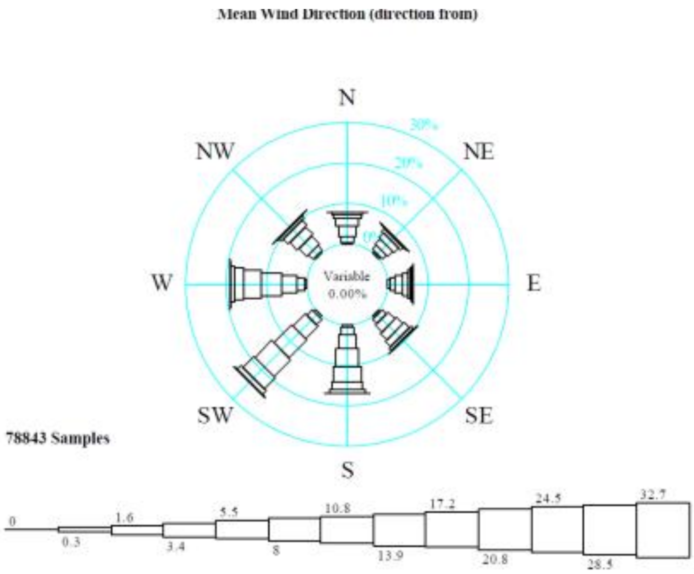

Figure 8.2-1. Wind rose (Mean wind speed at 19,5 m ASL) for WoB

The following table gathers up the mean wind speed for the different incoming wind direction sectors. The direction, clockwise from true North, is from which the wind is blowing. Direction measures were performed for 1-hour average direction at a height of 19,5 m (despite the mean wind speed, that is given at 10 m height).

Mean Wind Speed at 10 m [m/s]	Mean Wind Direction [º] [19]							
	0	**45**	**90**	**135**	**180**	**225**	**270**	**315**
0,00-0,30								
0,30-1,60	2	4	6	2	3	4	2	
1,60-3,40	333	413	403	430	535	469	366	326
3,40-5,50	1091	1138	1116	1229	1515	1527	1576	1170
5,50-8,00	1932	1385	1395	1782	2668	3385	3049	2217
8,00-10,80	1421	1294	1105	1841	3496	4850	3750	2016
10,80-13,90	928	641	510	1408	2847	4729	3451	1420
13,90-17,20	397	215	192	605	1576	3035	1782	549
17,20-20,80	52	55	68	162	561	948	731	160
20,80-24,50	5	5	1	30	86	182	132	46
24,50-28,50					3	27	46	10
28,50-32,70							1	1
32,70-51,50								

Table 8.2-5 Wind direction distribution for WoB

8.2.5 Turbulence Intensity

There is no specific data for the site turbulence, so it is assigned a Class C, as described in IEC-61400-1.

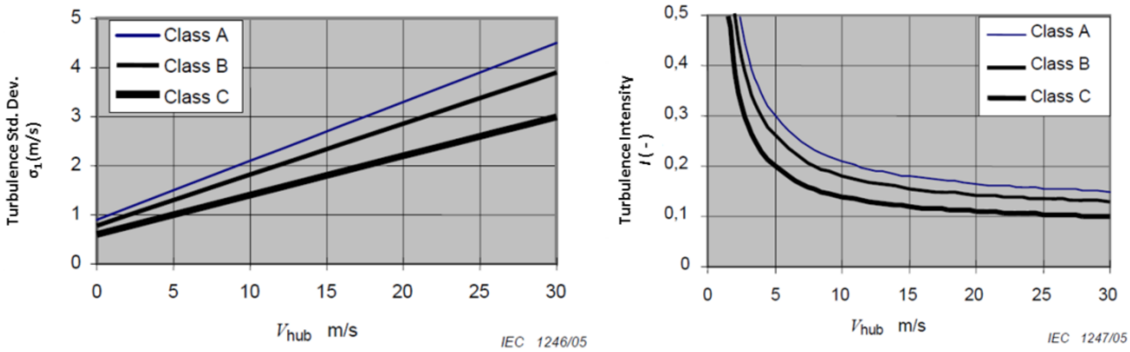

Figure 8.2-2. Turbulence Intensity for different Wind Turbine Classes, as defined in IEC-61400-1

Wind speed [m/s]	NTM [%]	ETM [%]
2	0.426	0.980
3	0.314	0.678
4	0.258	0.527
5	0.224	0.436
6	0.202	0.376
7	0.186	0.332
8	0.174	0.300
9	0.165	0.275
10	0.157	0.255
11	0.151	0.238
12	0.146	0.224
13	0.142	0.213
14	0.138	0.203
15	0.135	0.194
16	0.132	0.187
17	0.130	0.180
18	0.127	0.174
19	0.125	0.169
20	0.124	0.164
21	0.122	0.160
22	0.121	0.156
23	0.119	0.152
24	0.118	0.149
25	0.117	0.146
26	0.116	0.143
27	0.115	0.141
28	0.114	0.138
29	0.113	0.136
30	0.112	0.134
31	0.112	0.132
32	0.111	0.130

Table 8.2-6 – Turbulence intensity for NTM and ETM for Class C

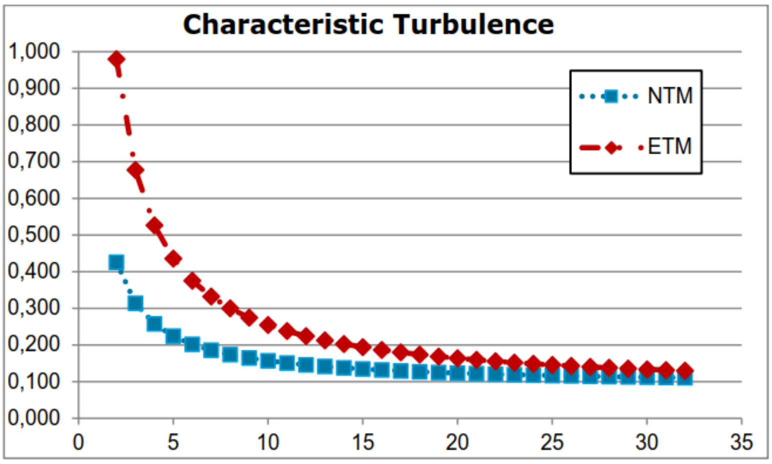

Figure 8.2-3 – Normal and extreme turbulence

8.2.6 Wind Spectrum

In absence of more detailed information and following DNVGL recommendations, it has been decided to assume the Kaimal model as the most representative of wind spectral density at West of Barra. The Kaimal model provides the distribution of wind energy over the different frequencies.

For this project, same philosophy as in LIFES50+. is used for NTM and ETM that will use values in Table 8.2-6.

8.2.7 Wind Gust Characteristics

No information is available at West of Barra site in regard to wind gust. Hence, reference is made to IEC-61400-1 Ref. [S7], where it can be found mathematical models that allow characterizing wind gust and accounting for its effects on the design load cases (DLC´s).

Section 3.2.2.9 of DNVGL-ST-0119 shall be taken into account when defining the DLCs that involve gusts. The gust events presently specified are based on a duration of 10.5 seconds that may be not sufficient for dynamic characteristics of the floating offshore wind turbines.

The duration of the events shall be selected accounting for the natural periods of the platform, without disregarding the 10.5 seconds currently specified in the standards.

8.3 WAVES

8.3.1 Extreme Waves

Based on Weibull distribution and assuming 3-hour storms sea states, significant wave heights associated to 50, 20, 10 and 1 year return period are provided in the following table. For each of these values, the wave peak period has been extrapolated as the most probable value associated to that height. In order to do so a curve fitting analysis (see below) has been performed to allow for determining the most probable values to be associated to those wave heights that are not contained within the available data.

Return period (years)	Hs (m)	Tp (s)
50	15.6	12.0-18.0
20	14.7	12.0-18.0

10	14.0	12.0-18.0
1	11.5	12.0-18.0

Table 8.3-1 - Extreme Wave data for WoB

Within the LIFES50+. project, the wave peak period was extrapolated as the most probable value associated to each wave height. In order to do so, a curve fitting analysis (see below) was performed to allow for determining the most probable values to be associated to each wave heights that are not contained within the available data.

A sensitivity analysis might be required for identifying the critical sea states for each floater.

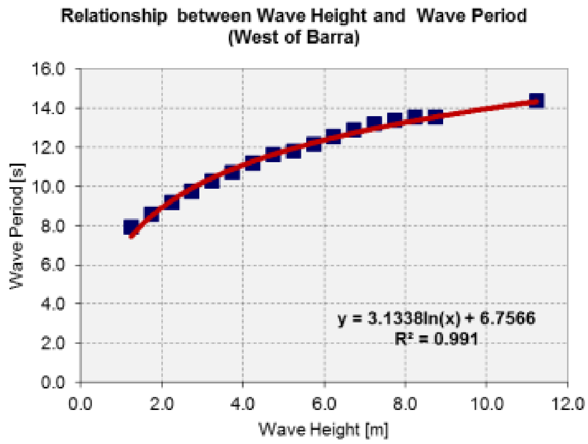

Figure 8.3-1 - Extrapolation curve for Peak period-Significant wave height correlation

A sensitivity analysis might be required for identifying the critical sea states for each floater.

8.3.2 Waves Scatter Diagram

The following table shows the frequency distributions of significant wave height and spectral peak period.

Hs	Tp [s]																					
[m]	0-1	1-2	2-3r	3-4r	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16	16-17	17-18	18-19	19-20	20-21	21-22
0.0-0,5			1																			
0.5-1,0				129	337	681	581	1242	774	341	88	24	11	40	28	11						
1.0-1,5				18	589	1721	1189	2403	3333	1824	754	284	120	23	20	6						
1.5-2,0					21	1260	1855	1644	2765	2720	1444	744	235	131	50	27	3	2				
2.0-2,5				1	4	164	1804	1614	1843	2055	1773	1273	562	222	40	31		4	1			
2.5-3,0					1	8	607	1536	1290	1462	1659	1184	686	338	101	40	1	8	3			
3.0-3,5							85	989	970	1014	1170	1140	749	265	167	61	11	9	1			
3.5-4,0							10	397	846	859	971	873	754	319	221	76	20	5				
4.0-4,5							1	53	646	706	744	893	791	353	206	127	30	4				
4.5-5,0								8	221	529	586	790	659	414	167	76	44	27	4			
5.0-5,5									44	340	558	517	441	250	252	56	9	10	4			
5.5-6,0									7	169	293	433	424	214	182	75	9	16				
6.0-6,5									1	67	101	315	263	186	100	54	21	13	6			
6.5-7,0										3	42	220	301	218	101	35	17	13	2			
7.0-7,5											15	106	160	156	69	54	17	1				
7.5-8,0											8	32	145	117	59	50	1	4				
8.0-8,5												10	121	112	67	37		3				
8,5-9,0												3	115	148	62	25	4	2				
9,0-13,5													78	277	321	197	15	21				

Table 8.3-2 - Significant wave height – Peak period frequency for WoB

8.3.3 Wave Rose

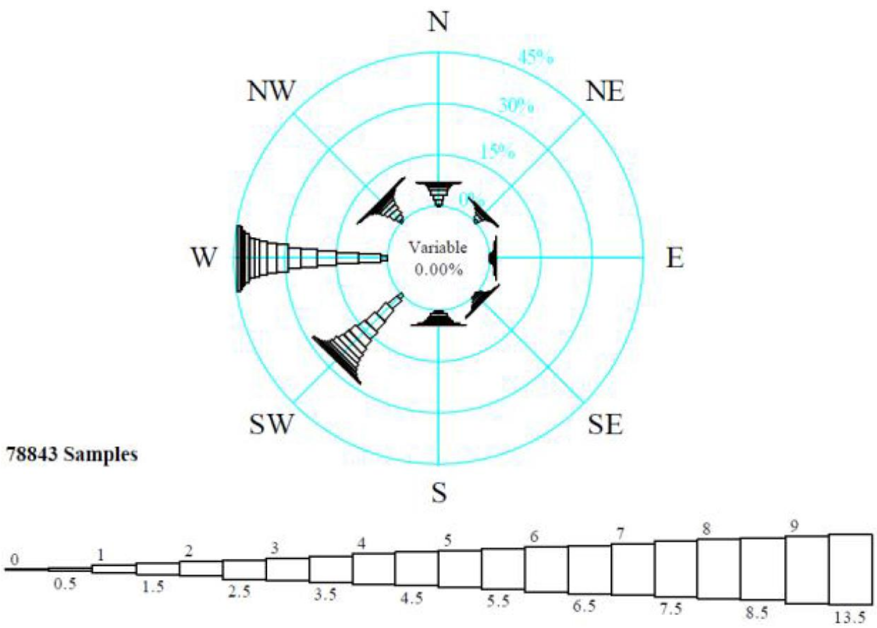

Figure 8.3-2. West of Barra wave rose (Significant wave height)

The following table gathers up dominant wave direction for the different incoming wave direction sectors. The direction, clockwise from true North, is from which the waves are travelling.

Significant Wave Height [m]	Dominant Wave Direction [º] [21]							
	0	45	90	135	180	225	270	315
0,00-0,50				1				
0,50-1,00	410	568	151	44	59	1100	1435	520
1,00-1,50	1159	950	330	326	311	2933	5156	1119
1,50-2,00	1344	597	324	376	490	3279	5111	1380
2,00-2,50	1029	356	190	541	684	2951	4507	1133
2,50-3,00	624	217	89	403	499	2375	3755	962
3,00-3,50	343	175	63	227	371	1972	2870	610
3,50-4,00	234	107	65	170	294	1587	2544	350
4,00-4,50	151	44	58	117	301	1343	2292	248
4,50-5,00	104	14	14	81	160	1221	1705	226
5,00-5,50	73	12	13	28	136	870	1191	158
5,50-6,00	56	2	8	26	84	542	1030	74
6,00-6,50	9		11	24	35	339	658	51
6,50-7,00				1	15	316	582	38
7,00-7,50					9	192	348	29
7,50-8,00					9	114	268	25
8,00-8,50						100	233	17
8,50-9,00						105	237	17
9,00-13,50						190	664	55
13,50-20,00								

Table 8.3-3 Wave direction for WoB

8.3.4 Waves Spectrum

A Jonswap wave spectrum is usually sufficient for the representation of the power spectral density of wind generated waves (as is the case of West of Barra). However, for floating offshore structures that may be usually affected by swells of 20-25 seconds period, a two-peak power spectrum model shall be used, based on the recommendations given in DNV standards Ref.[S6].

For this project, same philosophy as in LIFES50+ is used for NSS that will use values in Table 8.3-2.

For this project, same philosophy as in LIFES50+ is used for SSS and ESS that will use values in Table 8.3-1.

8.4 WIND – WAVES COMBINED CONDITIONS

8.4.1 Wind-Wave Scatter Diagram

Significant Wave Height [m]	Mean Wind Speed at 10 m [m/s]												
	0,00-0,30	0,30-1,60	1,60-3,40	3,40-5,50	5,50-8,00	8,00-10,80	10,80-13,90	13,90-17,20	17,20-20,80	20,80-24,50	24,50-28,50	28,50-32,70	32,70-51,50
0,00-0,50						1							
0,50-1,00		5	1054	2316	897	14	1						
1,00-1,50		14	1061	4055	5701	1444	9						
1,50-2,00		1	632	2070	5126	4736	335	1					
2,00-2,50		3	284	1083	3024	5167	1809	21					
2,50-3,00			139	468	1570	3645	2933	169					
3,00-3,50			58	197	762	2080	2981	550	3				
3,50-4,00			40	119	398	1190	2586	997	21				
4,00-4,50			4	33	193	747	2157	1324	96				
4,50-5,00			2	10	81	409	1441	1418	164				
5,00-5,50			1	10	32	184	767	1180	301	6			
5,50-600				1	22	87	452	869	370	21			
6,00-6,50					4	39	207	532	320	25			
6,50-7,00					3	12	116	463	334	24			
7,00-7,50						12	64	276	194	31	1		
7,50-8,00						2	38	195	137	44			
8,00-8,50						2	22	152	138	33	3		
8,50-9,00						2	10	98	201	45	3		
9,00-13,50							6	106	458	258	79	2	
13,50-20,00													

Table 8.4-1 – Wind – Wave Scatter Diagram

Based on this information, it has been performed some studies to try to preview the most probable wind speed associated to each significant wave height. To ensure the best correlation possible with the real sea state conditions (represented by the achieved raw data).

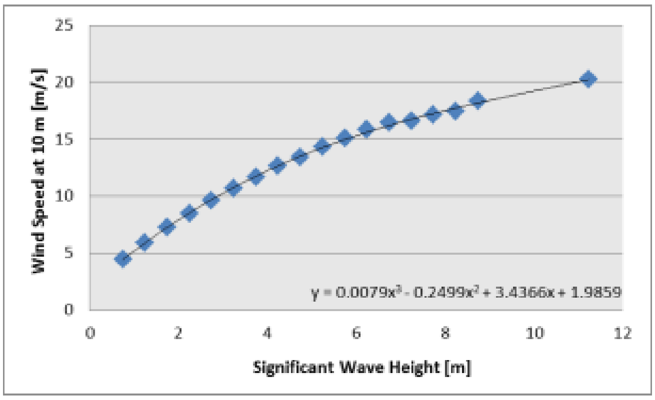

Table 8.4-2 Third order polynomial equation for WoB

In addition to the data in Table 8.4-1 and Table 8.4-2 some extreme assumptions are recommended in order to cover cases with high waves and low wind speed and viceversa that are reflected in the scatter above.

8.4.2 Wind-Wave Misalignment

No metocean data is available about the correlation of wind direction and wave direction. Design assumptions have to be made based on wind and wave roses provided. **[HOLD]**

8.5 CURRENT CONDITIONS

Surrounding Scotland seas are directly affected by oceanic circulation due to its position at the UK continental shelf. The steep bathymetry of the continental slope acts as a barrier between oceanic regions and the shelf sea systems, reducing the amount of water that can travel from the deeper waters of the North Atlantic into the shallower waters on the continental shelf. Tidal currents are stronger than the non-tidal in most of Scottish areas and these are better predictable. Moreover, tidal currents are intensified in localised areas usually where the flow is constrained by topography. This includes areas such as between Orkney and Shetland, the Pentland Firth, off the Mull of Kintyre and Hebrides where tidal streams can be as high as 3.5-4.5 m/s.

The non-tidal circulation on the shelf west of Scotland, (the Scottish Coastal Current) is mainly northwards. However, this circulation is strongly affected by winds and density-driven coastal currents and jets, which can lead to large changes in currents and even a reversal of this general pattern for short periods.

Besides this general overview, no site-specific current data are available at West of Barra. Hence currents at site location have been characterized based on available met-ocean numerical model data] and making certain assumptions in regards to wind generated currents following main recognized standards.

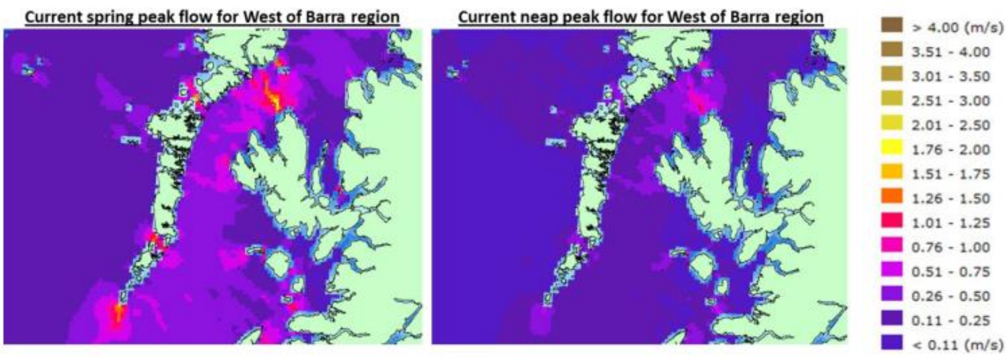

Figure 8.5-1. Current peak flow for the West of Barra region: Current spring peak (left), Current neap peak (right)

8.5.1 Current induced by wind

Current induced by wind has been extracted from the LIFES50+ project and indicated in the following table.

Return Period	Wind induced current speed (at surface) [m/s]
1	0.88
50	1.15

Table 8.5-1 – Current induced by wind speed at sea surface

8.5.2 Deep Water Current

Deep water current has been extracted from the LIFES50+ project and indicated in the following table.

Return period	Tidal current		Storm surge current		Combined current	
	Vc [m/s]	Dir [º]	Vc [m/s]	Dir [º]	Vc [m/s]	Dir [º]
1	0,39	50	0,53	0	0,84	21
50	0,44	50	0,6	0	0,94	21
(*) 0º direction is relative to North.						

Table 8.5-2 Deep water current speed at sea surface

8.5.3 Current Speed profile

Since no information is available at West of Barra regarding the current speed profile, reference is made to DNVGL-RP-C205 section 4. Based on this standard the two following mathematical models have been used to estimate the variation of current speed with depth depending on the type of current under consideration:

Current induced by wind

$$v_{c,wind}(z) = v_{c,wind}(0) \cdot \left(\frac{d_0 + z}{d_0}\right) \; for - d_0 \leq z \leq 0$$

Where $d0$ is taken as half of the water depth at West of Barra following DNVGL recommendations, hence $d0 = 50\ m$.

Tidal current

$$v_{c,tide}(z) = v_{c,tide}(0) \cdot \left(\frac{d + z}{d}\right)^{\alpha} \; for\ z \leq 0$$

Resulting current speed profiles for each of the currents defined in previous sections are given in the following tables for the 1-year and 50-year return period currents respectively. Last column of this table represents the vectorial summation of the aforementioned components.

Depth	WIND COMPONENT	TIDAL & SURGE COMPONENT	TOTAL CURRENT SPEED PROFILE
[m]	[m/s]	[m/s]	[m/s]
0	0.881	1.023	1.570
-10	0.705	1.008	1.421
-20	0.528	0.992	1.279
-30	0.352	0.973	1.147
-40	0.176	0.952	1.029
-50	0.000	0.928	0.928
-60	0.000	0.900	0.900
-70	0.000	0.864	0.864
-80	0.000	0.817	0.817
-90	0.000	0.741	0.741
-100	0.000	0.000	0.000

Table 8.5-3 - Total current speed profile associated to the 1-year return period probability

Depth	WIND COMPONENT	TIDAL & SURGE COMPONENT	TOTAL CURRENT SPEED PROFILE
[m]	[m/s]	[m/s]	[m/s]
0	1.053	1.158	1.822
-10	0.842	1.141	1.642
-20	0.632	1.122	1.471
-30	0.421	1.101	1.312
-40	0.211	1.078	1.169
-50	0.000	1.051	1.051
-60	0.000	1.018	1.018
-70	0.000	0.978	0.978
-80	0.000	0.924	0.924
-90	0.000	0.839	0.839
-100	0.000	0.000	0.000

Table 8.5-4 - Total current speed profile associated to the 50-year return period probability

8.5.4 Current Direction

In absence of more detailed statistical information regarding current direction, only most probable current speed directions can be provided. Based on tidal current direction provided in previous section and assuming that wind induced current direction will be driven by wind´s direction, the following table provides most probable headings with respect to the North.

	Most probable heading	
	Direction [º]	Compass Coordinates
Wind induced current	90	E
Tidal & Surge current	21	NNE
(*) 0º direction is relative to North.		

Table 8.5-5 - Most probable current direction

8.6 ICE LOADS

No specific information is available on site. Figure 8.6-1 shows limit areas in the North-West Europ region for sea ice and collision with icebergs events with and associated annual probability of 10^{-2} and 10^{-4}.

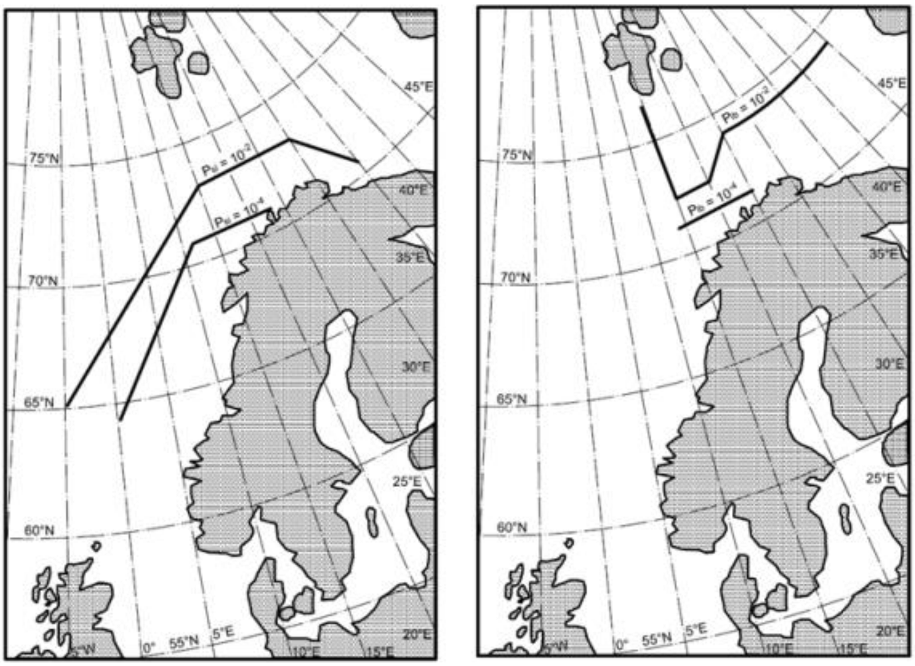

Figure 8.6-1. Annual probabilities of exceedance for sea ice (left) and collision with icebergs (right). ISO 19901-1:2005

Based on the aforementioned information, sea ice and iceberg collision need not to be considered in the design of offshore structures in the UK waters, since there is no evidence to suggest that these events may occur.

Snow accumulation is more likely to occur than ice at West of Barra. Snow may settle on non-horizontal windward-facing parts of an installation if the snow is sufficiently wet.

On vertical surfaces it is only likely to stay in position as snow for a few hours although it may then freeze, hence remaining as ice. Snow accumulation will affect all exposed elements above the splash zone.

Ice may form on an offshore structure through the following mechanisms: (i) freezing sea spray, (ii) freezing fog and super-cooled cloud droplets, (iii) freezing rain and (iv) freezing old wet snow. On a 50-year return period criterion there is no reason to believe that any of the aforementioned mechanisms to form ice on offshore structures is of any significance at the West of Barra site.

The following table provides indicative values for snow and ice accumulation at 57,7 º N.

Structural element	Wet snow		Ice from freezing sea spray		Ice from frozen snow	
	Thickness (mm)	Density (kg/m^3)	Thickness (mm)	Density (KG/m^3)	Thickness (mm)	Density (kg/m^3)
At latitude 57.7°N[#] Tubular member below deck level [*]	-	-	25	850	-	-
Tubular member below deck level [+]	40	500	-	-	30	900
Lattice member above deck level	40	500	-	-	25	900
Horizontal surface	200	100	-	-	-	-

The values in the table have been predicted from a model covering North Sea waters west of 3° E. There is no available data for other UK designated waters but it is suggested that the values in the table may also be used for comparable latitudes west of the UK mainland. The thickness relates to increase in radius in relation to tubular members.

[*] *Icing on members below deck level from freezing sea spray is likely to start about 4-7m above MSL at the thickness indicated and reduce to zero thickness at a height of about 9-15m above MSL*

[+] *Snow and ice from freezing of old wet snow will accumulate on members below deck level only above the splash/spray zone.*

[#] *Because of the absence of data no estimates can be made of the depth of accumulations north of 57.5°N. However, the values for 57.5°N are sufficiently conservative to be used for UK designated waters north of this latitude.*

Table 8.6-1 Extreme snow and ice accumulations. Source OTH 2001/010 for WoB

8.7 OTHER CONDITIONS

8.7.1 Water Temperature

Sea temperatures around Scotland are affected by local climatic conditions (heat flux with atmosphere) and the heat transferred to the shores of Scotland by ocean currents (advective effects). Sea surface temperatures vary with an annual cycle, lagging behind the cycle of atmospheric temperature by around one month.

The coldest sea water temperatures are recorded in the Scottish continental shelf ranging from 6ºC in winter to 14ºC in summer. Since no on-site data are available, sea-surface temperature data have been obtained from the nearest possible location: The Isle of Lewis, located around 120 km North East from West of Barra.

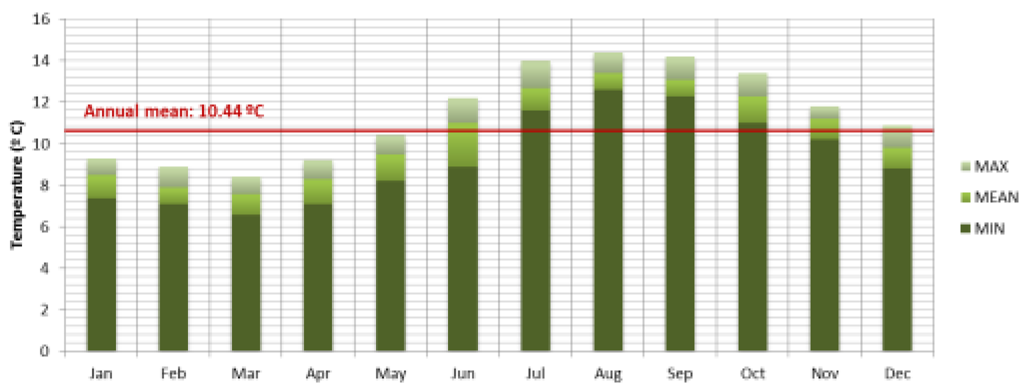

Table 8.7-1 Isle of Lewis average monthly seawater temperature.

8.7.2 Air Temperature

Table below summarizes indicative values for the probable extreme maximum/minimum air temperatures at West of Barra location as well as the lowest observed daily mean air temperature (LODMAT). The values provided in the table below may vary in +/- 1º C.

Air temperature at West of Barra		
Probable extreme max air temperature	22	[º C]
Probable extreme min air temperature	-4	[º C]
LODMAT	-4	[º C]

Table 8.7-2 Air temperature in West of Barra at sea level

8.7.3 Air Density

Air density is 1.225 kg/m^3.

9 ENVIRONMENTAL DATA.SITE B. GRAN CANARIA ISLAND

9.1 WATER DEPTHS AND LEVELS

The design water depth at Gran Canaria Island is set to 200 m. Summary of Southeast Gran Canaria water levels is given below.

WATER LEVELS FOR GRAN CANARIA	
Highest Still Water Level (HSWL)	3.19
Highest Astronomical Tide (HAT)	3.11
Mean Sea Level (MSL)	1.58
Lowest Astronomical Tide (LAT)	0.00
Lowest Still Water Level (LSWL)	-0.13

Table 9.1-1 Water levels for GC

These values have been taken from the tide gauge in the Arinaga port.

9.2 WIND

The wind data for this site has been extracted from the data provided by the SIMAR point 4038006, from the Spanish Ports Authority. This is a grid of points at which models are run to generate wave simulations and data. This point has the following coordinates:

15°19’48.00” W 27°45’0.00” N

These simulations provide the 1-hour wind speed at 10 m above the sea level.

9.2.1 Normal wind profile

The best fit for the wind speed profile in normal conditions has been found to be the logarithmic law:

$$V(z) = V_{hub} \cdot \frac{ln(z/z_0)}{ln(z_{hub}/z_0)}$$

The 1-hour mean wind speed is set at **9.0 m/s**, deducted form wind speed series of the last 10 years. A 1-hour mean wind speed can be extrapolated to a 10-minute mean wind speed following section 2.3.2.11 in Ref. [S5], providing a value of 9.83 m/s.

The resulting 10-minute mean wind speed profile is the following:

Normal Wind Profile	
Height	Speed
[m]	[m/s]
10	9.83
20	10.48
50	11.33
100	11.98
119	12.14
150	12.36

Table 9.2-1 Normal wind speed profile for GC

9.2.2 Extreme wind profile

The maximum 1-hour average wind speed recorded is 19.0 m/s at 10 meters, which is translated to 20.75 m/s of maximum 10-minute average wind speed. Same profile as in WoB is used following recommendations in references [S5] and [S6].

$$V(z) = V_{hub} \cdot (z/z_{hub})^{0.12}$$

The extreme wind speed profile for a return period of 50 years and 10 minutes mean, would be the following:

Extreme Wind Profile Tr = 50 years	
Height	Speed
[m]	[m/s]
10	29.77
20	32.35
50	36.11
100	39.24
119	40.07
150	41.20

Table 9.2-2 - Extreme conditions wind speed profile for GC (Tr = 50 years)

Note that a revision of this profile may be required when hub height is established for the different floaters designers.

Following the same procedure as for the 50 years return period profile, the 1 year return period profile is provided.

Extreme Wind Profile Tr = 1 year	
Height	**Speed**
[m]	**[m/s]**
10	16.00
20	17.39
50	19.41
100	21.09
119	21.54
150	22.14

Figure 9.2-1 - Extreme conditions wind speed profile for GC (Tr = 1 year)

9.2.3 Wind Speed Histogram

Following table summarizes the exceedance probability for the 1-hour averaged wind speed.

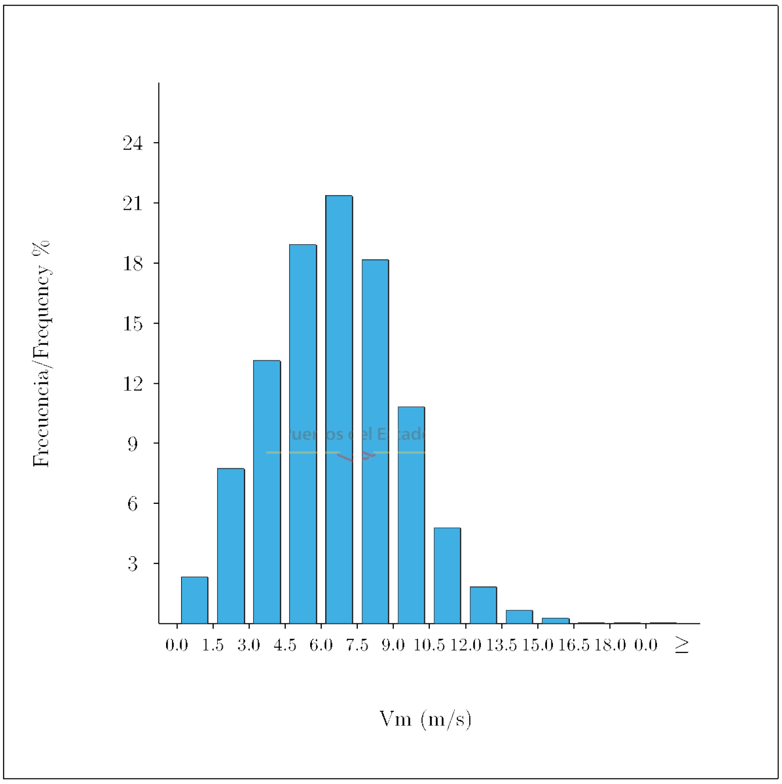

Figure 9.2-2 – Exceedance probability 1-hour averaged wind speed

Wind speed (m/s)		Frequency (%)
0.0	1.5	2.30%
1.5	3.0	7.70%
3.0	4.5	13.15%
4.5	6.0	18.90%
6.0	7.5	21.35%
7.5	9.0	18.15%
9.0	10.5	10.80%
10.5	12.0	4.80%
12.0	13.5	1.80%
13.5	15.0	0.70%
15.0	16.5	0.25%
16.5	18.0	0.05%
18.0	19.5	0.05%

Table 9.2-3 Wind speed exceedance probability for GC

9.2.4 Wind Speed Rose

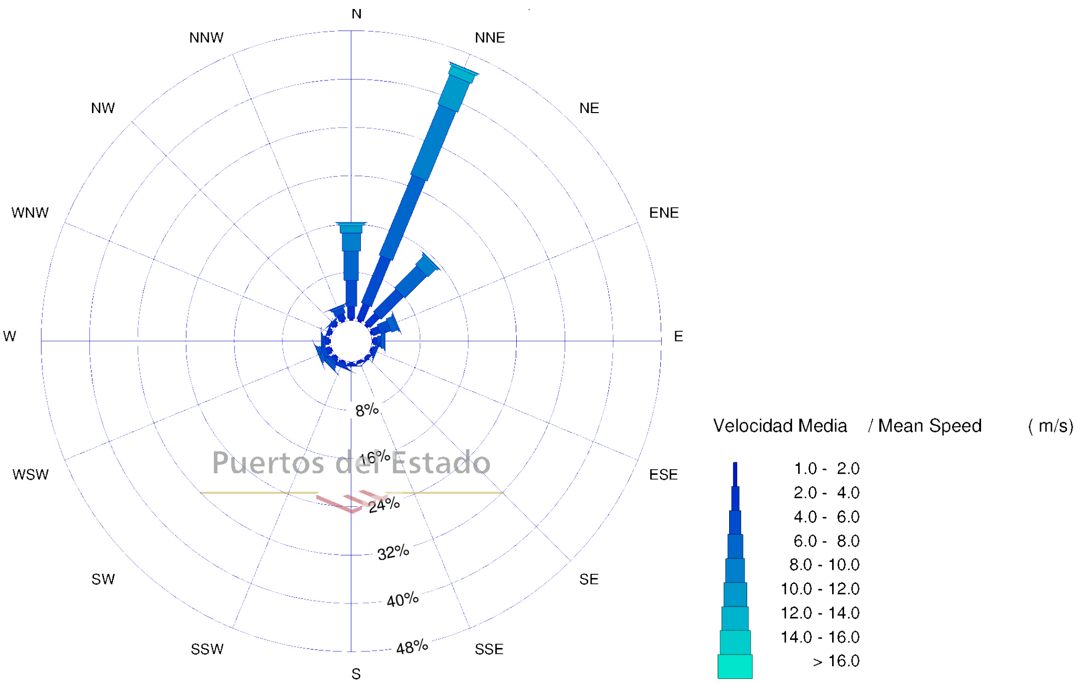

Figure 9.2-3. Wind rose for 1-hour mean speed at GC

9.2.5 Turbulence intensity

There is no specific data for the site turbulence, so it is assigned a Class C, as described in IEC-61400-1.

9.2.6 Wind Spectrum

In absence of more detailed information and following DNVGL recommendations, it has been decided to assume the Kaimal model as the most representative of wind spectral density at Gran Canaria. The Kaimal model provides de distribution of wind energy over the different frequencies.

For this project, same philosophy as in LIFES50+. is used for NTM and ETM that will use values in Table 8.2-6.

9.2.7 Wind Gust Characteristics

No information is available at Gran Canaria site in regard to wind gust. Hence, reference is made to IEC-61400-1, where it can be found mathematical models that allow characterizing wind gust and accounting for its effects on the design load cases (DLC´s).

Section 3.2.2.9 of DNVGL-ST-0119 shall be taken into account when defining the DLCs that involve gusts. The gust events presently specified are based on a duration of 10.5 seconds that may be not sufficient for dynamic characteristics of the floating offshore wind turbines.

The duration of the events shall be selected accounting for the natural periods of the platform, without disregarding the 10.5 seconds currently specified in the standards.

9.3 WAVES

The wave data for this site have been extracted from the data provided by the SIMAR point 4038006, from the Spanish Ports Authority.

9.3.1 Extreme Waves

Return period (years)	Hs (m)	Tp (s)
50	5.11	9.0 - 11.0
20	4.69	9.0 – 11.0
10	4.40	9.0 – 11.0
1	3.35	8.0 – 10.0

Table 9.3-1 Wave data for GC

The peak periods shown above correspond to the most probable occurrence as shown in the scatter diagram in next section. A sensitivity analysis might be required for identifying the critical sea states for each floater.

9.3.2 Waves Scatter Diagram

The following table shows the frequency distributions of significant wave height and spectral peak period.

Peak Period (s)	%	Significant Wave Height (m) 0-1	1-2	2-3	3-4	4-5	Total
	1-2	0.037	0.001	0	0	0	0.038
	2-3	0.771	0.3	0	0	0	1.071
	3-4	2.603	1.845	0	0	0	4.448
	4-5	4.524	5.132	0.003	0	0	9.659
	5-6	5.392	10.973	0.049	0	0	16.414
	6-7	4.907	14.608	0.465	0	0	19.980
	7-8	4.211	9.569	2.593	0.012	0	16.385
	8-9	3.504	5.006	2.552	0.11	0	11.172
	9-10	2.836	3.119	1.087	0.147	0.001	7.190
	10-11	2.252	1.865	0.522	0.073	0.003	4.715
	11-12	1.766	1.250	0.275	0.028	0	3.319
	12-13	1.244	0.823	0.161	0.005	0	2.233
	13-14	0.827	0.542	0.12	0.001	0	1.490
	14-15	0.512	0.326	0.085	0.002	0	0.925
	15-16	0.27	0.21	0.052	0.003	0	0.535
	16-17	0.129	0.119	0.034	0.001	0	0.283
	17-18	0.04	0.058	0.005	0	0	0.103
	18-19	0.01	0.018	0	0	0	0.028
	19-20	0.001	0.006	0.001	0	0	0.008
	20-21	0	0.002	0	0	0	0.002
	21-22	0	0.001	0	0	0	0.001
	Total	35.84	55.77	8.004	0.382	0.004	100.000

Table 9.3-2 Significant wave height – Peak period frequency for GC

9.3.3 Wave Rose

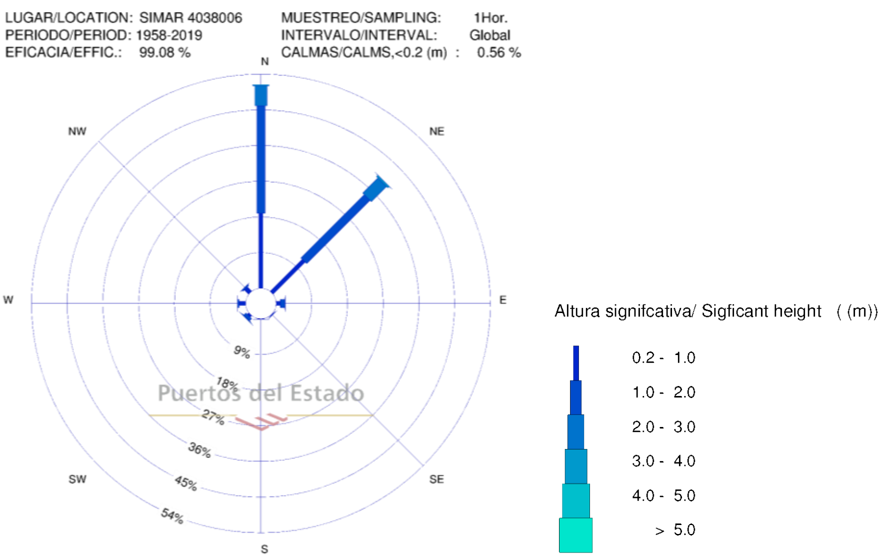

Figure 9.3-1. Gran Canaria Site wave rose (Significant wave height)

The following table gathers up dominant wave direction for the different incoming wave direction sectors. The direction, clockwise from true North, is from which the waves are travelling.

		Wave direction (º)							
		0	45	90	135	180	225	270	315
Hs (m)	0.0-0.5	1.154	0.439	0.059	0.017	0.004	0.000	0.171	0.322
	0.5-1.0	10.887	7.317	0.591	0.094	0.033	0.262	0.631	0.912
	1.0-1.5	17.103	14.722	0.892	0.155	0.042	0.569	0.431	0.661
	1.5-2.0	12.711	11.966	0.556	0.066	0.019	0.362	0.141	0.094
	2.0-2.5	5.260	5.626	0.105	0.012	0.017	0.122	0.012	0.011
	2.5-3.0	1.809	2.180	0.018	0.000	0.001	0.050	0.015	0.010
	3.0-3.5	0.387	0.530	0.000	0.000	0.004	0.056	0.001	0.000
	3.5-4.0	0.100	0.163	0.000	0.000	0.004	0.016	0.000	0.000
	4.0-4.5	0.001	0.060	0.000	0.000	0.000	0.000	0.000	0.000
	4.5-5.0	0.000	0.024	0.000	0.000	0.000	0.000	0.000	0.000
	5.0-10	0.000	0.004	0.000	0.000	0.000	0.000	0.000	0.000

Table 9.3-3 Wave direction for Gran Canaria

9.3.4 Waves Spectrum

A two-peak power spectrum model shall be used, based on the recommendations given in DNV standards Ref.[S6].

For this project, same philosophy as in LIFES50+ is used for NSS that will use values in Table 9.3-2.

For this project, same philosophy as in LIFES50+ is used for SSS and ESS that will use values in Table 9.3-1.

9.4 WIND – WAVES COMBINED CONDITIONS

No data of the correlation between wind speed and wave height is available at the Southeast coast of Gran Canaria. The following joint distribution corresponds to the Northeast coast of Gran Canaria, in the PLOCAN area. The extreme values are very close together, so it is considered to be a good reference.

9.4.1 Wind-Wave Scatter Diagram

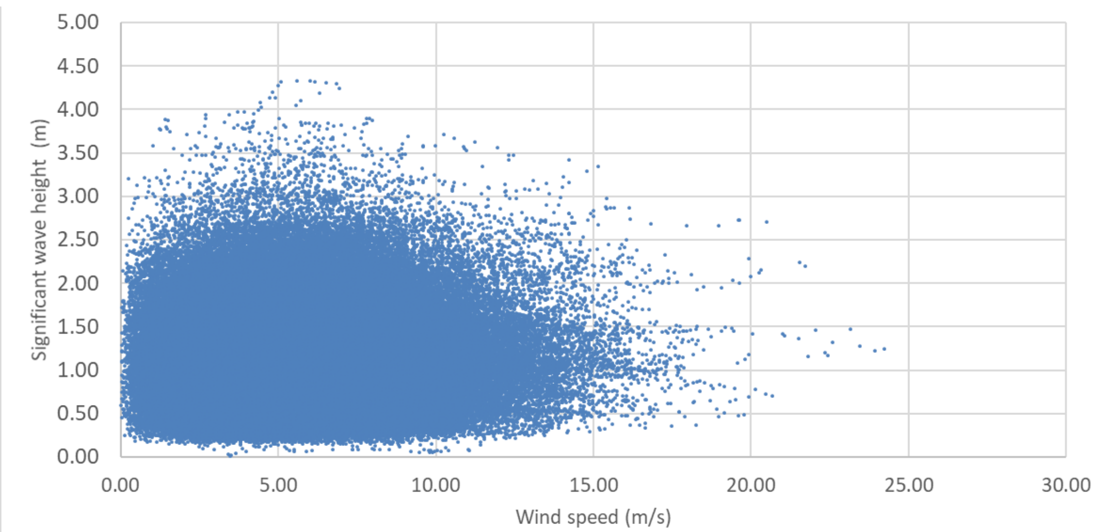

Figure 9.4-1. Joint distribution between wind speed and significant wave height at GC

The data from the graph above has been processed in the following scatter diagram.

Significant Wave Height [m]	WIND SPEED (1-hour at 10 m)										
	0.00 - 2.00	2.00 - 4.00	4.00 - 6.00	6.00 - 8.00	8.00 - 10.00	10.00 - 12.00	12.00 - 14.00	14.00 - 16.00	16.00 - 18.00	18.00 - 20.00	>20.00
0.00 - 1.00	2.083	8.396	12.354	8.754	4.174	1.685	0.588	0.144	0.044	0.010	0.001
1.00 - 2.00	3.012	12.063	18.533	12.298	5.582	2.195	0.777	0.248	0.062	0.010	0.006
2.00 - 3.00	0.384	1.387	2.041	1.568	0.785	0.295	0.126	0.055	0.012	0.003	0.002
3.00 - 4.00	0.014	0.060	0.109	0.076	0.034	0.009	0.007	0.003			
4.00 - 5.00			0.005	0.003							
5.00 - 6.00											
6.00 - 7.00											
> 7.00											

Table 9.4-1 – Wind – Wave scatter diagram

In addition the 50 – year return period contour is given below.

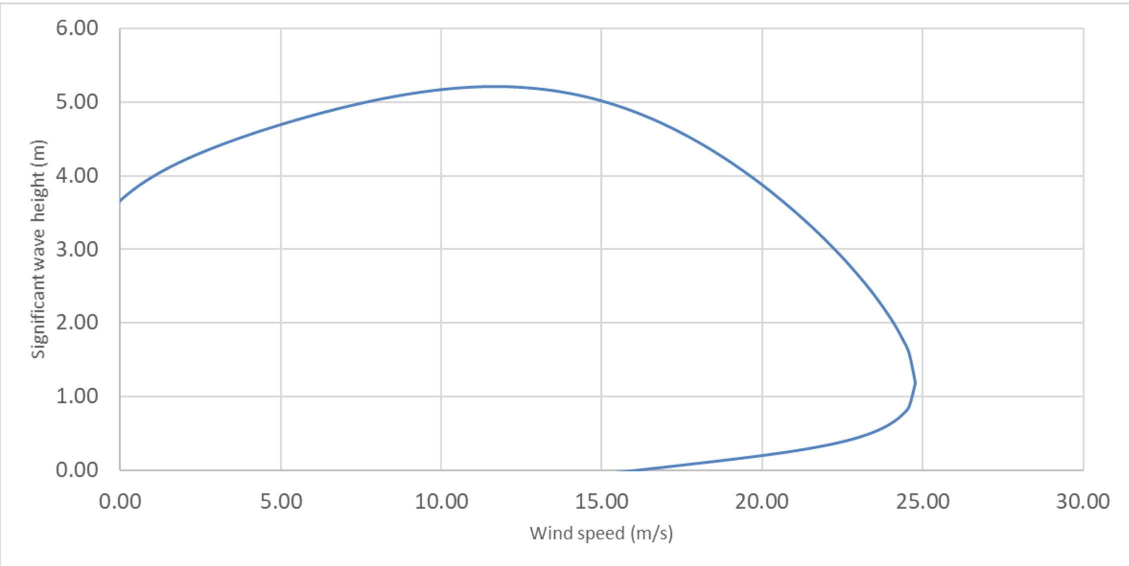

Figure 9.4-2. 50-year return period envelope at GC

Design assumptions are recommended in order to cover all cases, not disregarding cases with high waves and low wind speed and viceversa that are reflected in the scatter above.

9.4.2 Wind-Wave Misalignment

No metocean data is available about the correlation of wind direction and wave direction. Design assumptions have to be made based on wind and wave roses provided. **[HOLD]**

9.5 CURRENT CONDITIONS

The data available for the GC site, extracted from the simulation at SIMAR point 4038006, is not that abundant. Some of the data will be extrapolated.

9.5.1 Current induced by wind

This value will be obtained using the same formula than in GC:

$$Vc(s0) = k\ U_{1hour}$$

Where (k) coefficient will be taken as 0.03 in order to account for the worst-case scenario and obtain a safety side current speed value. The 50-year 1-hour maximum annual wind speed at 10 m from the sea level is 19.0 m/s, so the current speed induced by wind is:

CURRENT INDUCED BY WIND DATA FOR GRAN CANARIA, SPAIN	
Current speed for a T_R of 50 years (c_{50})	0.57 m/s

Table 9.5-1 Current induced by wind speed at sea surface at GC

The direction associated to these current speed values will be taken as the most probable wind direction obtained from the scatter diagram. Therefore, wind induced current direction will be taken as North-northeast to South-southwest direction for all cases.

9.5.2 Deep Water Current

There are no available data for the southeast coast of Gran Canaria, so it is proposed to use the available data for the PLOCAN area in the northeast coast of the island. The direction of the current is parallel to the coast, following tidal patterns, so goes NNE and SSW twice a day.

DEEP WATER CURRENT DATA FOR GRAN CANARIA, SPAIN	
Current speed for a T_R of 50 years (c_{50})	0.49 m/s
Direction of the current (º)	22.5 – 202.5

Table 9.5-2 Deep water current at surface at GC

9.5.3 Current Speed profile

There is no information available on the current profile, so the same ones used above for West of Barra are to be used here:

Current induced by wind

$$v_{c,wind}(z) = v_{c,wind}(0) \cdot \left(\frac{d_0 + z}{d_0}\right) \; for - d_0 \leq z \leq 0$$

Where $d0$ is taken as half of the water depth at West of Barra following DNVGL recommendations, hence $d0 = 125\ m$.

Tidal current

$$v_{c,tide}(z) = v_{c,tide}(0) \cdot \left(\frac{d + z}{d}\right)^{\alpha} \; for\ z \leq 0$$

Resulting current speed profiles for each of the currents defined in previous sections are given in the following table 50-year return period current. Last column of this table represents the vectorial summation of the aforementioned component. Since both components run parallel to the coast, their values are directly added.

Depth (m)	Wind component (m/s)	Tidal component (m/s)	Total current speed (m/s)
0.00	0.57	0.49	1.06
-10.00	0.52	0.49	1.01
-20.00	0.48	0.48	0.96
-30.00	0.43	0.48	0.91
-40.00	0.39	0.48	0.87
-50.00	0.34	0.47	0.82
-60.00	0.30	0.47	0.77
-70.00	0.25	0.47	0.72
-80.00	0.21	0.46	0.67
-90.00	0.16	0.46	0.62
-100.00	0.11	0.46	0.57
-110.00	0.07	0.45	0.52
-120.00	0.02	0.45	0.47
-130.00	0.00	0.44	0.44
-140.00	0.00	0.44	0.44
-150.00	0.00	0.43	0.43
-160.00	0.00	0.42	0.42
-170.00	0.00	0.42	0.42
-180.00	0.00	0.41	0.41
-190.00	0.00	0.40	0.40
-200.00	0.00	0.39	0.39
-210.00	0.00	0.38	0.38
-220.00	0.00	0.36	0.36
-230.00	0.00	0.34	0.34
-240.00	0.00	0.31	0.31
-250.00	0.00	0.00	0.00

Table 9.5-3 Total current speed profile associated to the 50-year return period probability

9.5.4 Current Direction

No data are available about current direction. Conservative assumption shall be made during design analyses.

9.6 ICE LOADS

The Canary Islands are located in the subtropical region in the middle of the Atlantic Ocean. There is no written register of sea ice in the area. Regarding snow it is unlikely to occur at sea level.

9.7 OTHER CONDITIONS

9.7.1 Water Temperature

Over the last 20 years, the water temperature varied from 17.4ºC in winter to 25.6ºC in summer (extreme values).

Boya de Gran Canaria 1997 - 2018				
Mes/*Month*	Ts Max./*Max. Ts*	Año/*Year*	Dia/*Day*	Hora/*Hour*
Enero/*January*	21.7	2011	06	18
Febrero/*February*	20.8	2010	25	21
Marzo/*March*	20.7	2002	21	19
Abril/*April*	21.4	2003	29	00
Mayo/*May*	22.1	2017	10	18
Junio/*June*	23.3	2009	29	05
Julio/*July*	23.3	2017	28	17
Agosto/*August*	25.1	2004	29	00
Septiembre/*September*	25.6	2012	26	20
Octubre/*October*	25.6	2015	03	17
Noviembre/*November*	24.7	2015	04	18
Diciembre/*Dececember*	22.6	2009	01	05

Table 9.7-1 Max surface temperature of water for GC (º)

Boya de Gran Canaria 1997 - 2018				
Mes/*Month*	Ts Min./*Min. Ts*	Año/*Year*	Dia/*Day*	Hora/*Hour*
Enero/*January*	18.3	2018	30	10
Febrero/*February*	17.6	2015	21	00
Marzo/*March*	17.4	2015	29	01
Abril/*April*	17.7	2015	01	01
Mayo/*May*	18.0	2015	22	08
Junio/*June*	18.9	2018	07	01
Julio/*July*	20.4	2017	01	10
Agosto/*August*	20.9	2009	03	07
Septiembre/*September*	21.3	2017	08	07
Octubre/*October*	21.1	2008	31	23
Noviembre/*November*	20.0	2008	30	23
Diciembre/*Dececember*	19.0	2008	17	23

Table 9.7-2 Max and min mean surface temperature of water for GC (º)

9.7.2 Air Temperature

Air temperature ranges from 17ºC to 30ºC over the last 20 years.

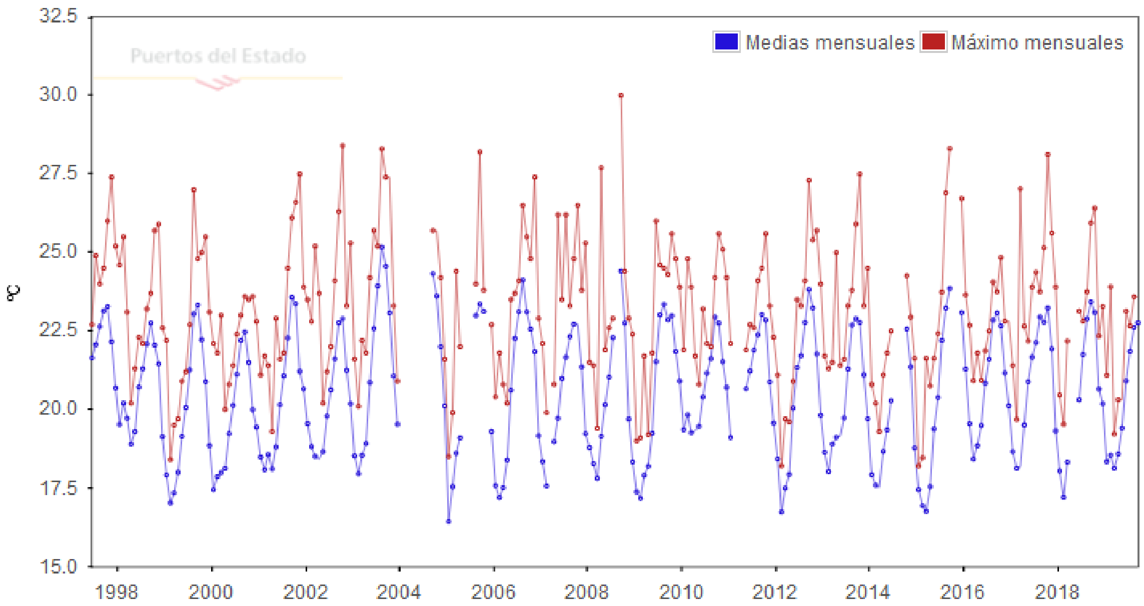

Table 9.7-3 Monthly Mean (blue) and monthly max(red) air temperature (1998-2018) for GC

9.7.3 Air Density

Air density is 1.225 kg/m^3.

10 ENVIRONMENTAL DATA.SITE C. MORRO BAY

All data provided in this section are extracted from study provided by FIHAC.

10.1 WATER DEPTHS AND LEVELS

The design water depth at Morro bay is set to 870 m. Summary of water levels is given below:

WATER LEVELS FOR GRAN CANARIA	
Highest Still Water Level (HSWL)	2.98
Highest Astronomical Tide (HAT)	2.73
Mean Sea Level (MSL)	1.45
Lowest Astronomical Tide (LAT)	0.00
Lowest Still Water Level (LSWL)	-0.15

Table 10.1-1 Water levels for Morro Bay

These values have been taken from the tide gauge in Monterey from NOAA. The highest and lowest correspond to the extreme observed values. It is proposed to be used as the 50 year return period values. For other return periods no data have been found.

10.2 WIND

The main reference considered when evaluating the wind conditions of Morro Bay site is the report issued by FIHAC. This document states that all the data available are 1-hour averaged wind speeds at 10 m above MSL, so all the numbers will be generated by extrapolating to 10-minute averaged and to other heights.

As per IEC 61400-3-1 the conversion between 1 hour average winds and 10 minutes will be done applying a factor of 0.95.

10.2.1 Normal Wind Profile

The available data in the area about normal wind speeds are summarized in the following figure.

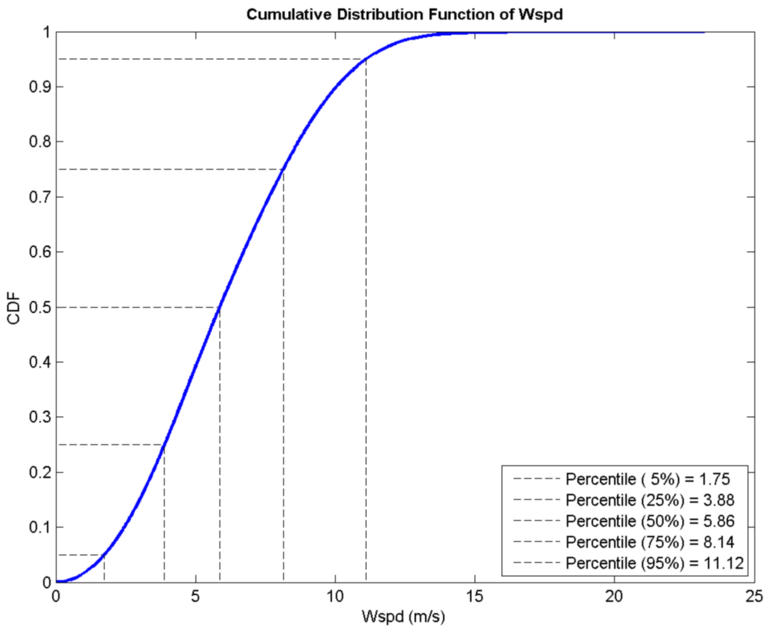

Figure 10.2-1 – Mean wind speed Distribution function

Based on above figure, the mean average speed is assumed as 5.86 m/s which in a 10 minutes average means 6.17 m/s (10-min) at 10 meters.

The wind speed profile in normal conditions has been produced assuming the following power law as indicated in IEC 61400-1.

$$V(z) = V_{\text{hub}}(z/\, z_{\text{hub}})^{\alpha}$$

The alpha factor is taken as 0,2 as recommended in the IEC standard.

The resulting 10-minute mean wind speed profile is the following:

Normal Wind Profile	
Height [m]	Speed [m/s]
10	6.2
20	7.1
50	8.5
100	9.8
119	10.1
150	10.6

Table 10.2-1 Normal wind speed profile for Morro Bay

10.2.2 Extreme Wind Profile

Extreme wind speed data is available from the FIHAC metocean study of the area. The summary of that data is included in the following figure.

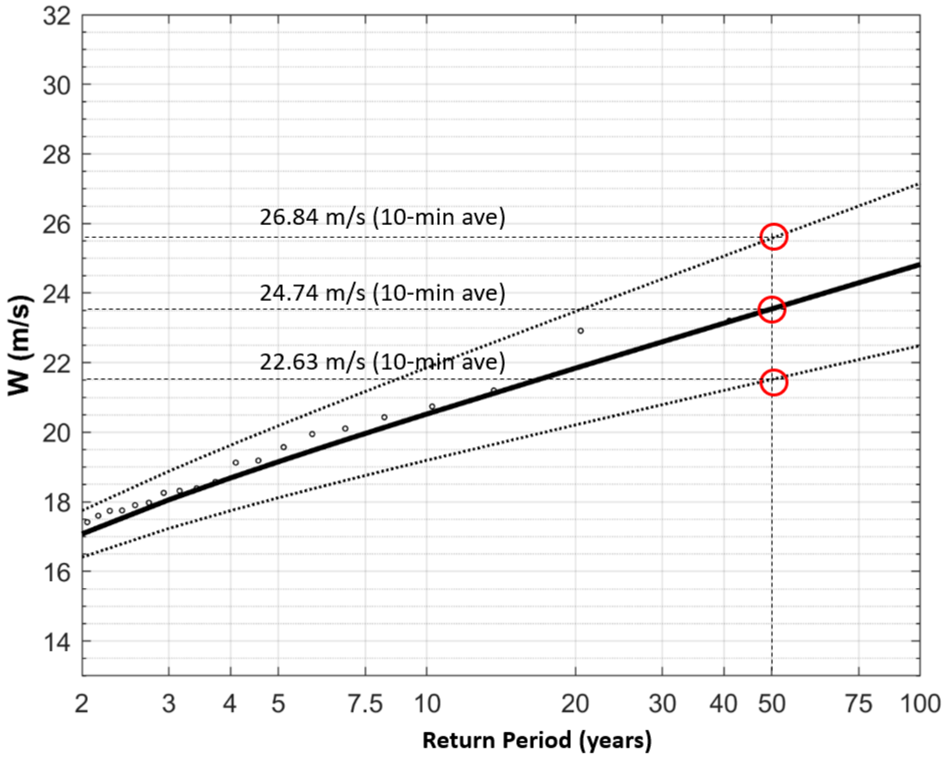

Figure 10.2-2 – Extreme Wind speeds (1-h at 10 m)

The upper value of the fitted laws is taken, i.e. 26.84 m/s at 10 min and 10 meters height.

The extreme wind profile is deduced using a power law similar to the normal wind profile but using an alpha factor of 0.12.

Extreme Wind Profile Tr=50-year	
Height	Speed
[m]	[m/s]
10	26.84
20	29.17
50	32.56
100	35.38
119	36.16
150	37.15

Table 10.2-2 – Extreme wind profil for Morro Bay (Tr=50 years)

The 1-year wind speed is estimated to be 17 m/s (1-hour and 10 meters). The following table shows the 1 year return period profile.

Extreme Wind Profile Tr=1-year	
Height	Speed
[m]	[m/s]
10	17.89
20	19.45
50	21.71
100	23.59
119	24.11
150	24.77

Table 10.2-3 – Extreme wind profile for Morro Bay (Tr=1 year)

10.2.3 Wind Speed Histogram

Wind speed histogram is provided in figure below. Note that is based in 1-hour at 10 meters observations.

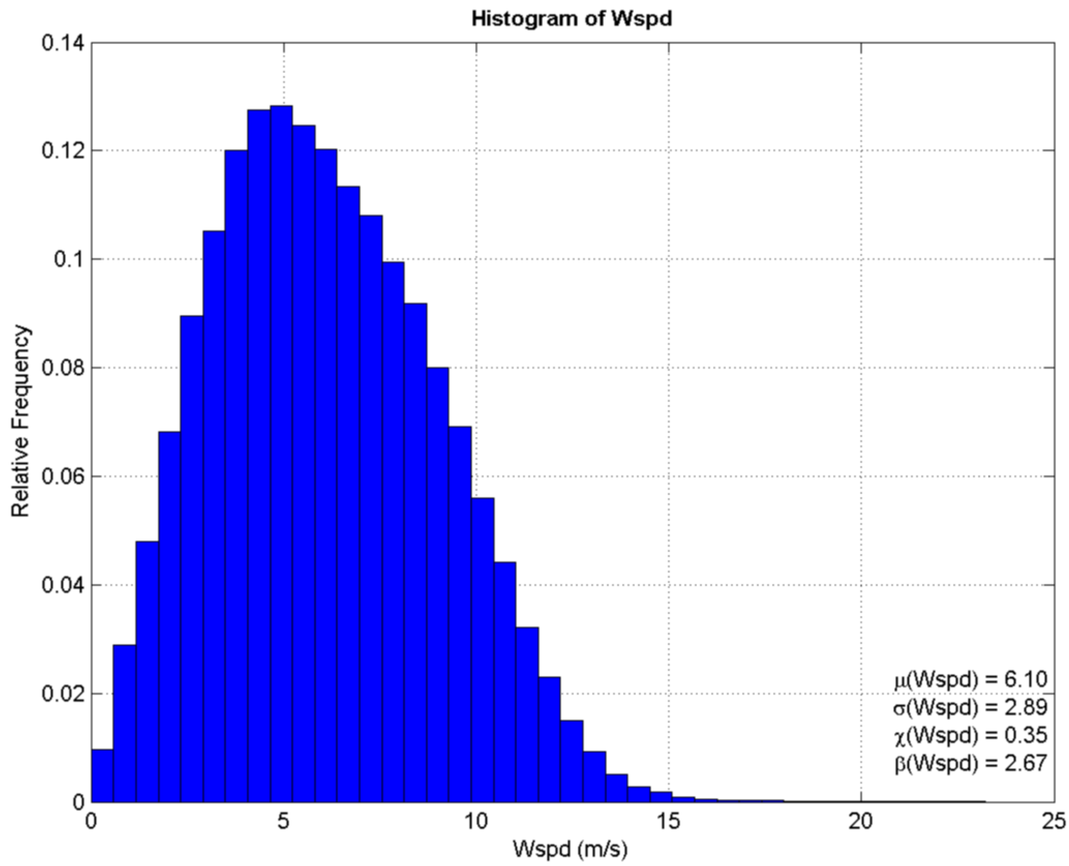

Figure 10.2-3 – Wind speed histogram for Morro Bay

10.2.4 Wind Speed Rose

Wind speed rose is provided in the following 3 figures.

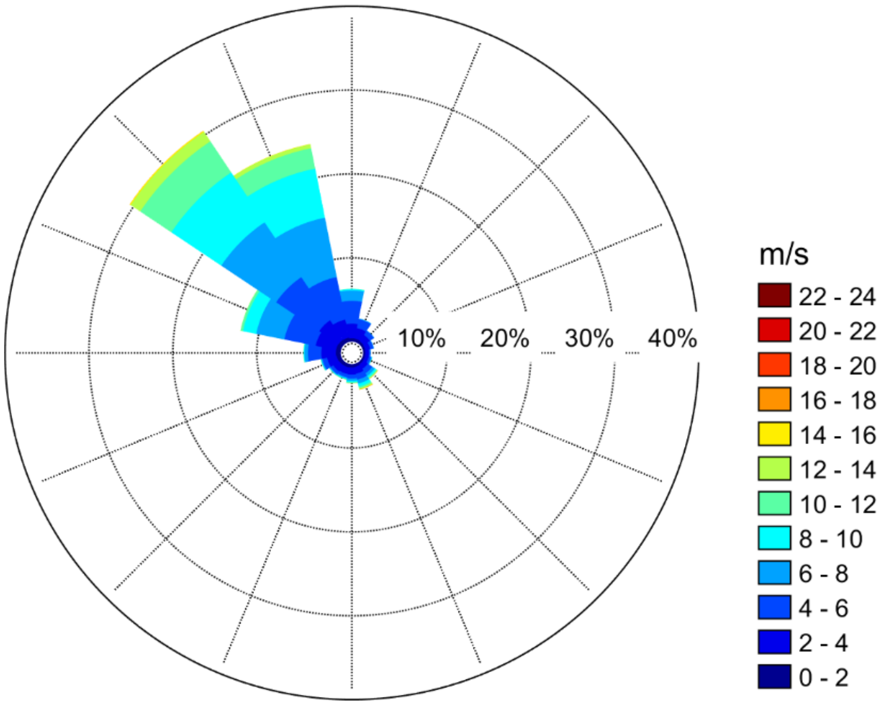

Figure 10.2-4 – Wind speed rose

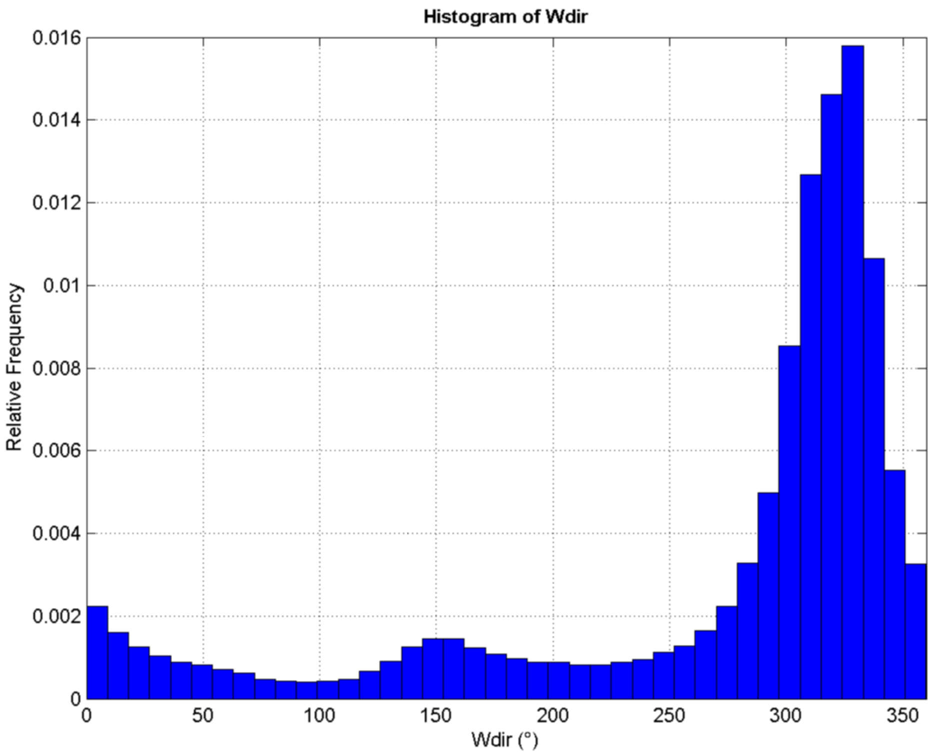

Figure 10.2-5 – Histogram of wind direction

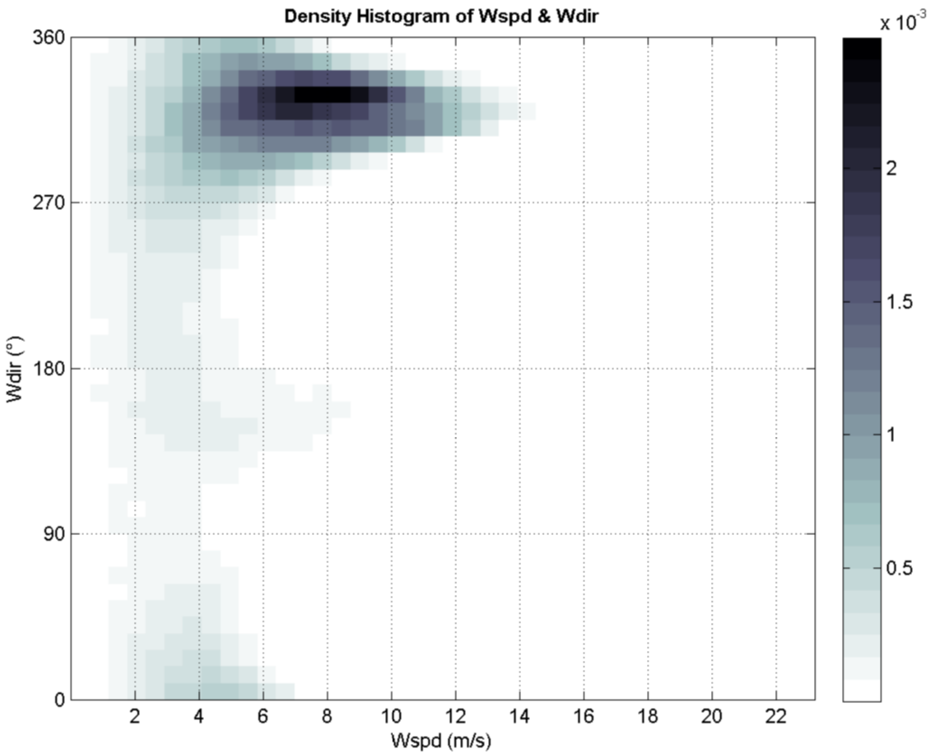

Figure 10.2-6 – Histogram wind speed and Wind direction

10.2.5 Turbulence intensity

There is no specific data for the site turbulence, so it is assigned a Class C, as described in IEC-61400-1 based on the extreme wind speed at 150 meters which is the preliminary selected hub height.

10.2.6 Wind Spectrum

In absence of more detailed information and following DNVGL recommendations, it has been decided to assume the Kaimal model as the most representative of wind spectral density at Gran Canaria. The Kaimal model provides de distribution of wind energy over the different frequencies.

For this project, same philosophy as in LIFES50+ is used for NTM and ETM that will use values in Table 8.2-6.

10.2.7 Wind Gust Characteristics

No information is available at Morro Bay site in regard to wind gust. Hence, reference is made to IEC-61400-1, where it can be found mathematical models that allow characterizing wind gust and accounting for its effects on the design load cases (DLC´s).

Section 3.2.2.9 of DNVGL-ST-0119 shall be taken into account when defining the DLCs that involve gusts. The gust events presently specified are based on a duration of 10.5 seconds that may be not sufficient for dynamic characteristics of the floating offshore wind turbines.

The duration of the events shall be selected accounting for the natural periods of the platform, without disregarding the 10.5 seconds currently specified in the standards.

10.3 WAVES

The wave data for this site have been extracted from the data provided by FIHAC.

10.3.1 Extreme Waves

The data available is summarized in the following figure.

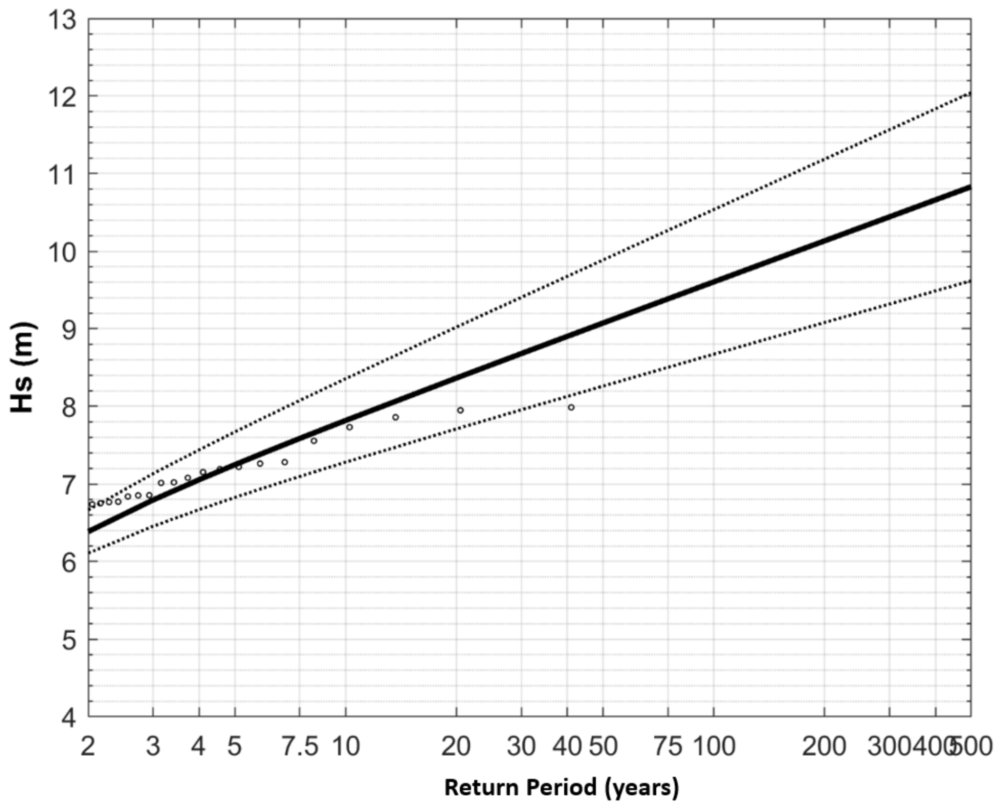

Figure 10.3-1 – Extreme waves data at MB

According to the above figure, the representative values to be used in the analyses are extracted. Note that the conservatively the upper fit of the data has been considered. The peak periods have been preliminary selected from the scatter diagrams in next section.

Return period (years)	Hs (m)	Tp (s)
50	9.9	16.0 - 18.0
20	9.0	16.0 - 18.0
10	8.3	16.0 - 18.0
1	6.0	12.0 – 16.0

Table 10.3-1 – Extreme Waves at Morro Bay

10.3.2 Waves Scatter Diagram

The following table shows the frequency distributions of significant wave height and spectral peak period.

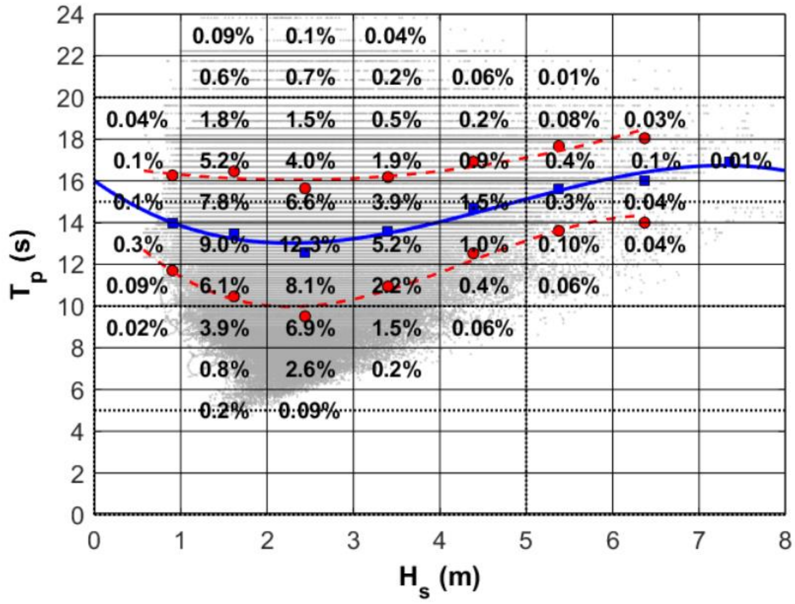

Figure 10.3-2 – Scatter diagram Hs – Tp

The laws indicated in Figure 10.3-2 are analiticly expressed below:

μ

$\mu \pm \sigma$

$$\mu(x|H_s) = a_o + a_1 H_s + a_2 H_s^2 + a_3 H_s^3$$
$$\sigma(x|H_s) = a_o + a_1 H_s + a_2 H_s^2 + a_3 H_s^3$$
$$x = \{T_p, T_m\}$$

T_p	a_o	a_1	a_2	a_3
μ (T_p|H_s)	16.007	-2.941	0.849	-0.059
σ (T_p|H_s)	0.811	2.281	-0.708	0.060

Figure 10.3-3 – Analytic Scatter diagram

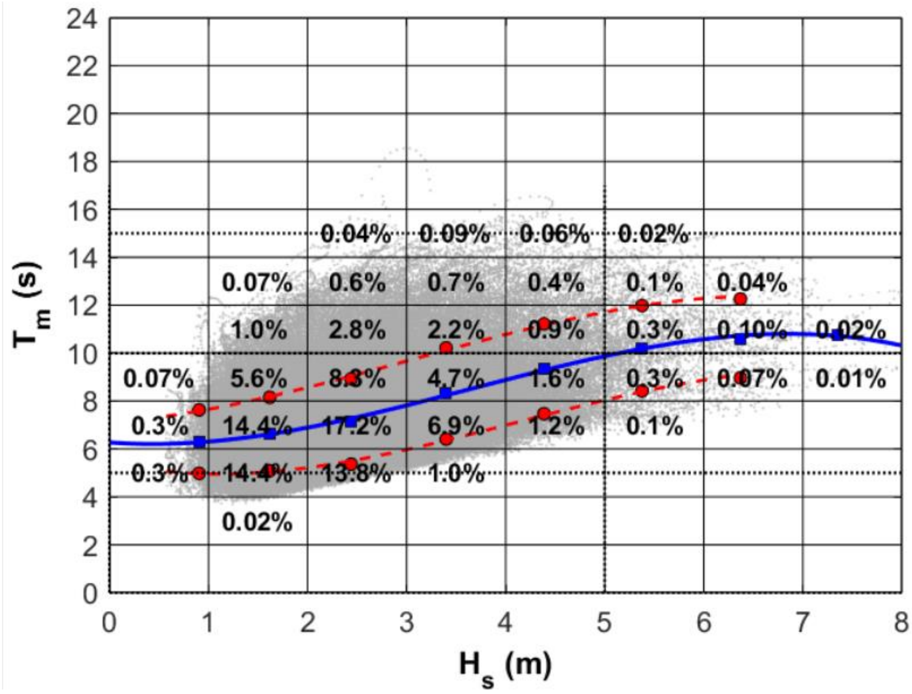

Figure 10.3-4 – Scatter Diagram Hs – Tz at Morro Bay

The laws indicated in Figure 10.3-4 are analiticly expressed below:

μ

$\mu \pm \sigma$

$$
\begin{aligned}
\mu(x|H_s) &= a_o + a_1 H_s + a_2 H_s^2 + a_3 H_s^3 \\
\sigma(x|H_s) &= a_o + a_1 H_s + a_2 H_s^2 + a_3 H_s^3 \\
x &= \{T_p, T_m\}
\end{aligned}
$$

T_m	a_o	a_1	a_2	a_3
μ $(T_m \| H_s)$	6.274	-0.312	0.380	-0.035
σ $(T_m \| H_s)$	0.842	0.604	-0.102	0.004

Figure 10.3-5 – Analytic Scatter Hs -Tz at Morro Bay

10.3.3 Wave Rose

The following figures indicate the directionality of the sea states.

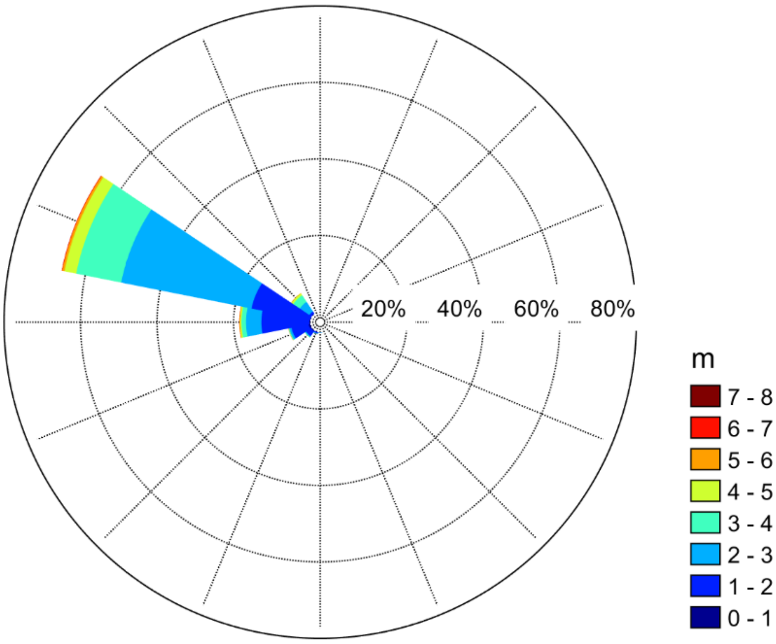

Figure 10.3-6 – Wave Rose at Morro Bay

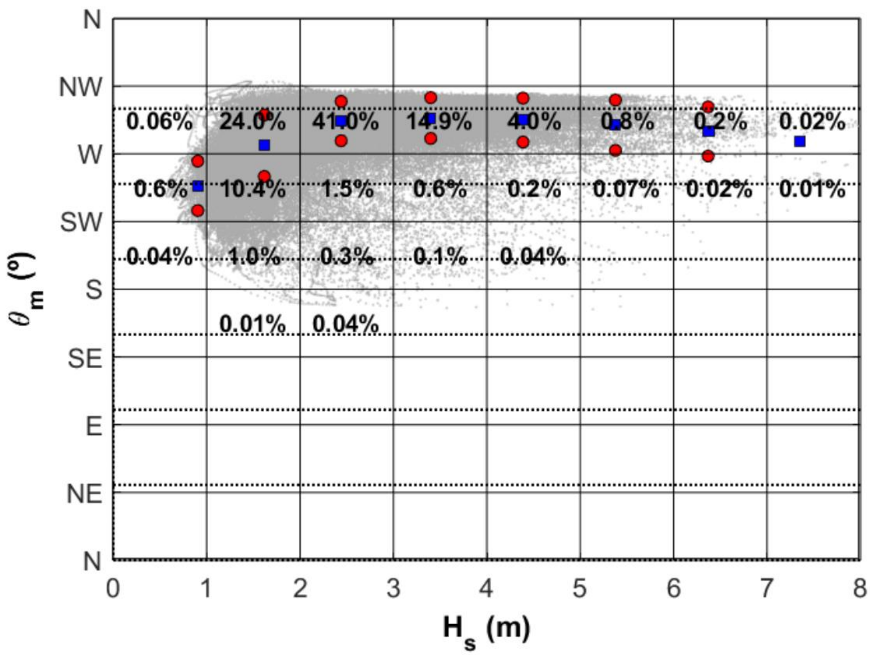

Figure 10.3-7 – Wave direction Scatter diagram at MB

10.3.4 Waves Spectrum

A two-peak power spectrum model shall be used, based on the recommendations given in DNV standards Ref.[S6].

For this project, same philosophy as in LIFES50+ is used for NSS that will use values in Figure 10.3-2.

For this project, same philosophy as in LIFES50+ is used for SSS and ESS that will use values in Figure 10.3-1.

10.4 WIND – WAVES COMBINED CONDITIONS

10.4.1 Wind-Wave Sactter Diagram

Significant Wave Height [m]	WIND SPEED (1-hour at 10 m)										
	0.00 - 2.00	2.00 - 4.00	4.00 - 6.00	6.00 - 8.00	8.00 - 10.00	10.00 - 12.00	12.00 - 14.00	14.00 - 16.00	16.00 - 18.00	18.00 - 20.00	>20.00
0.00 - 1.00	0.084	0.270	0.280	0.084	0.004						
1.00 - 2.00	3.103	9.247	10.896	8.021	3.621	0.550	0.017	0.002			
2.00 - 3.00	2.453	7.425	9.817	9.540	8.452	4.282	0.781	0.037	0.006	0.001	
3.00 - 4.00	0.665	2.201	3.296	3.293	2.911	2.227	0.897	0.118	0.014	0.003	0.001
4.00 - 5.00	0.215	0.529	0.779	0.857	0.818	0.619	0.302	0.088	0.014	0.002	0.003
5.00 - 6.00	0.047	0.127	0.165	0.171	0.169	0.104	0.082	0.037	0.014	0.003	0.002
6.00 - 7.00	0.008	0.023	0.033	0.041	0.036	0.028	0.025	0.017	0.005	0.001	0.001
> 7.00		0.003	0.004	0.005	0.004	0.002	0.005	0.007	0.004	0.002	

Table 10.4-1 – Wind – Wave Scatter Diagram

10.4.2 Wind-Wave Misalignment

A preliminary analysis of the data provided by FIHAC has been done regarding the wind wave misalignment. The following table shows the misalignment versus the probability (in percentage).

Misalignment	Prob (%)
0-10	9.20
10-20	13.23
20-30	16.50
30-40	15.58
40-50	11.27
50-60	7.70
60-70	5.26
70-80	3.91
80-90	3.02
90-100	2.51
100-110	2.22
110-120	2.01
120-130	1.85
130-140	1.60
140-150	1.34
150-160	1.09
160-170	0.91
170-180	0.82

Table 10.4-2 – Wave-Wind Misalignment

10.5 CURRENT CONDITIONS

No current speed information is available for Morro Bay area. **[HOLD]**

10.6 ICE LOADS

The Morro Bay site is located in a warm zone. There are no registers of below zero temperatures and therefore it is considered negligibles ice loads of any kind.

10.7 OTHER CONDITIONS

10.7.1 Water Temperature

Morro bay water temperatures peak in the range of 18 to 22 degrees Celsius with minimums of around 13 to 16 degrees.

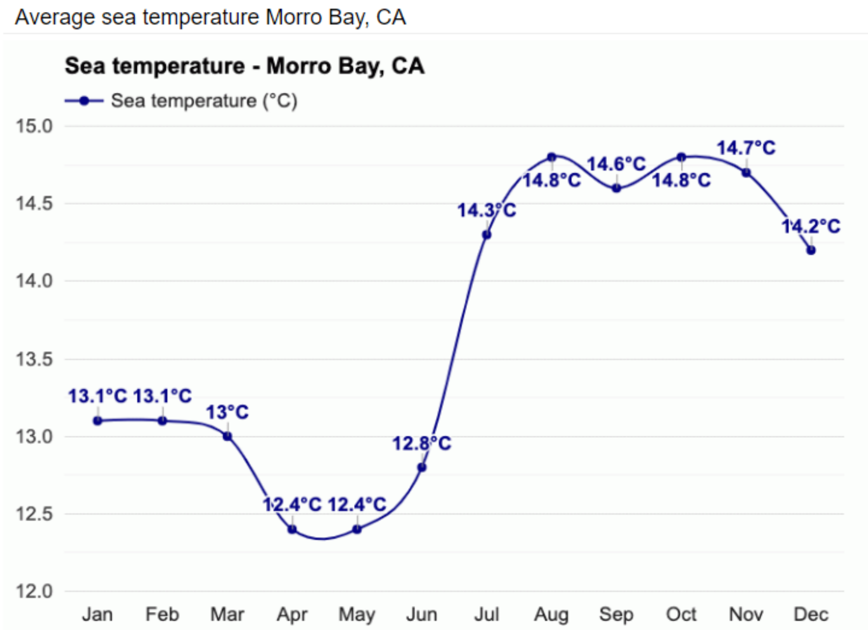

Figure 10.7-1 – Sea temperature at Morro Bay

10.7.2 Air Temperature

The temperature in Morro bay is very stable as it can be seen in the following figure.

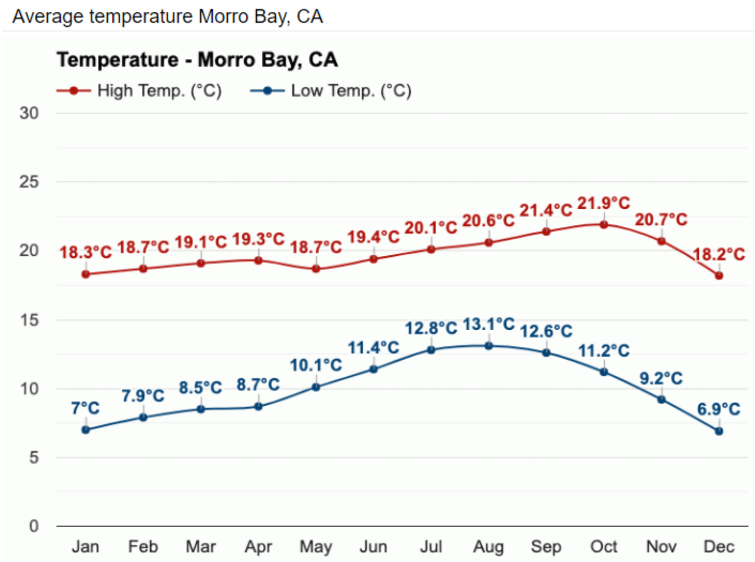

Figure 10.7-2 - Air temperature at Morro Bay

10.7.3 Air Density

Air density shall be considered 1.225 kg/m^3.

11 SOIL CONDITIONS. SITE A. WEST OF BARRA ISLAND

The data in this section have been obtained from LIFES50+ project. West of Barra site lies entirely over rocky sea bottom that has been deepened by glacial scouring action. The predominant rock type is Lewisian gneiss, which has a similar hardness to granite.

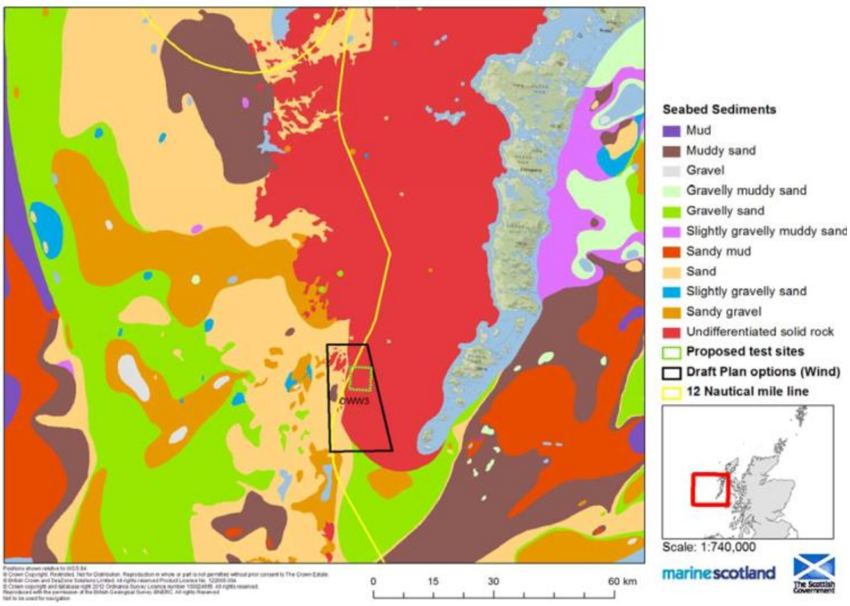

Figure 10.7-1. West of Barra seabed general characteristics

A multibeam bathymetry is provided form a nearby area located approximately 30 km North from West of Barra site. This information has been gathered from Joint Nature Conservation Committee report.

As shown in the next figure, seabed is dominated by extensive areas of highly fractured bedrock. The fractures form a regular network of gullies, some as wide as 130m with sides up to 30m in height. Although not extensively ground-truthed, the gullies appear to be infilled by coarse sands.

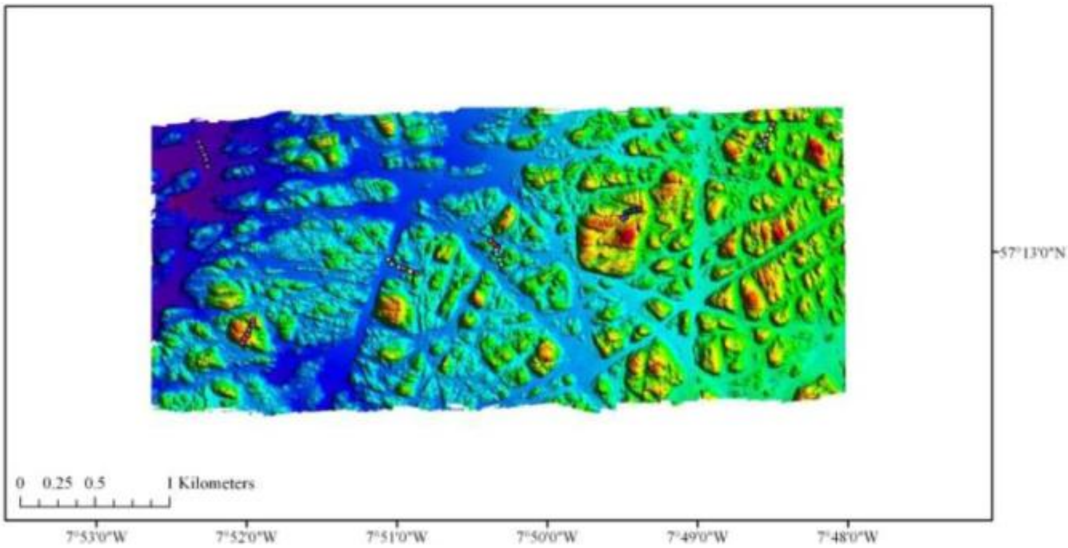

Figure 10.7-2. Multibeam bathymetry of an area in the vicinity of West of Barra

With all this general information gathered for the seabed characterization a standard soil profile for the characterization of the West of Barra site seabed is defined.

Soil Profile Characteristics			
Layer	Soil Type	Layer Length (m)	Compressive strength (MPa)
1	Rock (Basalt)	20	200

Table 10.7-1 Soil profile characteristics for WoB

12 SOIL CONDITIONS. SITE B. GRAN CANARIA ISLAND

The soil in the area is known from previous experiences with projects in the area, although no geotechnical report is available for the specific site.

The first meters from the shore are pebbles, up to 15 m of depth. The next area, up to 60 m deep, has variated granulometries of sand; and further down, we can find sand with bioclasts.

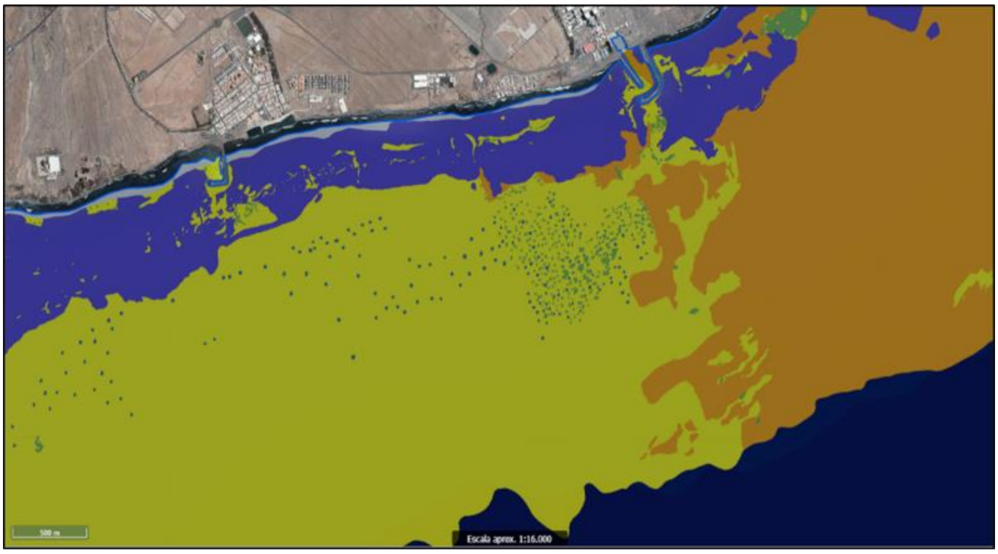

Figure 10.7-1. Geology in GC

Yellow corresponds to fine sand, while orange is coarse sand. The depth of the exterior line is 60 m.

The design soil profile can be considered as a continuos layer of sand, with the following design parameters:

Soil Profile Characteristics	
Internal friction angle	35 º
Cohesion	0 kPa
Unit weight	20 kN/m3
Deformation modulus for large strains	30 MPa
Deformation modulus for small strains	150 MPa
Poisson ratio	0.3
Shear modulus for large strains	12 MPa
Shear modulus for small strains	60 MPa

Table 10.7-1 Geotechnical parameters at GC

13 SOIL CONDITIONS. SITE C. MORRO BAY

Little information is available about Morro Bay soil conditions. The US government offers 1:35.000 cartography about the offshore geology. Map 3327 sheet 3 shows the surrounding area of the selected site.

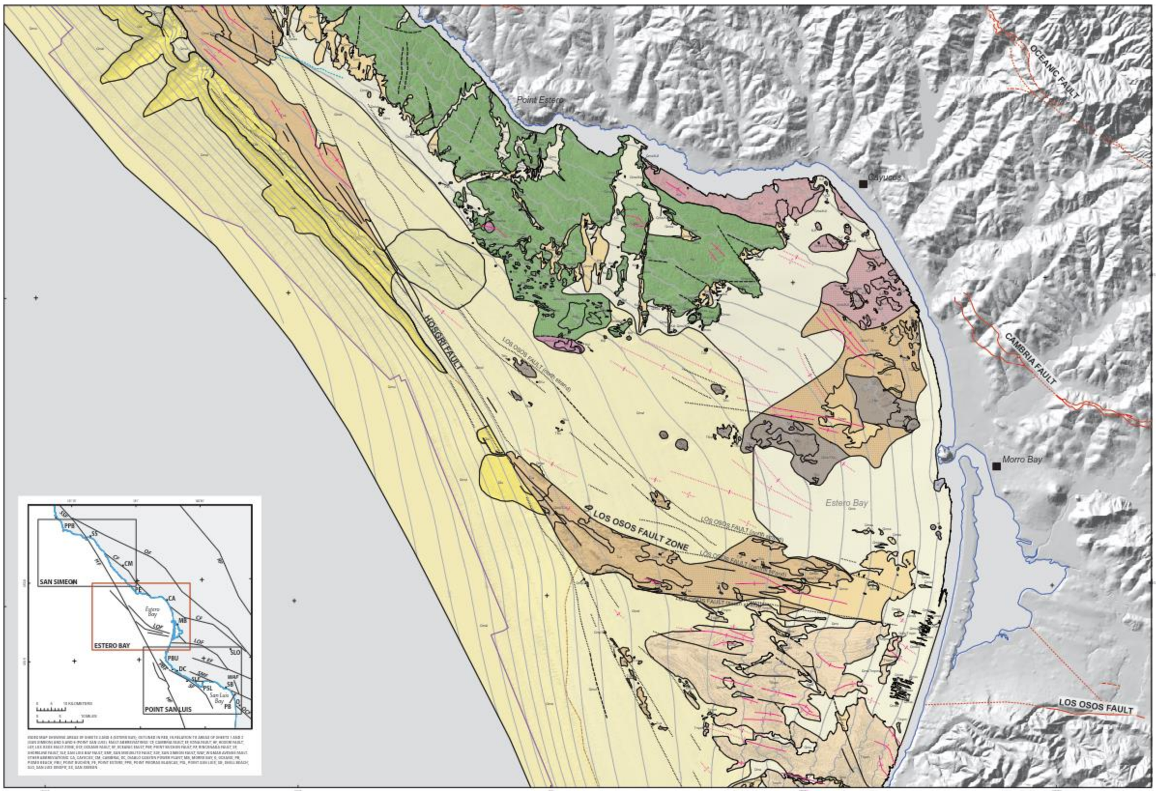

Figure 10.7-1 – Offshore geology of the area

The deeper area is classified as Marine slope deposits with a composition of sand and mud. Other sources allow to set that the thickness of these sediments can be considered over 300 m.

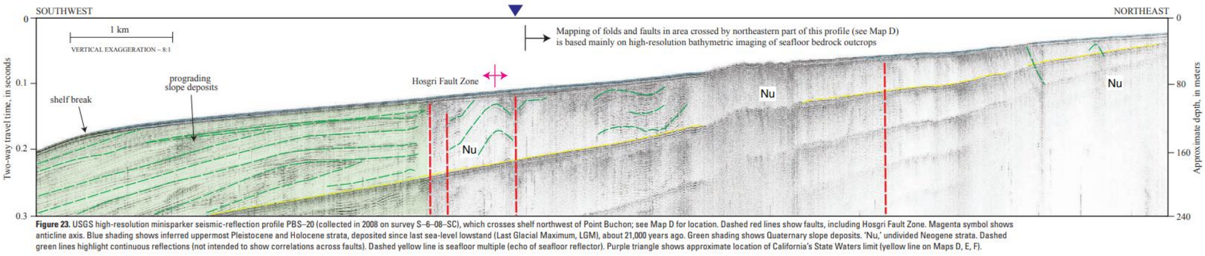

Figure 10.7-2 – Geology profile

In the absent of more geotechnical information it can be considered medium dense sands for design purposes and typical parameters in Ref.[S1] used.

Soil type	*Friction angle φ*	*Submerged unit weight γ' (kN/m³)*	*Poisson's ratio ν*	*Void ratio e*
Loose	28 – 30º	8.5 – 11.0	0.35	0.7 – 0.9
Medium	30 – 36º	9.0 – 12.5	0.35	0.5 – 0.8
Dense	36 – 41º	10.0 – 13.5	0.35	0.4 – 0.6

Table 10.7-1 – Typical sand geotechnical values in Ref.[S1]

Some cohesion may be considered given the presence of mud in the geological description. A sensitivity analysis of the soil is recommended in order to assess the risks due to the lack of information.

14 FUNCTIONAL REQUIREMENTS

All FOWT for the COREWIND project are to be designed providing the following elements:

- Boat landing. Allocated in the most sheltered sector of the platform (opposite to the most likely occurring wind and waves combination)
- Access platform equipped with:
  - 1 davit crane allocated near to the laydown area on top of the boat landing. The crane will be strategically positioned to optimize support for construction and O&M operations. The outreach and hook capacity will be enough to access the laydown area.
  - Lay down area strategically positioned to lift weights with the davit crane from service ships. The area will be enough to guarantee safe lift operations including access and scape ways for personnel during such operations.
- Power cable guides (J tubes or similar) which allow the safe installation of the cable (pull in operations) and guarantee the in-service conditions during the lifetime of the platform.
- Internal platforms and tertiary structures for holding and accessing the switchgear, cabinets, etc.

15 AIR GAP

Minimum air gap must be such that:

- It is avoided the water contact of any downflooding point in the most severe conditions (50-years return period).
- There is no blade contact with the water.

Motions shall be considered when assessing air gap. Air gap for blade tip shall be calculated with an extra allowance of 1.5 m is to be used for air gap calculations as per Ref. [S2] which is conservative compered with Ref. [S1].

In case of not having a flooding point that may restrained the height of the access platform, the platform and all members on it shall be designed for slamming forces.

16 MARINE GROWTH

According to DNVGL-ST-0437 section 2.4.11, marine growth has to be taken in account, for both locations, following the data provided in the next table.

MARINE GROWTH THICKNESS (mm)	
From 2 m above the sea level to 40 m below it	100
40 m to 100 m below the sea level	50
100 m to 900 m below the sea level	**HOLD**

Table 10.7-1 Marine growth data

17 WIND FARM DESCRIPTION

This section may be included in order to propose a size of the wind farm in case some tasks involve assessments at windfarm level. **[HOLD]**

18 PRE-SERVICE PHASES

18.1 CONSTRUCTION PHASES

Pre-service condition shall be investigated if, once the fabrication and transport philosophy is defined, it is deemed that this phase may governed the design of certain areas. The following phases may mean critical conditions for certain members:

- Lifting operations of major sub-assemblies. If a fabrication based on prefabrication and transportation to an erection area, there might be phases not covered by the in-service calculations.
- Float off. The structures may be fabricated and transported to a quay side and afterwards put into the water by different means.

18.2 TRANSPORT AND INSTALLATION

Transport conditions may govern aspects of the structural design of the floating platforms, or some localized areas. Since the project is focused on the cost reduction of the mooring and dynamic cable systems, it is adviced that a high level assessment of the transport phase is performed in order to allow for the necessary continegencies. It must be noted that these operations are normally weather restricted operations in which metocean loads can be adjusted to platform capacities.

A typical weather restriction that shall find enough installation window is indicated in the following table.

Parameter	Limit
Hs	2.0 m
Wind Speed (10 min at 10 m)	12.0 m/s

Table 18.2-1 – Weather restriction for T&I

Different wave lengths shall be analysed based on each site provided scatter diagrams. This criteria may be revised in the T&I dedicated tasks (2.2.5, 3.4 and 4.5) of the COREWIND project.

WTG manufacturer may fix some limits for the transport phase. Based on previous experiences the following limits are proposed:

MOTIONS CRITERIA DURING TRANSPORT	
Maximum nacelle acceleration	0.6 m/s^2 (0.06g)
Maximum pitch / roll angle	[-2º, +2º]

Table 18.2-2 – Motions criteria during T&I

18.2.1 WindCrete Erection Process

The erection process of the WindCrete is one of the design drivers for the concept. This process rotates the whole structure from an horizontal to a vertical position. Then, at the beginning of the process, the tower acts like an horizontal canteliver beam that can produce large bending moments at the tower base. Also, restraining forces may be applied to avoid the structure overturning freely and allowing to control the process (section 3.1.2).

18.2.2 Transportation routes

A study on the routes for transport, ports of shelter shall be performed given the impact that they might have on the averall project costs.

19 DESIGN LIFE

Activefloat design life has been set in coherence with current state of art of the turbines and references in Ref.[S8]. If some components want to be designed for higher lives the metocean extreme data in this document shall be revised appropriately.

19.1 Windcrete

WindCrete design life is 60 years. This design life affects all permanent elements and mooring connectors fixed to the concrete hull, as well as the splash zone for protection and corrosion allowance.

19.2 Active float

The required design life for the platform and the wind turbine is 27 years, including 25 years of operation, 0.5 years of installation, and 1.5 years of decommissioning.

20 DESIGN CLASS

DNVGL-ST-0119 safety philosophy is based on the consequence class methodology. Providing the failure consequences a different safety level is assigned to the design.

FOWT are unmanned during severe environmental loading conditions and therefore they can be considered and designed to consequence class 1. However, it is stated in Ref. [S1] that in order to design the station keeping system to consequence class 1, redundancy shall be provided.

Station keeping redundancy is provided if the failure of one line does not cause instability of the platform or if the damaged station keeping system can withstand the 1-year loads in conjunction with load factors of ALS.

Concluding, the design classes to be considered are:

- Structural Design: Class 1
- Station keeping: Class 1 if redundant // Class 2 if non redundant

21 DESIGN LOAD CASES

DNVGL-ST-0437 states a full load case table, equivalent to the indicated in the IEC 61400-3-1. Recently issued standards, DNVGL-ST-0119 and IEC 61400-3-2 have added specific load cases for FOWT.

There are specific tasks within the COREWIND project destined to identify critical load cases that serve to made a reduce load case matrix that allows to assess the mooring and dynamic cable to the level required by the project.

22 MOTIONS AND LOAD ANALYSES

Motions and load analyses for FOWT design shall be calculated through coupled analyses and verified against tank tests. The COREWIND project is divided into different tasks that require several accurate levels of modelling for achieving the respective goals.

References [S5] and [OP2] gather recommended practices to perform coupled analysis of FOWT.

23 ACCEPTANCE CRITERIA

Motion constraints are mainly imposed by wind turbine and power cable design limitations. WTG limits are related to tilt angles and accelerations, and power cable limits are mainly related to horizontal excursions.

The following sections indicate a preliminary set of limits that may need revision in future phases of design.

23.1 NATURAL PERIODS CRITERIA

One of the main advantages of spar systems is that their periods in heave and pitch/roll can be positioned well far from the waves periods. Especial attention shall be kept to the heave motion and the possibility of VIM phenomenon.

In the case of ACTIVEFLFOAT, as in most semi-submersible configurations, natural periods are condemned to be within the range of wave periods.

However, semi-submersibles usually have much higher damping possibilities in heave, by installing heave plates and, at the same time, the distance between heave plates also rises the pitch/roll damping, that way limiting the effect of resonance. In any case, the designer may want to at least be out of the most typical wave periods (including swell periods).

23.2 MOTIONS VALID RANGES

23.2.1 Angular Motions

Angular motions restrictions are applied to mean values and/or extreme values. Tilt angles limitations come from two different sources: the WTG OEM, which needs a certain range to ensure power production, and the platform’s SCADA, which has its own range of operation. Based on previous experiences, the following limitations are proposed.

OPERATION	
DoF / Limit typology	**Limit**
Yaw (10 min. max)	<15º
Yaw (10 min. std)	<3º

Pitch / roll (max.)	[-5º, +5º]
Pitch (10 min. average)	[-2º, +2º]
Roll (max.)	[-2º, +2º]
Pitch (10 min. std)	<1º
Roll (10 min. std)	<0.4º
IDLING CONDITION	
Pitch (10 min. average)	[-5º, +5º]
Pitch (10 min. max)	[-7º, +7º]
EMERGENCY STOP	
Max. pitch	[-15º, +15º]

Table 23.2-1 – Motions criteria during Operation and Idling

23.2.2 Excursions

Excursions are usually restricted by the power cable or the windfarm layout. The maximum excursions limits of the platform are the subject of several tasks within the COREWIND project, therefore, the following are indicative limits for excursions limits.

Excursion limit for depths of 100 m

The maximum allowed excursion during idling conditions is 30 m in each direction. Before that, an alarm is generated when 15 m are reached, which is the limit for operation conditions. If reaching 30 m, the turbine is stopped.

EXCURSION RESTRICTIONS	
DoF / Limit typology	**Limit**
Horizontal offset (alarm limit) (mean during operation)	15 m
Horizontal offset (WTG shutdown). Maximum during parked conditions	30 m

Table 23.2-2 – Excursions Limits

Excursion limit for depths of 200 m

[HOLD]

Excursion limit for depths of 870 m

[HOLD]

23.2.3 Heave

No limitation to the heave DoF is provided for this project.

23.3 ACCELERATIONS VALID RANGES

Accelerations are restricted by the turbine manufacturer. Based on previous experiences the following limitations are proposed.

ACCELERATIONS LIMITS	
Operation (acc. XY / acc. Z)	1.85 m/s^2 (0.18 g)
Survival (acc. XY / acc. Z)	2.94 m/s^2 (0.3g)

Table 23.3-1 – Accelerations Criteria

23.4 FLOATING STABILITY

Floating stability implies a stable equilibrium and reflects a total integrity against downflooding and capsizing.

According to IEC 61400-3-2, the floating behavior shall be consistent with the requirements in all conditions including intact and damaged configurations, for both temporary and in-service conditions.

23.4.1 Intact Stability

Stability requirements established by recognized standards shall be fulfilled by both floater designs using quasi-static effects of turbine operations conditions and any extreme design load conditions. Methodology proposed in this project is found in DNVGL-ST-0119:

For Semi-submersible platforms:

- The area under the righting moment curve to the second intercept or downflooding angle, whichever is less, shall be equal to or greater than 130% of the area under the wind heeling moment curve to the same limiting angle.
- The righting moment curve shall be positive over the entire range of angles from upright to the second intercept.

For Spar platforms:

- The metacentric heigh GM shall be equal to or greater than 1.0 m. The metacentric height GM is defined as the difference between the vertical level of the metacentre and the vertical level of the centre of gravity and shall be calculated on the basis of the maximum vertical center of gravity VCG.

The above may be used in early stages of design. For a detailed design the dynamic-response based intact stability as recommended in Ref.[S2] is to be considered.

23.4.2 Damaged Stability

Damaged stability shall not considered as a requirement for the present project based on the design class defined in section 20.

24 FLOATER STRUCTURAL DESIGN

The COREWIND project is not focused on the structural design of the floaters. A sufficient definition based on preliminary analyses shall be done in order to perfom the upscaled floaters design to the 15 MW wind turbine.

24.1 DESIGN PHILOSOPHY

The design philosophy for structural design shall be the LRFD as established in the relevant standards.

24.2 DURABILITY

Floaters are mainly made of concrete and structural steel. Concrete durability shall be ensured by defining correctly the exposure classes of the different members. The exposure classes to be considered in the structural and durability calculations are obtained according to EN 1992-1-1 Table 4.1 and presented in the following paragraphs.

2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
4 Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
6. Chemical attack		
XA1	Slightly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA2	Moderately aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA3	Highly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water

Table 24.2-1. Exposure clases to be considered in the design of the substructure

Concrete rebars and prestressing tendons are adequately protected by the concrete itself, provided there is an adequate concrete coverage and the type/quality of the concrete is suitable. The latter will be established by the exposure class definition.

Structural members made of steel that are not effectively embedded in concrete may required corrosion protection.

Structural members made of steel that are above the sea level, shall be protected by appropriate coating.

24.3 MATERIAL PROPERTIES

This section gathers a list of the potential materials to be used in the floaters design. Final design analyses may imply variation from the material listed below.

The reference codes to define the materials characteristics are the Eurocodes, or the corresponding ETAs. The main structural materials used in the substructure and their main characteristics and design parameters are shown in the following groups.

24.3.1 Concrete

Concrete is defined according to EN 1992 Table 3.1:

- **Concrete C60**:

Characteristic cylinder compressive strength at 28 days	f_{ck} = 60 MPa
Modulus of elasticity	E_C = 39000 MPa
Strain at maximum strength in parabola-rectangle diagram	ε_{C0} = 2,3‰
Ultimate strain in the parabola-rectangle diagram	ε_{CU} = 2.9 ‰

- **Concrete C70**:

Characteristic cylinder compressive strength at 28 days	f_{ck} = 70 MPa
Modulus of elasticity	E_C = 41000 MPa
Strain at maximum strength in parabola-rectangle diagram	ε_{C0} = 2,4‰
Ultimate strain in the parabola-rectangle diagram	ε_{CU} = 2.7‰

24.3.2 Reinforcing Steel

Reinforcing steel is defined according to EN 1992 Section 3.2:

- **Reinforcing steel B500 SD:**

Characteristic yield strength	f_{YK} = 500 MPa
Modulus of elasticity	E_S = 200000 MPa
Ultimate strain	ε_{UD} = 14%

24.3.3 Prestressing Strands

"EN 1992 – Eurocode 2" and "prEN 10138-3" will be used as reference for the prestressing strands in absence of a specific ETA or another equivalent certification.

- **Prestressing cables Y 1860 S7 15.2 A:** (EC-2 art. 3.3 and prEN 10138-3:2000)

Characteristic tensile strength	f_{PK} = 1860 MPa
Characteristic yield strength	$f_{P0,1K}$ = 1600 MPa
Modulus of elasticity	E_S = 195000 MPa
Net area per strand	A_{NET} = 140 mm^2

Ultimate tensile force	F_{PK} = 260 kN
Maximum prestressing load	0,8·F_{PK} = 208 kN
Characteristic strain at maximum load	ε_{UK} = 3,5%

24.3.4 Prestressing Bolts

Prestressing bars will be design based on their corresponding ETA, while prestressing bolts will be designed base on "EN 1993 – Eurocode 3".

- **Prestressing bolts:** (10.9, EN 14399-4)

Tensile strength	R_m = 1000 MPa
Strength at 0.2% elongation	R_{el} = 900 MPa
Modulus of elasticity	E_{SP} = 205000 MPa

24.3.5 Structural Steel

Structural steel present is defined according to "EN-1993 Design of Steel Structures".

- **Structural steel S355:**

Characteristic yield strength	f_Y = 355 MPa (for thk < 40 mm)
	f_Y = 335 MPa (for 40< thk < 80mm)
Ultimate strength	f_u = 510 MPa (for thk < 40 mm)
	f_Y = 470 MPa (for 40< thk < 80mm)
Modulus of elasticity	E = 210000 MPa
Shear modulus	G = 81000 MPa
Yield Strain	$\varepsilon_U \geq 15 \cdot \varepsilon_Y$
Ultimate strain	ε_U > 15%

24.4 STRUCTURAL VERIFICATIONS

24.4.1 Models

Tower

Tower is analyzed with in-house spreadsheets or adequate structural software.

Tower model inputs are:

- Geometry that shall be based on drawings
- Steel reinforcement: to be defined in the in-house software and reflected in drawings
- Prestressed steel: to be defined in the in-house software and reflected in drawings

- Loads: The loads at few sections will be obtained from the coupled models and extrapolation would be done in order to check other sections. Linear extrapolation will be done.
- Stresses generated by thermal actions (DNVGL-ST-0126 section 5.4.2) will be added to the structure in all load cases.
- Materials properties will be obtained as described in section 24.3.

Tower model outputs:

- Verification ratios of ULS, SLS and FLS.

Floater

Calculation of floaters or tanks of both technologies shall be approach in two different ways:

- Direct sectional control with loads obtained in coupled models based on procedures in EN-1992 and EN-1993. If this is the case, additional models need to be developed for verify the resistance to local bending of walls.
- Model the tanks with plate elements. If loads at control sections are provided sub-models can be prepared from portions of the structure. The loads will be input at sections by means of kinematic couplings and defining the boundary conditions at other control section where reactions are monitored in order to match them to the derived from coupled analysis.

FE models are to be developed in an appropriate structural software. Meshing will be done in following basis:

- Element max. size: <1.0 m
- Element aspect ratios: < 5.0

For both calculation methodologies the inputs required are:

- Geometry that shall be based on drawings
- Reinforcement steel and prestressing steel
- Loads at control section and motions. Motions will be transformed in to water pressures on walls of the structure based on recommendations in section 4.9.3 in DNVGL-ST-0119.

Model outputs (FE models):

- Internal forces at each plate element from linear solver.

The internal forces require a post-processing performed with in-house spreadsheets in order to verify the structural integrity. All models shall be defined in detailed in the structural reports.

24.4.2 Ultimate Limit State

The structural verifications will be mainly based on controlling the Ultimate Limit State. All concrete and structural sections will be calculated in accordance with EN-1992 and EN-1993 codes, respectively.

Structural members where the EN-1993 methods are out of the range of application due to the large slenderness, like the steel tower, the use of the DNVGL-RP-C202 is indicated, for ensuring the structural integrity of the member.

The combinations for the ULS verification will be constructed as indicated in the following table.

ULS	Load Categories			
	G	Q	E	P
ULS a	1.25	1.25	0.7	0.90 / 1.10
ULS b	1	1	1.35	0.90 / 1.10
ULS c (Abnormal DLC's)	1.1	1.1	1.1	0.90 / 1.10

Table 24.4-1 – ULS load combinations

For ULS verifications the material factors are shown in the following table:

ULS	γ_C for concrete	γ_S for reinforcing steel	γ_S for prestressing steel	γ_S for structural steel
Material Factor	1.5	1.15	1.15	1.1

Table 24.4-2- ULS Material factors

24.4.3 Service Limit State

For concrete structures, SLS verifications required to define the LDD 10^{-2}, or quasipermanent load. This load is obtained by finding the 99% percentile of the DLC 1.2 loads / motions series.

In addition, in order to control the maximum stress requirements stated in DNVGL-ST-0126, the characteristic combination is needed. These combinations are obtained finding the maximum values from the unfactored normal DLC's.

Both for steel and concrete structures, all load and material factors are set to 1.0 for SLS verifications.

Exposure classes

- Tower and other members in tidal/splash zone: XC2 + XS3
- Tanks and other members permanently under water: XC1 + XS2

Stress limitations

For reinforced concrete and prestressed concrete, the concrete compressive stresses for the characteristic extreme load shall be limited to 0.6 f_{ck}. In addition, concrete compressive stresses under permanent loads shall be limited to 0.45 f_{ck}.

The reinforcement stress shall be limited to 0.9 f_{yk}.

Crack Control

DNVGL-ST-0126 supersedes some requirements of EN-1992. All prestressing is provided with unbonded tendons. Attending to the exposure classes above the crack width maximum opening allowed is:

Area	Prestressed	Max. Crack width (mm)
Tower and member in splash zone	YES	0.2
	NO	0.2

Tanks and other members permanently under water	YES	0.2
	NO	0.3

Table 24.4-3 – Crack width requirement

Note that if bonded prestressing systems are to be used crack limitation requirements may change from Table 24.4-3.

Tightness against leakages of fluids

According to EC2 Part 3 section 7.3.1, in order to provide Tightness Class 3, it is required to provide a minimum of 50 mm of compressed concrete in each section for the quasi-permanent combination of actions

24.4.4 Fatigue Limit State

COREWIND project is not focused on the structural design of the floaters, therefore no global fatigue assessment is to be done during the project.

Task 2.2.4 of the project implies local design of fairleads and support area. The design report to be produced in that task shall detail the methodology used for the fatigue assessment that shall be done based on the following standards:

- For concrete structural members: Model Code 2010
- For steel structural standards: DNVGL-RP-C203

24.4.5 Accidental Limit State

No accidental loads are in the scope of the COREWIND project for the global structural assessment.

25 MOORING SYSTEM DESIGN

This section provides requirements for mooring lines system design.

The platform will be designed for the wind farm operating life as indicated in section 19 and their mooring systems must be designed to be in operation for the duration of the wind farm design life without replacement due to strength, fatigue, corrosion and abrasion.

25.1 LIMIT STATES

The mooring line must be designed for the following limit states: ULS, FLS, ALS. The load factors as a function of safety class are listed in DNVGL-OS-E301 Chapter 2, section 2, subsection 4.2 and 4.3.

25.2 DESIGN CONDITIONS

Operating, Survival and Accidental design conditions are the most relevant situation to take into account to carry out the mooring lines design.

The accuracy level required is Level I and therefore a dynamic model is required. The model shall reproduce the real dynamics of the mooring lines. The buoyancy and the drag of the lines shall be included.

25.3 LOAD FACTORS

Requirements for load factors in the ULS and the ALS are given in the next table as a function of safety class as reflected in DNVGL-ST-0119 section 8.2.2.6.

Load factor requirements for design of mooring lines			
Limit state	Load factor	Consequence Class	
		1	2
ULS	γ_{mean}	1.30	1.50
ULS	γ_{dyn}	1.75	2.20
ALS	γ_{mean}	1.00	1.00
ALS	γ_{dyn}	1.10	1.25

Table 25.3-1 – Load factor requirement for design of mooring lines

25.4 DESIGN CRITERIA FOR ULS AND ALS

The design criterion in the ULS is:

$$T_d < S_c$$

The design criterion in the ALS is:

$$T_d < S_c^*$$

For ALS purposes T_d is established under an assumption of damaged mooring system in terms of one broken mooring line.

When statistics of the breaking strength of a component are not available, then the characteristic capacity of the body of the mooring line may be obtained from the minimum breaking strength S_{mbs} of new components as:

$$S_c < 0.95 \cdot S_{mbs}$$

The design tension T_d in a mooring line is the sum of two factored characteristic tension components $T_{c,mean}$ and $T_{c,dyn}$,

$$T_d = \gamma_{mean} \cdot T_{c,mean} + \gamma_{dyn} \cdot T_{c,dyn}$$

Where:

- $T_{c,mean}$: characteristic mean tension
- $T_{c,dyn}$: characteristic dynamic tension

25.5 DESIGN CRITERION FOR FLS

Mooring lines shall be designed against fatigue failure. The design cumulative fatigue damage is:

$$D_D = DFF \cdot D_C$$

Where:

- D_D: design cumulative fatigue damage.
- DFF: design fatigue factor.
- D_C: characteristic cumulative fatigue damage caused by the stress history in the mooring line over the design life.

Requirements for the design fatigue factor DFF are given in DNVGL-ST-0119 section 8.2.5.1, which provides the following table:

Consequence class	DFF
1	5
2	10

Table 25.5-1 – DFF for mooring chain

Predictions of fatigue life may be based on calculations of cumulative fatigue damage under the assumption of linearly cumulative damage. The characteristic stress range history to be used for this purpose can be based on rain-flow counting of stress cycles.

When Miner's sum is used for prediction of linearly cumulative damage, the characteristic cumulative fatigue damage D_C is calculated as:

$$D_c = \sum_{i=1}^{I} \frac{n_{C,i}}{N_{C,i}}$$

in which:

- I = number of stress range blocks in a sufficiently fine, chosen discretization of the stress range axis.
- $n_{C,i}$ = number of stress cycles in the ith stress block, interpreted from the characteristic long-term distribution of stress ranges, e.g. obtained by rain-flow counting.
- $N_{C,i}$ = number of cycles to failure at the stress range $\Delta\sigma i$ of the i^{th} stress block, interpreted from the characteristic S-N curve.

25.6 CORROSION ALLOWANCES

DNVGL-ST-0119 section 13.1.3 defines the requirements for corrosion allowance for chains (Table 13-1):

Part of mooring line	*Corrosion allowance to be added to chain diameter (mm/year)*		
	Regular inspection [1)]	*Requirements for the Norwegian continental shelf*	*Requirements for tropical waters*
Splash zone	0.4	0.8[2)]	1.0
Catenary [3)]	0.3	0.2	0.3
Bottom [4)]	0.4	0.2	0.4

1) The regular inspection is carried out by ROV in accordance with DNVGL-RU-OU-0102 or in accordance with operator's own inspection program, approved by national authorities if necessary.
2) The increased corrosion allowance in the splash zone is required by NORSOK M-001.
3) Suspended length of mooring line below the splash zone and always above the touchdown point.
4) The corrosion allowance specified in the table is provided as guidance. Significantly larger corrosion allowance than the minimum values recommended in the table should be considered if microbiologically induced corrosion (MIC) can be expected.

Table 25.6-1 – Corrosion allowance for mooring lines

Impact of corrosion on weight per length and restoring forces will be evaluated. Several corrosion sceanrios might be evaluated on this basis.

25.7 MOORING LINES DRAG AND ADDED MASS COEFFICIENTS

As per DNVGL-OS-E301 Chapter 2, section 2, subsection 2.7, the following drag coefficients and added mass coefficients must be adopted for simulation purposes unless other information is available:

Mooring component	*Transverse*	*Longitudinal*
Stud chain	2.6	1.4
Stud less chain	2.4	1.15
Stranded rope	1.8	*
Spiral rope without plastic sheathing	1.6	*
Spiral rope with plastic sheathing	1.2	*
Fibre rope	1.6	*

(*) Longitudinal forces may often be neglected in these cases or the expressions from DNV-RP-C205 may be applied.

Table 25.7-1 – Drag and added mass coefficients for mooring lines

26 DYNAMIC CABLE SYSTEM

This section provides requirements for Power Transmission Cable design.

The cable system comprised of cables, buoyancy ancillaries to support a heave compensating catenary, cable bend protection ancillaries and connection hardware with appropriate cathodic protection mitigation.

The platform will be designed for the wind farm operating life (as stated in section 19) in operation and the cabling system must be designed to be in operation for the duration of the wind farm design life without replacement due to strength, fatigue, corrosion and abrasion. Minimal maintenance work is desirable in such a system to drive down costs of operation.

26.1 DYNAMIC CABLE MODEL

Orcaflex will be used to perform dynamic cable system analyses. Orcaflex is a 3D, non-linear; time domain finite element analysis program developed by Orcina consulting and is widely used for the analysis of dynamic umbilical and cable systems worldwide.

26.2 CABLE INTERFACE WITH THE FLOATING STRUCTURE

The entrance to the platform and subsequent routing to a tensile load termination position is a key consideration.

To reduce the motion induced into the cable due to platform movement, the ideal entrance location would be at the floating structures centre of gravity. Due to the limited movement seen at this point the subsequent fatiguing of the cable would be low and thus costs could be reduced by reducing the dynamic cable design. As installation costs are proportional to the time required for installation, in practice ease of access for installation purposes often drives the entrance points into the structure closer to the outer edge. The location of the entrance point shall be determined with this in mind to minimise costs over the floating system. It is from this point on each of the structures the motion data will be collected for cable analysis. In general it is best practice for the exit point to be below the splash zone to reduce loading on the cable and prolong its life.

The exit angle within the cable length is project specific. Often in these types of design, the cable will enter vertically which is ideal to avoid imposing unnecessary loading on the cable and structure, however exit angles may be modified to minimise risks of clashing with mooring lines, etc. For this project, the initial approach will be to assume the entrance point is vertical.

In static cable scenarios, the cable would be pulled through the internal structure to the hang off point. The routing of the cable is often controlled through a J-tube or I tube. There is significant distance between the entrance to the structure and the hang off. At the hang off the cable axial load is transmitted into the supporting steel. The section of cable held in the J-tube is not dynamic and so will not be modelled. Instead the dynamic analysis model starts at the tube exit.

In floating structures, it is likely there will also be a significant elevation between the hang off position and the exit of the structure, where the path is controlled in the same manner. As such, within this project the dynamic cable section of the system under evaluation is considered for the cable length from the exit of the structure down to the touchdown point on the seabed. The final arrangement for this system will be defined later in detail phases.

26.3 FLOATER MOTION DATA INPUT

Depending on the strength and direction of the prevailing weather, the floating structure will drift to an offset position. On completion of the preliminary floating structure analysis the maximum offset of the cable exit point during the accidental limit state (ALS) will be confirmed, therefore initially this will be considered as the baseline for conservatism.

The motion (response amplitude operator) data at the cable exit location of the floating structure, attached to the mooring system guides, will be provided to for dynamic cable analysis in the form of time trace data in an excel file. This data will be converted into standard text file input form for Orcaflex model input.

It is worth noting the connection of a cable into the system has a negligible effect on the structure in comparison to mooring lines.

Due consideration will also be undertaken in case of the extremely unlikely event of a mooring line failure.

26.4 LOADING FROM THE ENVIRONMENT

Extreme will be performed using the Dean Stream wave spectra (Regular wave approach). Directional wave parameters for the 10 year and 50 year return periods provided. Initial approach is to consider the most onerous directional wave parameters for both 10 and 50 year return periods as this is the most conservative.

26.5 DYNAMIC CONFIGURATION ANALYSIS WITHIN ORCAFLEX

The design and configuration of the dynamic cable system is sometimes given low priority however the failure of a cable can have significant consequence. Ensuring that the cable system is configured in an optimal way can minimise project risk and reduce CAPEX spend across the project.

The aim of the cable configuration optimization studies should be to reduce system loads. This in term will reduce the need for additional cable armoring and reduce ancillary hardware costs associated with items such as subsea anchors, tethers, buoyancy or bend stiffeners. In addition the cable should be configured in a way so that it does not present a clashing risk to other assets or limit allowable floating structure offsets creating the need for more expensive tighter mooring spreads.

A diagram of the cable system for modelling is shown below. Beyond the touchdown point will be considered to be static.

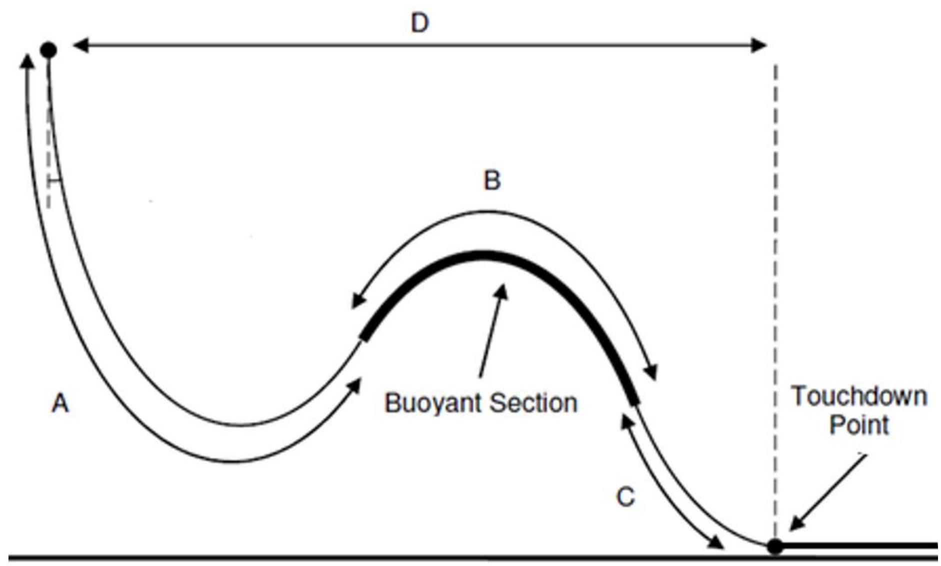

Figure 26.5-1 – Dynamic Cable Section

The exit point of the structure will be positioned in the Orcaflex model so that its centre at Orcaflex global X, Y, Z co-ordinate 0, 0, 0. In addition to the extreme near far and transverse offsets considered, all current directions are to be considered in the analysis. This ensures the most conservative offset/directional parameters are captured in the study.

At this exit position, the product typically will require protection to prevent overbending and lower local bending stress in the cable components. This protection is often in the form of a bend stiffner. Requirements for this protection device will be assess as part of the modelling requirement.

The application of a sufficient quantity of buoyancy modules to the cable is necessary to relieve topside tension as well as to decouple the touch-down point from the floating structure heave motion. The buoyancy upthrust requirement, considering both start of life and end of life condition, and local module positioning in section B of the diagram will be informed by the analysis.

Cable diameter, submerged weight and stiffness properties will inform the model. The cable should be designed to ensure maximum axial tension from the system is lower than the safe working loading of the cable. This will be verified as part of the analysis.

26.6 CORROSION

As per DNVGL-ST-0119 the sheathing of the cable in dynamic application is chosen to ensure that it has sufficient resistance to corrosion considering the service environment: exposure to sea water and temperature ranges. For the application no penetration of the sheath due to corrosion is allowable during service.

26.7 HYDRODYNAMIC DRAG COEFFICIENTS

Standard drag and added mass coefficients are used for the analysis as provided in Table 26.7-1, these are based on guidelines provided in DNV-RP-C205. The coefficients are considered conservative as they are based on upper bound drag value recommendations.

Parameter	Value
Normal Drag	1.200
Normal Added Mass	1.000
Tangential Drag	0.008
Tangential Added Mass	0.000

Table 26.7-1 – Drag Coefficients considered in cable analysis

26.8 STATIC CONFIGURATION CHECKS

Prior to commencing the detailed analysis checks on the static configuration, checks will be performed to verify that:

- Maximum safe working load and minimum bend radius are not compromised at any point along the cable route when the system is placed in nominal position.
- Maximum safe working load and minimum bend radius are not compromised at any point along the cable route when extreme wind turbine offsets are applied.
- The proposed deployment configuration is appropriate.
- The exit angle is suitable and does not place any unnecessary mean loading on the bend stiffeners that could lead to creep.
- The catenary lengths offer the optimum starting shape and allow for maximum flexibility of the system under various operating conditions.

- The buoyancy provides the correct up lift to give the cable its required static configuration while providing adequate mean sea level clearance at start of life (avoiding the splash zone) and yet still ensures enough seabed clearance at end of life.
- Touch-down point (TDP) will be checked to ensure it has been set at the correct location.

It is critically important that both start and end of life conditions are considered as the system needs to balance the sometimes contrasting requirements of both cases.

26.9 EXTREME EVENT (ULTIMATE LIMIT STATES) ANALYSIS

Extreme event analysis is typically carried out using regular wave time domain analysis. For this water depth wave loading is applied using Dean Stream wave theory. However, this approach may be modified later to consider either irregular and/or directional data for the most critical analysis cases identified.

Cable tension should remain below maximum safe working load (SWL) and cable bending should remain above minimum bend radius.

A suitably onerous load case matrix should be considered that encompasses all important variables and combinations thereof, typically including: extremes of wave and current loading, positional offset of exit point in the floating strucutre, as well as start and end of life conditions. The load case matrix to be used initially for the extreme analysis is presented below. The return period of 50 years will be used for operation condition analysis as per DNV-OS-J103.

Variable	Name	Variable Name	Total
1	Condition	Start of Life	2
		End of Life	
2	Platform Offset	As needed for cable modeling based on RAO data	TBC
3	Wave Period	50 Yr Wave (THmaxHigh) / 10 Yr Current	4
		50 Yr Wave (THmaxLow) / 10 Yr Current	
		50 Yr Current / 10 Yr Wave (THmaxHigh)	
		50 Yr Current / 10 Yr Wave (THmaxLow)	

Table 26.9-1 – Cable Analysis Load Case Matrix

Consideration of platform rotation will be undertaken where needed.

26.10 FATIGUE ANALYSIS

The fatigue analysis includes first order effects of floater motion and direct wave action on the cable and cable systems. Stress cycles are determined using simple counting method to give the un-factored fatigue life along the length of the cable. Stress concentration factors are determined using a proprietary calculation tool developed by JDR.

26.11 DESIGN CRITERION FOR FLS

The design cumulative fatigue damage assessment for cables is the same equation as that for mooring lines outlined in section 25.5 as:

$$D_D = DFF \cdot D_C$$

However the reinforcement component in a cable is armour wire where DNVGL-ST-0119 advises the DFF shall not be taken less than 10 unless otherwise agreed.

The Installation fatigue assessment shall determine the maximum floating structure hove to duration prior to prediction of fatigue failure.

The cable is cycled between its straight condition and minimum bend radius, and between a state of zero tension to its maximum allowable safe working load, repeatedly until the number of cycles to fatigue failure is identified. A wave period of 3 s is assumed between cycles, which is conservative.

The number of cycles to failure is calculated using Figure 26.11-1 below which is taken from DNV RP-C203.

$$\log N = \log \bar{a} - m \log \Delta\sigma \qquad (2.4.1)$$

N = predicted number of cycles to failure for stress range $\Delta\sigma$

$\Delta\sigma$ = stress range

m = negative inverse slope of S-N curve

$\log \bar{a}$ = intercept of log N-axis by S-N curve

Figure 26.11-1 – S-N curve definition

26.12 S-N CURVE DATA

The S-N curve for steel is taken from DNV-RP-C203. As fundamentally there is no reliance upon the strength of an individual weld in an armour strand upon the overall tensile capacity of the cable, it is appropriate to consider the armour wire in the cables to be classified as a non-welded part (B1 fatigue curve).

By the Eytelwein formula (Capstan equation), the reduction in tensile capacity in an individual armour wire due to the weld (as a result of the helical lay of the components, and in conjunction with the staggering of weld placement along the cable length) is negligible and can be disregarded in the context of the armour package system as a whole. The steel S-N curve to be used in the study is provided.

In 2004 JDR contracted Corus "Fatigue and Fracture Knowledge Group" to perform fatigue testing to create copper S-N data. In 2016 JDR performed further material testing at the University of Huddersfield, using conductor samples to augment the number of data points and increase reliability. Copper S-N curves developed by JDR will be used in this assessment. The copper S-N curve to be used in the study is provided.

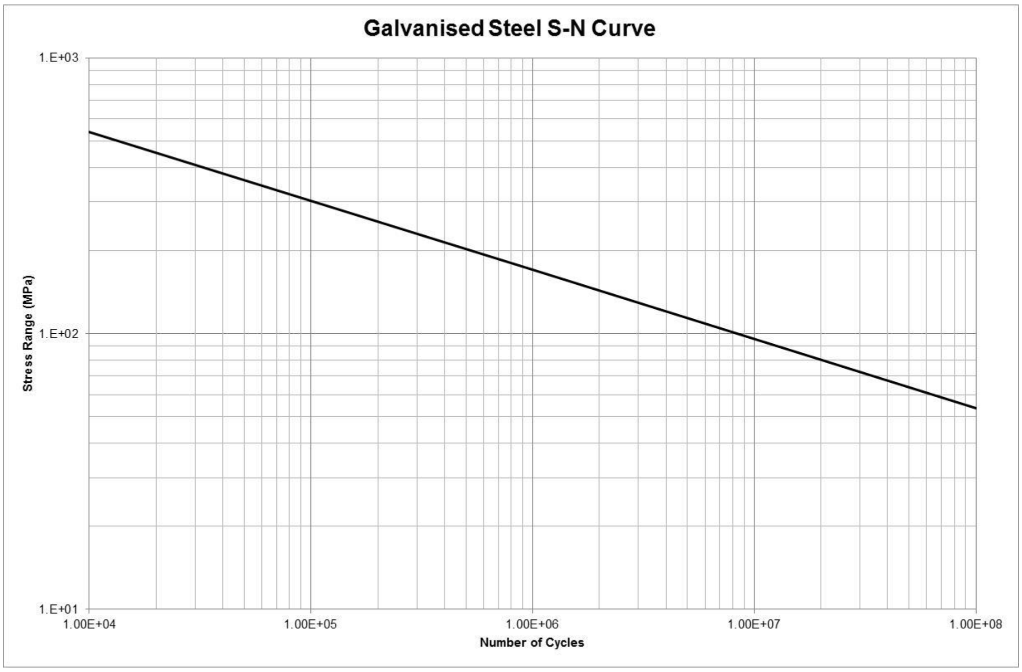

Figure 26.12-1. S-N curve Galvanised Steel

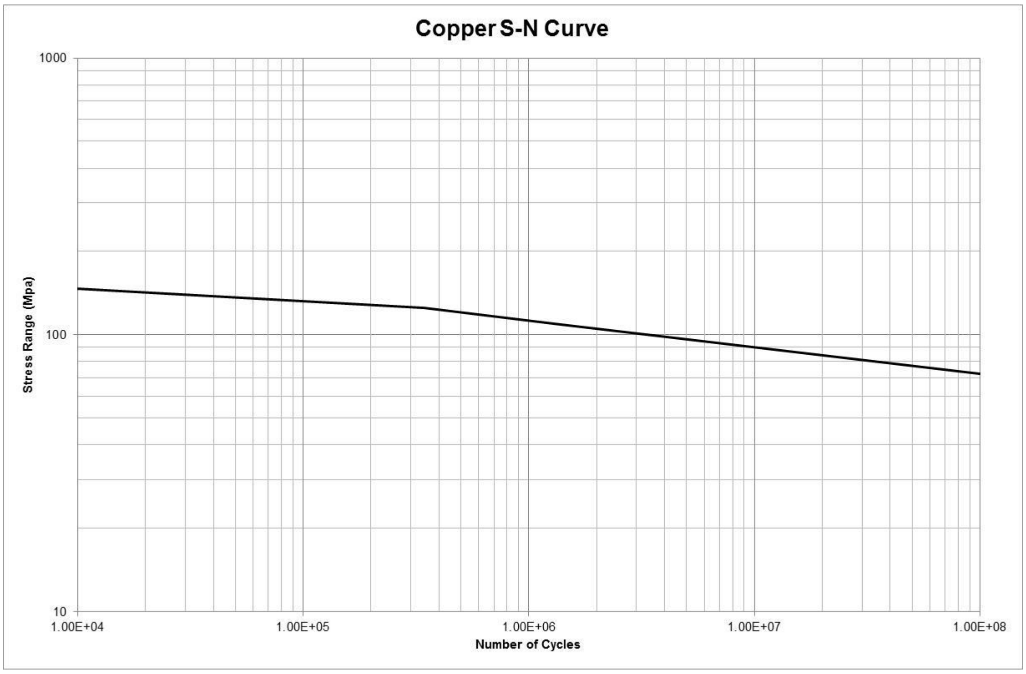

Figure 26.12-2. S-N Curve for Copper

26.13 INTERFERENCE ANALYSIS

If required interference analysis may be performed to ensure that there is no risk of clashing with mooring lines or floating wind device. In addition seabed surface and seabed clearances are reviewed to ensure adequate clearance for all cases considered.

27 REFERENCES

27.1 PROJECT DOCUMENTS

REF	Document	Document Title
D1.1	D1.1	Definition of the 15MW Reference Wind Turbine

27.2 OTHER PROJECT DOCUMENTS

REF	Document	Project	Document Title
OP1	D7.2	LIFES50+	Design Basis
OP2	D4.4	LIFES50+	Overview of the numerical models
OP3	D4.1	FLOTANT	Design Basis
OP4			Molins, C., Yagüe, A. & Trubat, P., 2018. Construction possibilities for monolithic concrete spar buoy serial production. Journal of Physics: Conference Series, 1104(1).
OP5			Campos, A. Molins, C., Gironella, X., Trubat, P., 2016. Spar concrete monolithic design for offshore wind turbines. Proceedings of the Institution of Civil Engineers - Maritime Engineering, 169(2), pp.49–63.