Base Load Power Hub (BLPH)

Structural Design Premises

Document Number: 123A90-DDL-ST-SP-90017

Document Class: 2

Rev.	Date	Description	Prepared	Reviewed	Approved
04	03/12/2024	Accepted for Construction (AFC)	F.Biancheri	A.Balzarotti	D.LoSterzo
03	20/06/2024	Accepted for Construction (AFC)	A.Balzarotti	A.Balzarotti	D.LoSterzo
02	26/02/2024	Accepted for Construction (AFC)	A.Castiello	A.Balzarotti	D.LoSterzo
01	21/11/2023	Issue for Approval (IFA)	A.Castiello	A.Balzarotti	D.LoSterzo
00	20/10/2023	Issue for Review (IFR)	A.Castiello	A.Balzarotti	D.LoSterzo

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Revision / Change Log

Rev.	Description
00	Issue for Review (IFR)
01	Issue for Approval (IFA)
02	Accepted for Construction (AFC)
03	Accepted for Construction (AFC)
04	Accepted for Construction (AFC)

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List of Attachments

Attachment A – Material selection according to DNV code DNV-ST-0145

Attachment B - 144671-VOOW-GEN-SUR-REP-0074 - FOUNDATION AS-BUILT SURVEY CW70 HNA6

Attachment C – Positioning Report Innovation at CW70 HNA6

Attachment D - 144671-VOOW-TF-ENG-DWG-0001 WTG coordinates and Cable Routing

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1 INTRODUCTION

1.1 PROJECT OVERVIEW

Crosswind, a Shell and Eneco joint venture, was awarded the tender to develop the offshore Hollandse Kust Noord (HKN) wind farm, located 18.5km off the coast of Netherlands near the town of Egmond aan Zee. The consortium plans to have HKN operational in 2023 with an installed capacity of 759MW generating at least 3.3TWh per year. This is sufficient renewable power to supply more than 1 million Dutch households with green electricity.

Offshore wind farms produce intermittent electricity and balancing these dips and peaks and integrating this electricity into the national energy system requires new technologies. For this reason, the offshore wind farm will include several new technologies that could be implemented at full-scale in the future. The new technologies being explored include a floating solar park, a short-duration battery storage as well as long-term storage via green hydrogen produced by water electrolysis and subsequently consumed in a fuel cell. The combination of these individual measures, herein referred to as the Base Load Power Hub (BLPH) will ensure a continuous power supply regardless of the wind situation.

1.1.1 Project Location

The Hollandse Kust Noord (HKN) wind farm is located 18.5km off the coast of the Netherlands near the town of Egmond aan Zee.

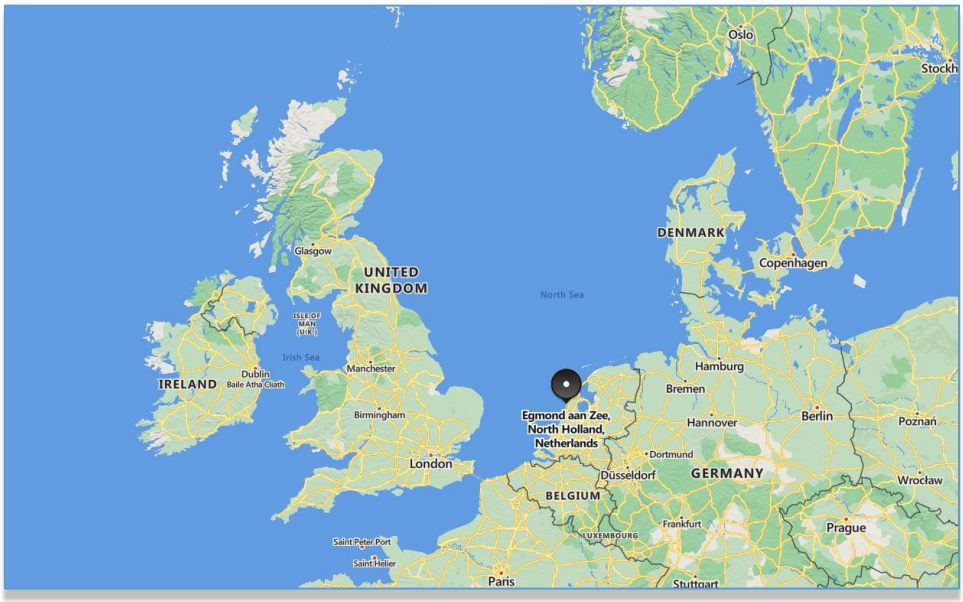


Figure 1.1 HOLLANDSE KUST NOORD WIND PARK Location Map

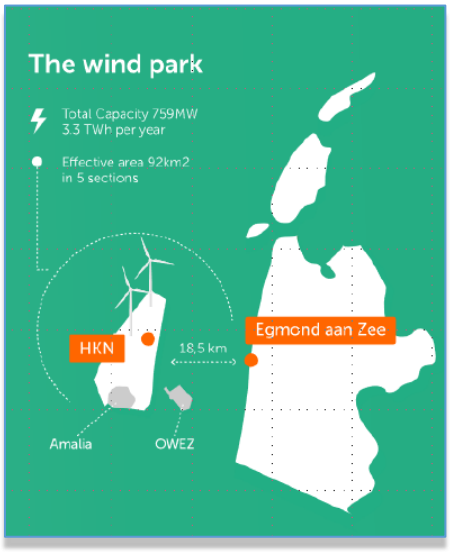


Figure 1.2 - Wind Park Details

The wind farm consist of No. 69 x 11.5MW wind turbine generators each to be installed on a dedicated monopile structure. A 70th monopile located on coordinates E 590719.1, N 5841602.3 is reserved for installing the platform accommodating the BLPH. The monopile structure was designed and installed by Van Oord Offshore Wind (VooW) and therefore not part of the BLPH scope.

1.1.2 Project Concept

All components included in the BLPH will be integrated onshore on a topsides and installed in an offshore location within the boundaries of the HKN wind farm. At its simplest, the BLPH will store energy during periods of high electricity generation from renewable sources (a wind turbine generator and the floating solar PV farm) and release it during periods of low generation. In practice. It is expected that the energy stored will supplement generation from renewable sources to ensure that at least 1.0MW of power is exported to the electrical grid regardless of wind/solar availability.

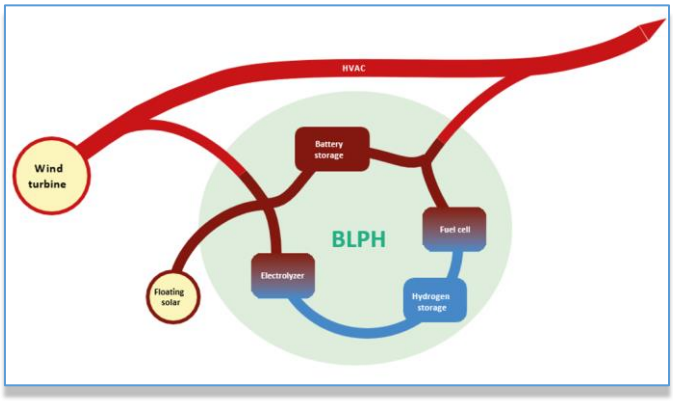


Figure 1.3 – BLPH Concept

Electricity in the BLPH will be stored in the form of electrons in a battery energy storage system (BESS) as well as converted into green hydrogen via water electrolysis. When the power output from the connecting wind turbine drops to below 1 MWe, the BLPH will return its stored energy to supplement the remaining power output. This will be by discharging the BESS, while stored hydrogen will be converted back into electricity using a fuel cell to extend the discharge duration.

The purpose of the BLPH s to function as a testbed for the integration of energy storage technologies with renewable generation. It is expected that equipment installed on the BLPH will operate as Normally Unmanned Installation for at least two (2) years with no or minimal maintenance and it will have its own independent control room with the ability to communicate with Crosswind's control room.

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2 DOCUMENT SCOPE

2.1 PURPOSE

The purpose of this document is to outline the approach and methods to be used for the detail engineering structural design of the Topside, Transition Piece (TP) and appurtenances of the BLPH platform. It incorporates the following:

- Key input data.
- Description of analyses to be undertaken.
- Codes and Standards employed.
- Interface information, including caisson support structure interface supports.
- Pre-service operations.

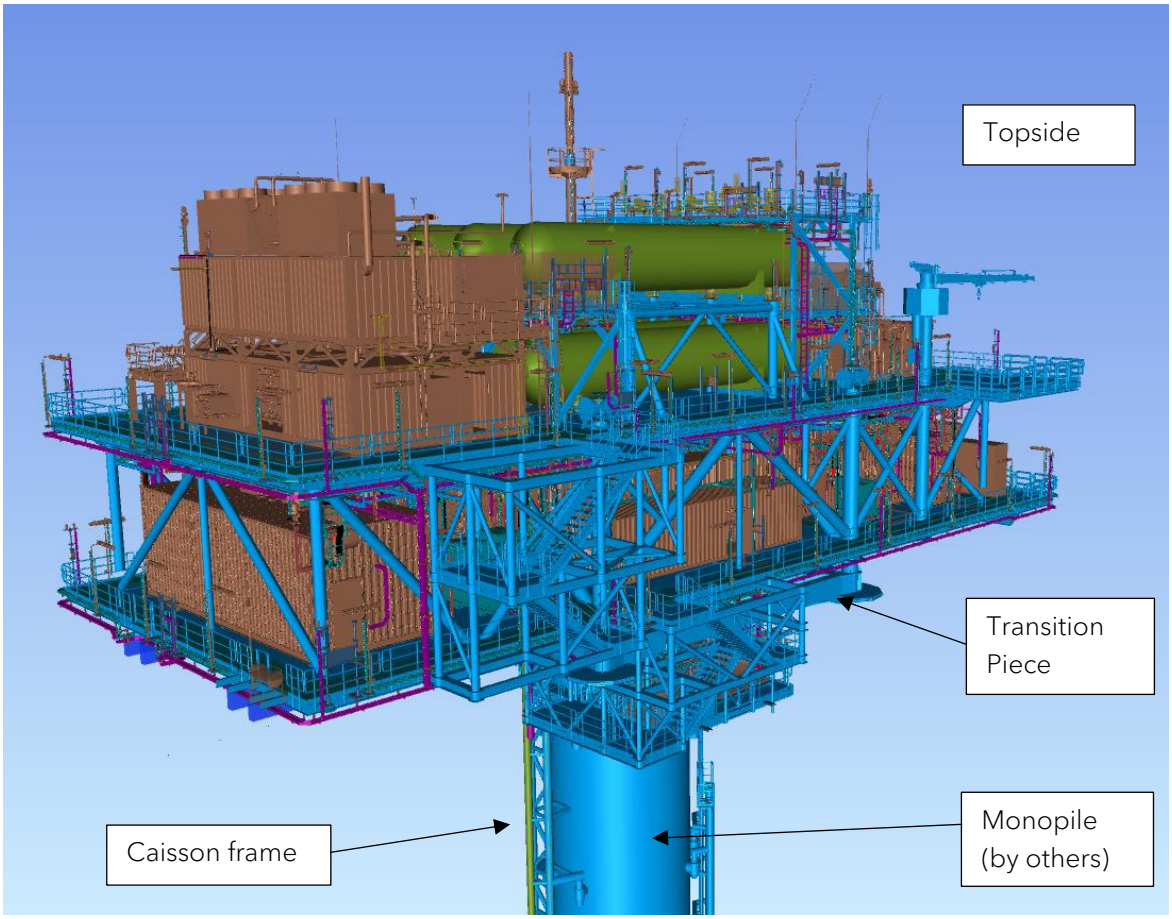


Figure 2.1 – BLPH Topside

2.2 MONOPILE

The monopile for the BLPH is referred to as CW70. The monopile has been installed in Jan 2023 and its verification is excluded from the BLPH scope. Both geotechnical and structural analyses are considered in Ref./50/.

Actions calculated during detail design of BLPH topside, will be compared with the design load of monopile at flange connection elevation (battery limit) obtained from the analyses performed in Ref./50/, to confirm the topside can be supported by existing Monopile.

A typical HKN monopile is shown in Figure 2.1 from FEED, Ref./40/.



Figure 2.2– HKN Monopile

2.3 TRANSITION PIECE DESIGN & ANALYSES

The monopile TP connects the topsides to the monopile. The following global analyses or assessments will be undertaken for the TP:

- In-Service analysis;
- Load-out Analysis;
- Transportation analysis;
- Fatigue Analysis in transportation conditions.

- Lifting Analysis;
- Grillage and sea-fastening design and checks.

The In-Service analysis of TP and Topside will be carried in one document which refer to an integrated structural model of both structures.

The following local checks will be undertaken for the TP:

- Connection between the monopile and TP
- Secondary structures and miscellanea.
- Joint checks (where not covered by global analyses). E.g. lift points, connection TP-topside

2.4 TOPSIDES DESIGN & ANALYSES

The following global analyses will be undertaken for the Topsides:

- In-Service Analysis.
- Loadout Analysis;
- Transportation Analysis;
- Fatigue Analysis in transportation conditions;
- Lifting Analysis;
- Grillage and sea-fastening design and checks.

Ship impact accelerations and blast loads will be considered in conjunction with normal operating loads on the topsides. Earthquakes are shown to be insignificant and are ignored Ref. /40/ and /42/.

The following local analysis and checks will be undertaken for the Topsides:

- Wind VIV verification checks for sensitive slender structural elements;
- Dropped objects verification checks;
- Secondary structures and miscellanea;
- Joint checks (where not covered by global analyses).

Wave run-up loads on the Topsides will not be considered. The cellar deck is plated so that water impingement (from below) on equipment is reduced to a practical minimum, refer to Section 6.11.

2.5 APPURTENANCES

The BLPH has the following appurtenances that are part of detail engineering structural scope:

- 2 x seawater lift caissons.
- 1 x brine water disposal caisson.

- Internal ladders
- Floating solar cable

2.5.1 Caissons frame

In-Service analysis and Vortex Induced Vibration (VIV) for the caissons (ignoring proximity of monopile) will be performed. Caissons are supported by a dedicated frame that will be supported on the monopile. The position of the frame has been pre-determined during Feed phase, see Figure 2.1.

The centreline of the frame is 155 degrees east of grid North. Pairs of stubs are provided on the monopile to support the frame at three levels: El.+14.500 m, El.+7.000 m and El.0.000 m LAT.

The seawater lift caissons contain submersible pumps, so the caissons must be vertical.

The caisson structural supports have been already welded to monopile and therefore they are already present offshore. All the supports will be surveyed offshore prior to proceed with caisson loadout.

As the caissons extend to the topside, additional supports are provided on the TP.

2.5.2 Infield Cable

Floating solar power cable entrance is foreseen to connect the floating solar installation. No J-tubes are foreseen, the cable is free hanging from the cable hang-off inside the monopile.

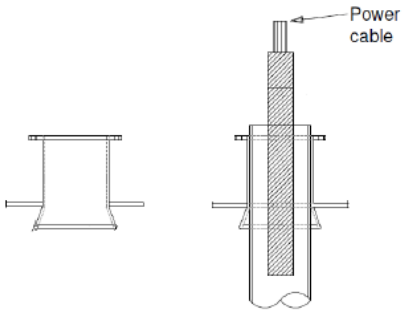


Figure 2.3 – Cable hang-off flange

2.6 STRUCTURAL WEIGHTS AND MATERIAL TAKE-OFF

Structural weights and Centre of Gravities (CoGs) will be provided for Weight Reporting and Material Take-off for Primary, Secondary and outfitting structures in the SoW.

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3 REFERENCES

3.1 CODES AND STANDARDS

Doc. No.	Description
/1/ BS EN ISO 19900	General requirements for offshore structures - 2019
/2/ BS EN ISO 19901-1	Specific requirements for offshore structures - Part 1: Metocean design and operating considerations - 2015
/3/ BS EN ISO 19901-2	Seismic design procedures and criteria
/4/ BS EN ISO 19901-3	Specific requirements for offshore structures – Part 3: Topsides structure - 2014
/5/ BS EN ISO 19901-5	Specific requirements for offshore structures – Part 5: Weight Management - 2021
/6/ BS EN ISO 19901-6	Specific requirements for offshore structures – Part 6: Marine Operations – 2009
/7/ BS EN ISO 19902	Petroleum and natural gas industries - Fixed steel offshore structures – 2020
/8/ ISO 19906:2019	Petroleum and natural gas industries – Arctic offshore structures
/9/ EN 1991-1-3:2003 & A1:2015	EN, Eurocode 1 – Actions on Structures – Part 1-3: General actions – Snow loads - Incorporating corrigenda December 2004 and March 2009, December 2015.
/10/ EN 1993-1-1	Design of steel structures – Part 1-1: General rules and rules for buildings
/11/ API RP 2A-WSD	Recommended practice for planning, designing and constructing fixed offshore platforms – Working stress design – 22nd edition – 2014
/12/ DNV-ST-N001	DNV - Marine operation and marine warranty - 2021
/13/ DNV-ST-0145	DNV, Offshore substations
/14/ DNV-RP-C201	DNV, Buckling Strength of Plated Structures, October 2010.
/15/ DNV-RP-C203	DNV, Fatigue design of offshore steel structures, September 2019 and Amendment January 2020
/16/ DNV-RP-C204	DNV, Design against accidental loads – 2019
/17/ DNV-RP-C205	DNV - Environmental conditions and environmental loads - September 2019, amended September 2021.
/18/ DNV-OS-C101	Design of offshore steel structures, general- LRFD method, 2021
/19/ DNV-OS-B101	Metallic materials, 2021
/20/ DNV-ST-0126	Support structures for wind turbine, 2018

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/21/ DNV-ST-0378	Standard for offshore and platform lifting appliance
/22/ ISO 14122-1 to 4	Safety of machinery – Permanent means of access to machinery

3.2 COMPANY SPECIFICATIONS

Doc. No.	Description
/23/ DEP 37.19.00.12	Gen - Offshore Topsides Structures
/24/ DEP 37.19.00.30	Gen - Fixed Steel Offshore Structures (Adoption of IOGP S-631-11, Amendments / Supplements to ISO 19902:2020).
/25/ DEP 37.81.10.19	Gen - Marine Operations Requirements – Loadout, Transportation and Installation of Offshore Structures (Amendments / Supplements to ISO 19901-6)
/26/ DEP 37.91.10.11	Mobile Mooring Systems (Amendments / Supplements to API RP 2SK, API RP 2SM AND API RP 2I)

3.3 PROJECT DOCUMENTS

Doc. No.	Description
/27/ 123A90-DDL-MD-RE-01006	Monthly Weight Report
/28/ 123A90-PML-MD-PRO-01002	Weight Control Procedure
/29/ 123A90-DDL-PP-DS-70070	Plot Plans
/30/ Pending	Handling Philosophy Report
/31/ 123A90-DDL-ST-SP-90018	Structural Material Specification for Topside & TP
/32/ 123A90-DDL-ST-SP-90118	Structural Material Specification for Flange & Bolts
/33/ 123A90-DDL-SAF-RE-30020	Fire and Explosion Risk Analysis (FERA)
/34/ 123A90-DDL-ST-RE-90000	In-service Analysis
/35/ 123A90-DDL-ST-RE-90006	Secondary Structure Design Report
/36/ 123A90-DDL-ST-RE-90120	Structural verification against accidental loads
/37/ 123A90-DDL-ST-RE-90007	BLPH Platform - FE analysis
/38/ 123A90-DDL-TI-SP-99014	Specification for TP, Topside and Caissons Transportation and Installation
/39/ 123A90-DDL-ST-TN-90119	Evaluating ISO and DNV approaches

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3.4 OTHER DOCUMENTS

Doc. No.	Description
/40/ K559–2254-ST-PHI-001	BLPH Structural Topsides Design & Installation Philosophy, Rev.1, 09 December 2022
/41/ K559–2254-ST-REP-001	BLPH Structural Topsides and Transition Piece Design and Analysis Report, Rev.0, 27 February 2023
/42/ 144671-VOOW-TF-ENG-REP-1011	Design Basis Part C, Re.4, 05-07-2021
/43/ SR.20.01165	Meteocean criteria for Hollandse Kust Noord Wind Farm zone (shell), Rev A03, 05-02-2021
/44/ K559–2254-WC-REP-001	BLPH Weight Control Report, Rev.2, 17 March 2023
/45/ 144671-VOOW-TF-ENG-REP-1051	Design Basis Part C for BLPH, Rev. 3, 28 June 2022
/46/ 144671-VOOW-TF-ENG-REP-1052	Wave Loads and Effects for BLPH, Rev. 0, 16 December 2021
/47/ 144671-VOOW-TF-ENG-REP-1016	Wave Run-up Study, Rev.2, 29 November 2021
/48/ 144671-VOOW-TF-ENG-REP-1103	Wave Impact Analysis, Rev.1, 7 May 2021
/49/ 144671-VOOW-TF-ENG-REP-1053	Axial Pile Capacity Assessment for BLPH, Rev.3, 07 July 2022
/50/ 144671-VOOW-TF-ENG-REP-1054	Foundation Load Document for BLPH, Rev.1, 07 April 2022
/51/ 144671-VOOW-TF-ENG-REP-1055	Design Verification Primary Steel for BLPH, Rev.1, 13 April 2022
/52/ 144671-VOOW-TF-ENG-REP-1056	Ship Impact for BLPH, Rev.0, 07 March 2022
/53/ 144671-VOOW-TF-ENG-REP-1502	Main Access Platform – Wave Run-up Braces
/54/ 144671-VOOW-GEN-SUR-REP-0074	Foundation As-built Survey CW70 HNA6
/55/ K559-2254-PM-PLN-003	Basic Design Engineering Package, Rev2, 06 July 2023
/56/ DK-TYYY-TNI-RPT-000-10002_B_IFC	Meteocean data for Tyra field development, 15/06/2018

3.5 SOFTWARE

Doc. No.	Description
/57/ SACS	Structural Analysis Computer System – BENTLEY Inc- v15
/58/ ANSYS	Ansys Simulation Platform – ANSYS Inc

4 DEFINITIONS AND ABBREVIATIONS

COMPANY or Client (CPY)	Crosswind
CONTRACTOR (CTR)	ROSETTI MARINO S.p.A.
PROJECT	HOLLANDSE KUST NOORD WIND PARK Project
T&I CONTRACTOR	DEME
ALS	Accidental Limit State
BEB	Best Estimate Bathymetry
BESS	BATTERY ENERGY STORAGE SYSTEM
BG	Box Girder
BLPH	Base Load Power Hub
Ce	Snow Exposure Coefficient
CF	Caisson Frame
CL	Concentrated Load
CoG	Centre of Gravity
CTV	Crew Transfer Vessel
CW	CrossWind
DAF	Dynamic Amplification Factor
DFF	Damage Fatigue Factors
DLB	Ductility Level Blast
DNV	Det Norske Veritas
DOP	Dropped Object Protection
DP	Dynamic Positioning
El.	Elevation
EPCI	Engineering Procurement Construction and Installation
FEED	Front End Engineering Design
HAT	Highest Astronomical Tide
HKN	Hollandse Kust Noord
ICCP	Impressed Current Corrosion Protection
LAT	Lowest Astronomical Tide
LRFD	Load and Resistance Factor Design

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MSL	Mean Sea Level
NTE	Not-to-Exceed
OD	Outside Diameter
PAU	Pre-Assembled Unit (H2 Vessels)
PG	Plate Girder
PV	Photovoltaic
SCE	Safety Critical Equipment
sk	Characteristic Value of Snow on the Ground/Platform
SGSI	Shell Global Solutions International
SHL	Static Hook Load
T&I	Transport & Installation
ToS	Top of Steel
TP	Transition Piece
TSO	Transmission System Operator
SLB	Strength Level Blast
UC	Unity Check
UDL	Uniformly Distributed Load
VIV	Vortex Induced Vibration
VOOW	Van Oord Offshore Wind
WTG	Wind Turbine Generator
W2W	Walk to Work

Table 4.1 - Definitions and Abbreviations

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5 PLATFORM DESCRIPTION

5.1 LOCATION AND ORIENTATION

The wind farm consists of 69 x 11.5 MW wind turbine generators each to be installed on a dedicated monopile structure. A 70th monopile, located on coordinates E 590719.1, N 5841602.3 (see Appendix B), will be reserved for installing the platform accommodating the BLPH.

The above co-ordinates are referenced to ETRS89 / UTM zone 31N global coordinate system.

Platform (Topsides) north is rotated 25° towards west with reference to True North so that the monopile main access platform is orthogonal to the Topsides, as per Figure 5.1.

5.2 BLPH TOPSIDES LAYOUT BASIS

The BLPH comprises of two main deck levels with overall deck plan dimensions of 40 m x 16 m, excluding external walkways, laydown areas and stairs, with an overall deck height of 7 m between decks. There are currently 6 hydrogen storage vessels which lie horizontally, aligned North - South, within the PAU structure located above the upper deck.

The Topsides layout is shown in Figure 5.1.

The Topsides is unmanned and there is no helideck. Main access is via Ladder access to sea for small Crew Transfer Vessel (CTV). Walk to Work (W2W) access is an alternative to W2W platforms provided on the lower deck, via Ampelmann.

Deck elevations are:

- Target top of TP, El. + 22.310 m;
- Lower Deck, El. + 23.310 m ToS;
- Upper Deck, El. + 30.310 m ToS.

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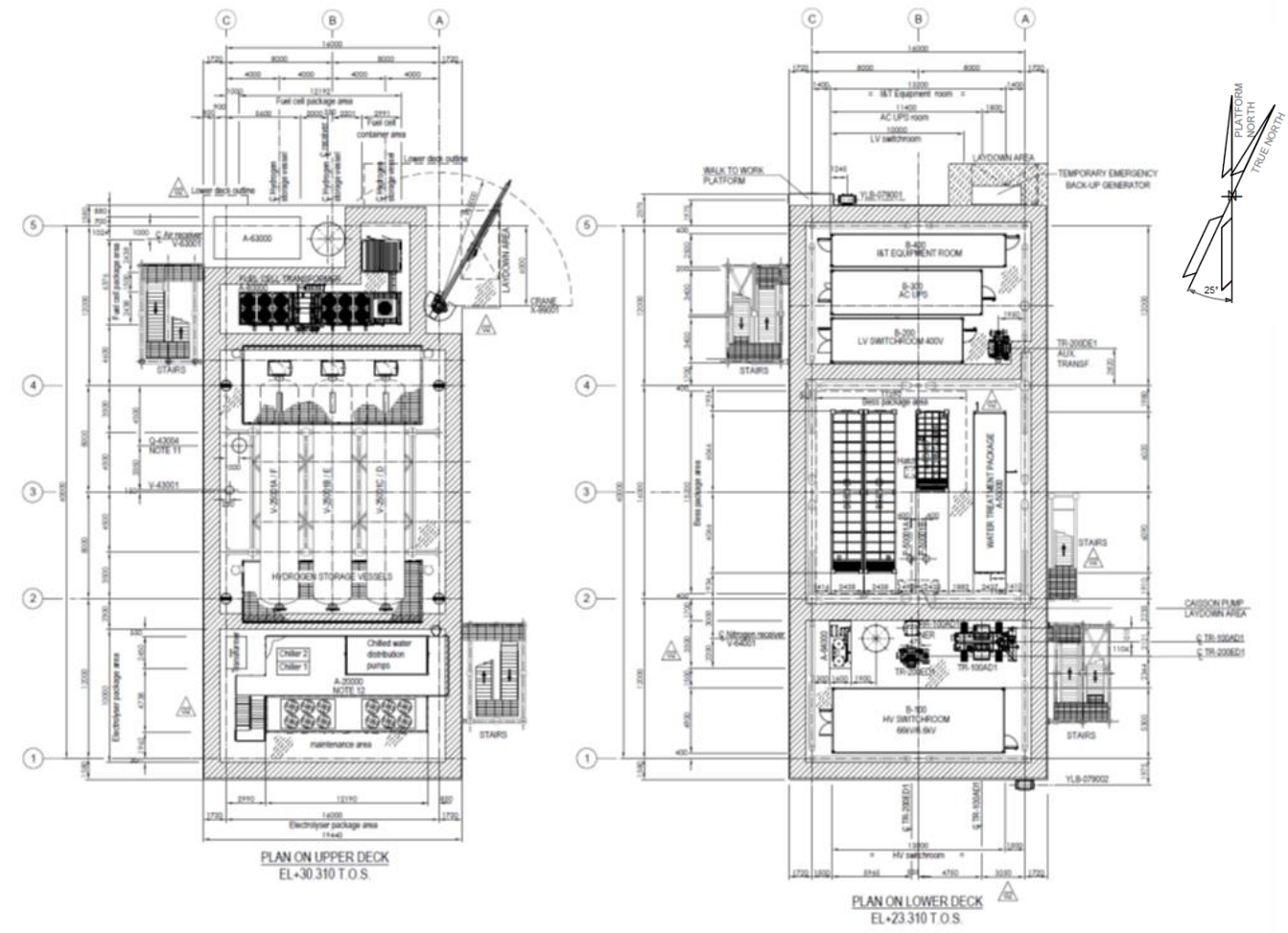


Figure 5.1 - Base Load Power Hub Plot Plan

5.3 MONOPILE-TOPSIDES INTERFACE

The interface level between Monopile and Topsides (comprising TP) is at El.+17.050 m relative to LAT, as per Ref. /40/ and /45/ and reported in Figure 5.2.

The TP will be connected to the monopile in offshore. The connection for applied TP in- place condition loads is within CONTRACTOR's scope.

VOOW has developed serviceability and ultimate limit state design forces and moments on the BLPH monopile. These are reported in Ref./50/and should not be exceeded.

The monopile has been surveyed on 29 May 2023 showing as-built interface connection as reported in Appendix 1.

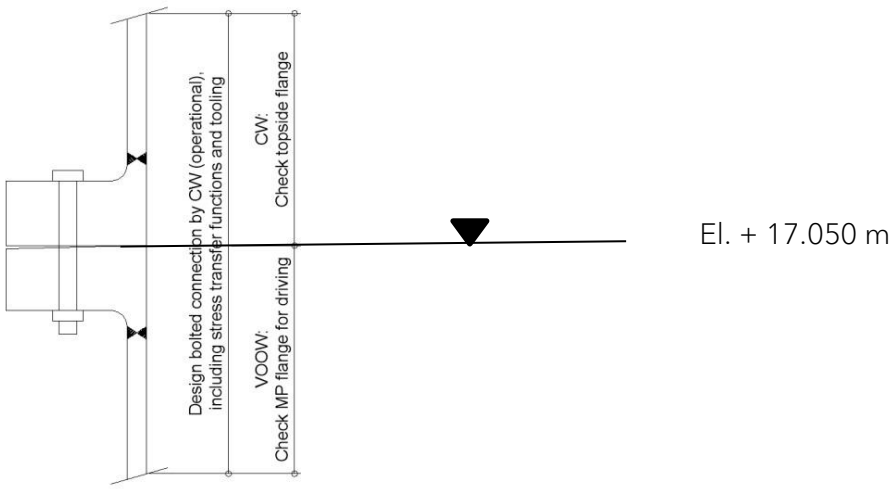


Figure 5.2– Monopile-to-Transition Piece Flange

The flanged connection will be designed to match the existing flange installed on monopile, following the same approach, Ref. /20/, and materials as per sections 7.4 and 7.5.

Analyses will be performed using FEM model with /58/ with design approach as per /20/.

6 DESIGN DATA

Design Data are herein collected according to Monopile and BLPH Feed Design, Ref. /40/ and /45/.

6.1 DESIGN LIFE

Despite the operational life of the platform is 2 years, a total design life of 25.5 years is required, as per following Table 6.1, from Ref. /40/

Description	Mode	Duration (years)
Construction Phase	Monopile Only	2.5
Operational Life	Monopile + Topsides	2.0
Non-Operational Life[1]	Monopile + Topsides	21.0
Total Design Life		25.5
Note: Topside assumed to be decommissioned and removed after 2 years, although this will be reviewed by CW.		

Table 6.1- Design Life

Corrosion allowances for caissons will be based on 25.5 years as per monopile. The monopile installation has been completed in 2023, with removal and decommissioning planned for 2048.

6.2 EXPOSURE LEVEL

ISO 19900 Ref./1/ classifies structure according to exposure level (L1, L2 or L3) based on:

- Life safety.
- Damage to environment.

The BLPH may be classified as exposure level L3 satisfying:

a) The platform is unmanned except for occasional inspection, maintenance and modification visits.

b) Visits are not planned to last more than 24 hours during seasons in which pre-determined hazardous events, for example environmental events, can occur.

c) Visits are not planned to last more than 24 hours on bottom founded platforms in regions with seismic Zone 4 as defined in ISO 19901-2, ref. /3/. (note zone 4 is the highest high seismicity zone, which does not apply to the North Sea).

6.3 DATUM

Elevations are refenced to LAT unless noted otherwise.

6.4 WATER LEVELS

Mean Sea Level (MSL) is 1.1 m above LAT, Ref./40/ and /45/.

Sea Level Rise (SLR) is: SLR= maximum 0.3 m by 2050 (>25 years), ref./42/.

Tidal water levels are given in Table 6.2. Maximum and minimum water levels for a range of return periods are provided in Table 6.3.

Tidal Value	Relative to LAT	Relative to MSL
Highest Astronomical Tide	2.2	1.0
Mean High Water Spring	1.9	0.8
Mean High Water Neap	1.6	0.5
Mean Sea Level	1.1	0
Mean Low Water Neap	0.6	-0.5
Mean Low Water Spring	0.3	-0.9
Lowest Astronomical Tide	0	-1.1

Table 6.2 - Extreme Tidal Levels, Ref. /40/ and /45/.

Extreme Water Level	Return period						
	1	2	5	10	50	100	1,000
Max	3.2	3.4	3.5	3.7	4.0	4.1	4.5
Min	-0.5	-0.6	-0.7	-0.8	-1.0	-1.1	-1.4

Table 6.3 – Extreme Water Levels including Storm Surge, Ref. /40/ and /45/, from MSL

6.5 MARINE GROWTH

Table 6.4 shows the recommended maximum marine growth according to Ref./42/.

Level [m] LAT		Thickness	Density
From	To	[mm]	[kg/m3]
HAT	LAT	150	1325
LAT	-6.0	150-50	1325
-6.0	Seabed	50	1325

Table 6.4- Marine Growth

6.6 WIND DATA

Omni-Directional wind speed data are reported in Table 6.5 , from Ref. /40/and /45/.

Averaging Time	Return period (years)						
	1	2	5	10	50	100	1000
1hr	31.8	33.4	35.1	36.4	39.1	40.1	43.4

10 min	33.7	35.5	37.4	38.8	41.7	42.9	46.5
3s	39.4	41.6	44.1	45.9	49.7	51.1	56.0

Table 6.5 – Omni-Directional Wind velocities [m/s] at 125 m MSL

Wind speeds are provided at 125m MSL. Wind speeds at any specified averaging time or elevation, will be calculated according to following equations, Ref./2/:

$U(z, t) = U(z) * [1 - 0.41 * Iu(z) * \ln(t/t0)]$

$U(z) =$ 1-hour mean wind speed given by:

$U(z) = U0 * [1 + C * \ln(z/zr)]$

$C = 0.0573 * (1 + 0.15 * U0)1/2$

$Iu = 0.06 * [1 + 0.043 * U0] * (z/zr)-0.22$

$U0 =$ 1-hour average at the reference elevation, zr

$zr =$ 10 m above sea level.

Directional exceedance data for wind is presented in Figure 6.1. The wind predominantly blows from the south- west.

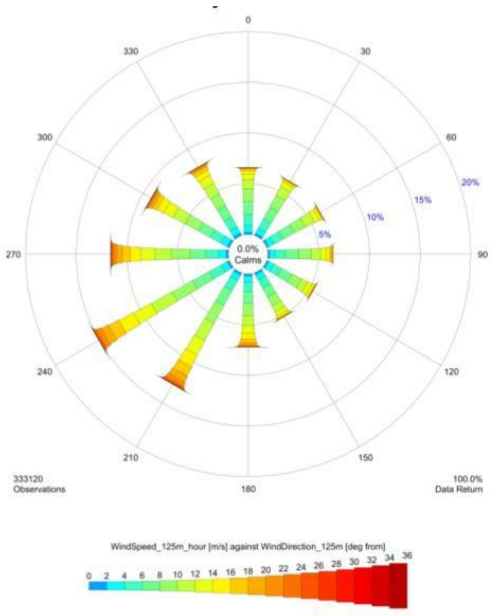


Figure 6.1 – Mean Hourly Wind Speed Distribution at 125 m MSL

6.7 TEMPERATURE & METEOROLOGICAL DATA

Description	Unit	Value
Ambient Conditions		
Ambient Temperature (min/nom/max) – hourly @ +10m	°C	-8.4 / 10.5 / 25.0
Atmospheric Pressure (nominal) – hourly @ +10m	mbar	1013.7
Specific Humidity (min/nom/max) – hourly @ +10m	x 10^{-3}	4.2/6.5/9.5
Wind Conditions		
Annual average wind speed (at 100 m)	m/s	9.56
Extreme Wind Speed for 1-Year Recurrence Period (at 100 m)	m/s	33.1
Extreme Wind Speed for 50-Year Recurrence Period (at 100 m)	m/s	41.0
Marine Conditions		
Sea Water Temperature (min/max)	°C	1.3 / 20.2
Average Sea Water Density	kg/m^3	1025

Table 6.6 – Meteorological and Oceanographic Data (source HKN2018 data @1979-2018), ref. /55/.

6.7.1 Rain Fall

Rain Fall [mm]													
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Min	14.1	18.1	9.1	1.2	10.3	17.3	6.2	20.7	27.2	43.0	22.2	28.1	646.6
Mean	75.0	62.5	48.6	39.6	47.4	47.9	58.2	73.0	91.0	102.7	108.5	89.3	843.6
Max	143.2	119.4	106.4	79.8	92.7	117.9	160.7	154.7	178.6	188.7	179.3	138.1	1056.1

Table 6.7 – Rain Fall, ref. /55/.

6.8 SNOW DATA

Snow loads will be calculated in accordance with FEED, Ref. /40/

- The Snow Exposure Coefficient (Ce) for every site in the Netherlands and Dutch Continental Shelf is Ce = 1.0.
- The Characteristic Value of Snow Load on the Ground/Platform (sk) shall be taken as sk = 0.7 kN/m^2.
- In the Netherlands including, its Continental Shelf, no exceptional snow loads need to be considered.

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6.9 ICE ACCRETION DATA

Atmospheric and marine ice accretion are specified in Table 6.8. Ice from sea spay and rain / snow will be considered separately, i.e., are not concurrent. According to Ref./46/ icing on top of marine growth is considered very conservative, therefore icing will be applied above marine growth.

Height	Sea Spray		Ice by Rain/Snow	
rel. to LAT	Thickness	Density	Thickness	Density
[m]	[mm]	[Kg/m³]	[mm]	[Kg/m³]
0 – 12.7	100	850	30	700
12.7 – 72.7	Linear 100 to 0	850	30	700

Table 6.8 - Ice Accretion on the Topside, Ref. /40/and /45/

6.10 BATHYMETRY

The area consists of a static and a dynamic bathymetry, where the dynamic part consists of moving sand banks and dunes. Following seabed levels from FEED data, Ref. /40/, will be used as input parameters for the structural detail design:

- Lowest Seabed Level: this is the lowest possible seabed level during the lifetime of the wind park until 2048;
- Highest Seabed Level: This is the highest possible seabed level during the lifetime of the wind park until 2048;
- Best Estimate Bathymetry (BEB): This is the best estimate seabed level during the lifetime of the wind park until 2048. The BEB is expected to have, on average, the smallest overall error.

The seabed levels (water depths) in 2023 (installation) to 2048 (decommissioning) are shown in Table 6.9.

Year	Lowest Seabed	Highest Seabed	Best Estimate
2023	-21.6	-20.0	-21.0
2048	-22.2	-20.0	-21.4

Table 6.9 – Seabed Level (m) relative to LAT

6.11 AIR GAP

The elevation of Lower deck is at El.+23.310 m LAT.

The offshore substation code DNV-ST-0145, sec.4.8.6, Ref./13/ states the air gap shall fulfil the below requirements:

- The air gap shall be at least 1 m for the 100-years design wave crest;
- The minimum air gap shall be > of the 20% of the 50-years significant wave height;

- The minimum air gap shall be > 0 for the 1000-years design wave crest.

The air gap calculation is shown in following Tables 6.10 and 6.11.

AIR GAP CALCULATION for GLOBAL CHECK				
Parameter	50-yr Return Period	100-yr Return Period	1,000-yr Return Period	Note
Significant Wave Height, Hs	7.30	7.60	8.40	values from Meteocean Report /43/ and in accordance with FEED
Wave Crest Elevation (Cmp)	9.20	9.60	10.90	values from Meteocean Report /43/ (FEED provides lower values)
Wave Crest Uncertainty, Hc_{unc}	1.38	1.44	1.64	As per ISO 19902 (sec. 6.3.3.2) Hc_{unc}=15%x Cmp
Sea Level Rise, SLR	0.30	0.30	0.30	from "Design Basis Part C for BLPH" Report /45/ (not considered in FEED)
EWL	12.30	12.70	14.30	Extreme water Level from Meteocean Report /43/
Settlement, f	0	0	0	Not considered due to short project duration
Highest Water surface El.+(LAT)	13.98	14.44	16.24	**=EWL + H_{cunc} + SLR + f**
Lower Deck Bottom of Steel EL.+(LAT)	22.31	22.31	22.31	design data
Clearance to Underside of Lower Deck PG Soffit	**8.33**	**7.87**	**6.08**	**Air Gap**

Topside Air Gap checks				
Check 1: Min Air Gap (100 yr)	7.87	m >	1 m	according to DNV -ST – 0145 – sec.4.8.6
Check 2: Min Air Gap (50 yr)	7.87	> 20% Hs =	1.52 m	according to DNV -ST – 0145 – sec.4.8.6
Check 3: Min Air Gap (1000 yr)	6.08	>	0 m	according to DNV -ST – 0145 – sec.4.8.6

Table 6.10 – Air Gap Check

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The wave run-up allowance is assumed as a rely upon data from FEED, ref./40/, where it is stated that, according to DNV-RP-C205, Ref./17/, "model tests are the only reliable way to establish wave run-up levels".

WAVE RUN-UP CALCULATION FOR LOCAL CHECKS				
Parameter	50-yr Return Period	100-yr Return Period	1,000-yr Return Period	Note
Significant Wave Height, Hs	7.30	7.60	8.40	values from Meteocean Report /43/ and in accordance with FEED
Wave Crest Elevation (Cmp)	9.20	9.60	10.90	values from Meteocean Report /43/ (FEED provides lower values)
Sea Level Rise, SLR	0.30	0.30	0.30	from "Design Basis Part C for BLPH" Report /45/ (not considered in FEED)
EWL	12.30	12.70	14.30	Extreme water Level from Meteocean Report /43/
Estimate of Wave Run-up, Hru	5.34	5.80	6.53	1000y Run-up calculated considering the ratio of (EWL 1000y / EWL 100y) x 5.80 = (14.30/12.70) x 5.80 = 6.53m
Settlement, f	0	0	0	Not considered due to short project duration
Highest Water surface El.+(LAT)	**17.94**	**18.80**	**21.13**	**=EWL + SLR + Hru+ f**
Lower Deck Bottom of Steel EL.+(LAT)	22.31	22.31	22.31	design data
Clearance to Underside of Lower Deck PG Soffit	**4.37**	**3.51**	**1.18**	**Air Gap**

Topside Air Gap checks				
Check 1: Min Air Gap (100 yr)	3.51	m >	1 m	according to DNV -ST – 0145 – sec.4.8.6
Check 2: Min Air Gap (50 yr)	3.51	> 20% Hs =	1.46 m	according to DNV -ST – 0145 – sec.4.8.6
Check 3: Min Air Gap (1000 yr)	1.18	>	0 m	according to DNV -ST – 0145 – sec.4.8.6

Table 6.11 – Wave run-up Calculation

It is noted that the lowest elevation of the TP support arms is El.+19.18 m LAT.

6.12 LEG GREEN

Based on the offshore survey results and as-built surveys of the completed topsides, green leg length s will be cut and prepare (bevel) the tops of the tubulars for welding. Only part penetration welds are required.

The TP will be fabricated to best suit the installed monopile position.

Conservatively assuming the leg green will be cut within a tolerance of +/-10mm (EEMUA 158 fabrication specification is more stringent), then the highest inclination (E/W) would be tan-1 (20/16000) = 0.072 degrees. The deck is a relatively stiff box, so if the cut-off elevation tolerance is higher the deck may be significantly over- stressed during set-down.

6.13 WAVES AND CURRENT

Waves and current characteristics are recalled from Ref./42/, /43/, /45/ and /46/. Following Normal Sea State (NSS), Extreme Sea State (ESS) and Impact Sea State (ISS) will be considered in structural analyses.

6.13.1 Normal Sea State (NSS)

From ref. Ref./46/, significant wave height with recurrence period of 1-year and associated peak period are:

- Hs,1y = 5.6m;
- Tp = 10.0s.

In Ref./45/ the omnidirectional the most severe Normal Sea State (NSS) as shown in Table 6.12 is provided and will be considered in structural analyses.

Hs [m]	Tp [s]	WL [m]	V_{Cavg} [m/s]	$V_{c;SWL}$ [m/s]
6.44	10.17	2.47	0.30	0.99

V_{Cavg} = total depth averaged current (tidal + wind)
$V_{c;SWL}$ = total current (tidal + wind) at Still Water Level

Table 6.12 – Most severe NSS load case for omni-directional wave

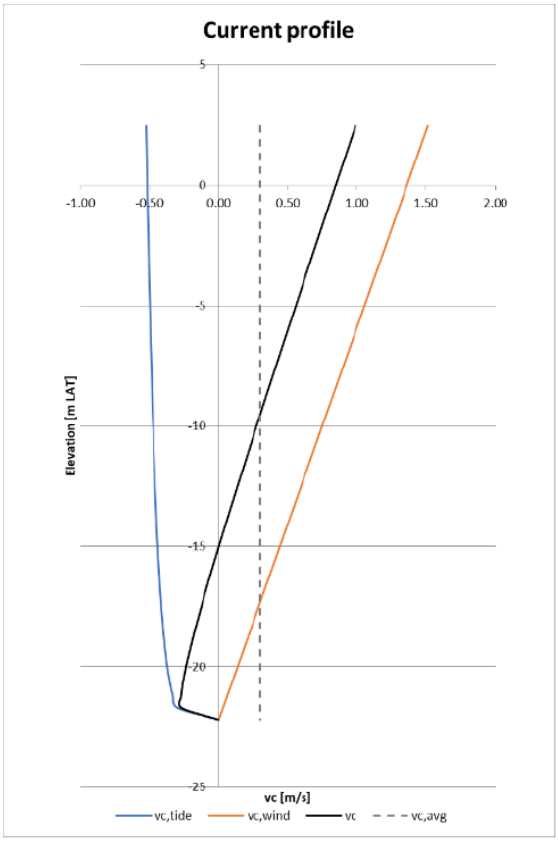


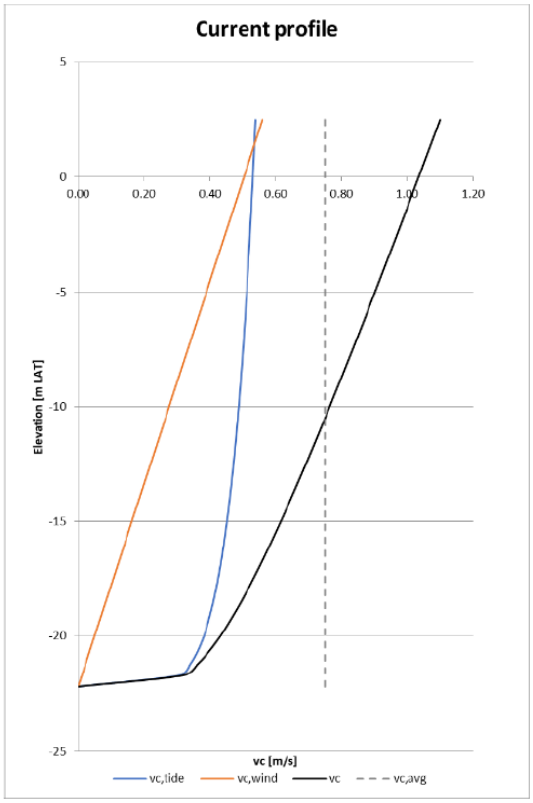
Table 6.13 – Current profile for NSS for $v_{wind,2hr,125\,[m]\,MSL}$ = 35 m/s:

6.13.2 Extreme Sea State (ESS)

In Ref./45/ the omnidirectional Extreme Sea State (ESS) as shown in Table 6.12 is provided and will be considered in structural analyses.

RP [years]	Hs [m]	Tp [s]	Wlmin [m]	Wlmax [m]	V_{Cavg} [m/s]	V_{CSWL} [m/s]	H_{mp} [m]	T_{Hmp_min} [s]	T_{Hmp_avg} [s]	T_{Hmp_max} [s]
50	7.3	11.5	2.5	3.6	0.75	1.1	14.0	10.54	11.71	12.88
100	7.6	11.8	2.5	3.6	0.75	1.1	14.5	10.70	11.89	13.08
Vcavg = total depth averaged current (tidal + wind) Vc;SWL = total current (tidal + wind) at Still Water Level										

Table 6.14 – ESS load cases



ESS100yr

Table 6.15 -Current profile for ESS (100 yr)

6.13.3 Impact Sea State (ISS)

The maximum 50 years wave from Table 6.14 is considered for slamming wave impact calculations according to Ref./46/.

6.14 WAVE DATA FOR SEA TRANSPORT FATIGUE

Omnidirectional scatter diagram applicable for the route and season with corresponding exposure and tow duration will be provided by Transportation study. Ref to sections 10.4 and 10.7.

6.15 SPLASH ZONE

The design splash zone is determined in Ref./42/.

External splash zone:

- Upper limit external splash zone = +7.0 [m] LAT;
- Lower limit external splash zone = -2.8 [m] LAT.

Internal splash zone:

- Upper limit internal splash zone = +3.6 [m] LAT;
- Lower limit internal splash zone = -0.6 [m] LAT.

6.16 CORROSION ALLOWANCE

No corrosion thickness allowance shall be added to any Topside and Transition piece structure above interface level at el.+17.05, as painting and coating system shall provide necessary protection.

According to Ref. /42/ corrosion allowance for zones that are subjective to coating damages (such as the boat landing fenders and upper side of ladder rungs) shall be taken as 0.3 mm/year. A corrosion allowance of 10mm will be considered for caissons design and check.

6.17 DRAG AND INERTIA COEFFICIENTS

Hydrodynamic coefficients for unshielded circular cylinders as per ISO code, Ref./7/, is given in Table 6.16. An additional 5% increase factor is applied to all Cd/Cm to account for minor appurtenances and anodes where applicable.

Surface of component	Cd	Cm
smooth	0,65	1,6
rough	1,05	1,2

Table 6.16 – Drag and Inertia Coefficients

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7 MATERIALS

7.1 STRUCTURAL STEEL PROPERTIES

For the structural analyses, steel types and grades to be used for the platform structures are summarised in Table 7.1 and specified in /31/ The material selection criteria are based on the requirements of ISO 19906:2019, ISO 19902:2020 and ISO 19901-3:2014. The materials are in accordance with the EN standard 10225:2019 and 10025:2019 and are suitable for plates, rolled sections and welded / seamless tubulars.

			EN10025-3	ISO EN 19902:2020						
Design Class	**Complexity**	**SQL**	**Hot rolled bar**	**Plates and Welded Tubular**	**Sections**	**SMLS pipes**	**THK**	**CVN inpact test**	**Material Designation**	**List of Options (EN10225-1 to -4) & EN10025-3**
DC1 TP to monopile flange connection	High	**I (Z properties)**	EN 10025 S420 NL	NA	NA	NA	t > 50	-40	DC1-I-420-40_Z	1, 2, 3, 4, 6, 12, 18, 24, 30
DC1 Complex lift-nodes and element connection to topside, Lift points, main complex joints	High	**I (Z properties)**	NA	EN 10225 S420 QLO / ML	NA	NA	25 < t <= 50	-40	DC1-I-420-40_Z DC1-I-460-40_Z	2, 7, 8, 11, 12, 17, 18
							t > 50	-40		
DC2 simple joints, primary deck girders and beams of Topside and TP, column and diagonal to lift points, main columns, Davit crane support, Bolts	Low	**II**	NA	EN 10225 S420 QLO / MLO	NA	NA	t < = 12	0	NA	-
							12 < t <= 25	-20	DC2-II-420-20 / DC2-II-460-20	2, 7, 8, 11 17 for t > 40mm
							25 < t <= 50	-40	DC2-II-420-40 DC2-II-460-40	
							t > 50	-40		
DC3 all other columns and diagonals, stiffeners, Secondary deck beams, Grillage and sea fastening	High	**II**	NA	EN 10225 S355 NLO / MLO	EN 10025 S355 MLO	EN 10225 S355 NLHHO / QLHHO API 5L X52	t < = 12	0	DC3-II-355-0 / DC3-II-X52-0	2, 7, 8 11 for plate 17 for t > 40mm
							12 < t <= 25	-20	DC3-II-355-20 / DC3-II-X52-20	
							25 < t <= 50	-40	DC3-II-355-40 / DC3-II-X52-40	
							t > 50	-40		
DC4 Equipment and vessel supports, modules supports, equipment fixing bolts	Low	**III**	NA	EN 10025 S355 J2	EN 10025 S355 J2	API 5L X52 EN 10210 S355 J2H	t < = 12	0	DC4-III-355-0 / DC4-III-X52-0 / DC4-III-X60-0	-
							12 < t <= 25	0		
DC5 floor plating, grating, Walkw ays, stairs, handrails	Any	**IV**	NA	EN 10025 S235 JR EN 10025 S275 J0 EN 10025 S355 J0	EN 10025 S235 JR EN 10025 S275 J0 EN 10025 S355 J0	API 5L Gr.B EN 10210 S235 JRH / EN 10210 S275 J0H / EN 10210 S355 J0H	t < = 12	10	DC5-IV-235-10 / DC5-IV-275-10 / DC5-IV-355-10 / DC5-IV-GRB-10	-
							12 < t <= 25	10		

Table 7.1 – Material Selection, Ref /31/, and requested Options

Mechanical properties for structural materials are presented in Table 6.8, Table 6.9 and Table 6.10.

- Young's Modulus = 205,000 N/mm^2
- Density = 7850 kg/m^3
- Poisson's Ratio = 0.3

Thickness (mm)	Min. Tensile Strength (N/mm²)	Min. Yield Strength (N/mm2)	Steel Type (1)
t > 250	460	320	I
Note: 1) Type I steel has Z (through the thickness) properties.			

Table 7.2 – Mechanical properties of Hot rolled bar

Thickness (mm)	Min. Tensile Strength (N/mm²)	Min. Yield Strength (N/mm2)	Steel Type (1)
t ≤ 16	500	420	I & II
16 < t ≤ 40	500	400	I & II
40 < t ≤ 63	480	390	I & II
63 < t ≤ 100	480	380	I & II
Note: 1) Type I steel has Z (through the thickness) properties.			

Table 7.3 – Mechanical properties of Plates

Thickness (mm)	Min. Tensile Strength (N/mm²)	Min. Yield Strength (N/mm2)	Steel Type
t ≤ 20	460	355	III
20 < t ≤ 40	460	345	III

Table 7.4 – Mechanical properties of Structural Seamless Tubulars

Thickness (mm)	Min. Tensile Strength (N/mm2)	Min. Yield Strength (N/mm2)	Steel Type
t ≤ 16	460/360	355/275	IV
16 < t ≤ 40	460	345	IV

Table 7.5 – Mechanical properties for Structural Sections

7.2 PLATING

Preferred plate thicknesses for primary structural steel will start at a minimum of 10 mm and increase in 5 mm steps Secondary steel lower and upper deck plate will be 8 mm thick with laydown areas of 10 mm for Dropped Object Protection (DOP).

Blast walls are not to be provided on a hydrogen processing facility, as free ventilation and free explosion venting is more important, and supportable, than obstruction walls.

7.3 GRATING

Grating will be with bearing bars 30 mm x 5 mm and mesh 30 mm x 50 mm, S235 material.

7.4 FLANGE

Flange will be designed in accordance with Ref. /20/ and to fit the existing one already installed on monopile; The specification /32/details the material requirements and tests.

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7.5 BOLTS

Bolts for TP to monopile will be classified as primary structure (or DC2), purchased in accordance with Ref./20/ and /23/. Requirements are reported in documents /32/.

Bolts for equipment fixing will be classified as secondary structure (or DC4).

8 DESIGN CRITERIA AND REQUIREMENTS

8.1 GENERAL

Structures in SOW will be designed in accordance with limit states approach of ISO codes 19901 and 19902, Refs./2/, /4/ and /7/. For non-tubular sections, EC code, Ref./10/, will be adopted. Member forces and moments derived from the global analyses will be used for checking member strength and stability, and joint strength.

Four different design limit states are defined and will be satisfied by the design:

- the Serviceability Limit State (SLS), corresponding to the criteria applicable to normal use or durability;
- the Ultimate Limit State (ULS), corresponding to the ultimate capacity for carrying loads;
- the Accidental Limit State (ALS) corresponding to damage due to an accidental event;

The general requirement of ISO code is that, for the structure as a whole and for each component, the following is satisfied:

$$R_d = \frac{R_k}{\gamma_M} > \gamma_F F_r = F_d$$

Where:

- R_d is the design resistance;
- R_k is the characteristic resistance;
- γ_M is the partial resistance factor reported in section 8.2;
- F_d is the design value of action;
- F_r is the characteristic value of action;
- γ_F is the partial action factor reported in Sec.8.4.

Design according to the provisions for load and resistance factor design (LRFD) of EC3, ref. /10/ shall be performed in accordance with following equation:

$$R_u \leq \phi R_n$$

Where:

- R_u = required strength using LRFD load combinations
- R_n = nominal strength
- ϕ = resistance factor reported in Sec.8.2
- ϕ Rn = design strength.

8.2 PARTIAL RESISTANCE FACTORS

8.2.1 Partial Resistance Factors for Tubular Sections:

The partial resistance factors for tubular sections from ISO 19902, Ref./7/, are listed below.

Resistance factor	Value	Definition
$\gamma_{R,t}$	1.05	Partial resistance factor for axial tensile strength
$\gamma_{R,c}$	1.18	Partial resistance factor for axial compressive strength
$\gamma_{R,b}$	1.05	Partial resistance factor for bending strength
$\gamma_{R,v}$	1.05	Partial resistance factor for shear strength
$\gamma_{R,h}$	1.25	Partial resistance factor for hoop buckling strength

Table 8.1- Resistance Factors for Tubular Sections

8.2.2 Partial Resistance Factors for Non-tubular Sections

Referring to ISO 19901-3, Ref./4/, section 8, to be consistent with ISO 19900 inherent the level of safety, a "building code correspondence factor" is to be applied to the EC partial factors.

The partial resistance factors for non-tubular sections from Eurocode 3, Ref./10/, are listed below.

Resistance factor	Value	Definition
Resistance of members and cross sections		
γ_{M0}	1.10*	Cross-section resistance to excessive yield strain including local buckling
γ_{M1}	1.10*	Resistance of unstable elements measured during element inspection
γ_{M2}	1.25	Tensile cross-section resistance to mechanical breakdown
Resistance of connections		
γ_{M2}	1.25	Bolts, rivets, pins, welds and slabs on supporting surface
γ_{M3}	1.25	Sliding resistance at Ultimate Limit State Category C
$\gamma_{M3,SER}$	1.10	Sliding resistance at Serviceability Limit State
γ_{M4}	1.10	Injection bolt collapse resistance
γ_{M5}	1.10	Resistance of connections in open lattice trusses
$\gamma_{M6,SER}$	1.00	Resistance of pins at Serviceability Limit State

γ_{M7}	1.10	Preload of high strength bolts

Table 8.2- Resistance for Non Tubular Section

These resistance factors apply to the design of members and are not applicable to local details, such as padeyes. Reference should be made to ISO 19902 for a full listing of partial resistance factors and to Eurocode 3, for welds and bolts design resistance factors.

Note *: The correspondence factor has been considered by increasing the Partial safety factor $\gamma M0 = \gamma M1$ from 1.00 to 1.10. That safety factors from Eurocode 3 (UNI-EN-1993-1-3) are greater than or equal to safety factors from ISO19902:

$\gamma M0 = \gamma M1 = 1.10$ (UNI-EN-1993-1-3) ≥ yR,t (=1.05); yR,c (=1.10); yR,b (=1.05); yR,v (=1.05) (ISO19902)

8.2.3 Partial Resistance Factors for Tubular Joints Check

The partial resistance factors for tubular joints checks are provided in following table.

Resistance factor	Value	Definition
$\gamma_{R,j}$	1.05	Partial resistance factor for joints
$\gamma_{z,j}$	1.17	Additional partial resistance factor for members during joint pre-strain collapse

Table 8.3 - Resistance Factors for Tubular Joints

8.2.4 Partial Resistance Factors for Accidental Limit State Actions

All partial resistance factors for the abnormal / accidental limit state conditions are set equal to 1.00.

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8.3 LOAD CLASSIFICATIONS

Design loads are classified as either global or local within ISO 19901-3, Ref. /4/ and ISO 19902, Ref./7/ and summarized in Table 8.4.

Load Type	ISO Designation	Description
Permanent (Dead)	G1	Structure self-weight; Equipment & piping dry weight; Any actions due to misalignment of structures such as between the topside and TP
Permanent (Dead)	G2	permanent action imposed on the topsides structure by self-weight of equipment and other objects that remain constant for long periods of time, but which can change from one mode of operation to another, or during a mode of operation, bulk loads
Variable (Live)	Q1	Equipment & piping contents weight; Superimposed area loads (see Section 9.2.2.2); Ice and snow accretions
Variable (Live)	Q2	Crane hook loads; Liquids in pipes and process vessels for pressure testing; Machine operations
Environmental (extreme)	Ee	Extreme quasi-static action due to wind, waves and current (eg.100-yr storm wind)
Environmental (operational)	Eo	1-yr storm wind
Inertial Response (operational)	Do	Equivalent quasi-static action representing dynamic response caused by the Eo wave condition
Inertial Response (extreme)	Do	Equivalent quasi-static action representing dynamic response caused by the Ee wave condition
Pressure (accidental)	-	Blast
Inertial (accidental)	-	Ship impact on monopile
Impact Energy (accidental)	-	Dropped Object
Transient	T	Transient Loads for Pre-Service Analyses

Table 8.4 – ISO Loads Classification

8.4 PARTIAL ACTION FACTORS

8.4.1 Partial Action Factors for In-Service Analysis

Partial action factors to be applied for the operating and seismic conditions in compliance with ISO codes are reported in Table 8.5.

Load combination		Dead load		Live load		Environmental load	Wave dynamics
		G1	G2	Q1	Q2	Eo / Ee	Do / De
		$\gamma_{f,G1}$	$\gamma_{f,G2}$	$\gamma_{f,Q1}$	$\gamma_{f,Q2}$	$\gamma_{f,Eo}$ / $\gamma_{f,Ee}$	$\gamma_{f,D}$
Operating Gravity / Buoyancy		1.3	1.3	1.5	1.5	-	-
Operating Storm		1.3	1.3	1.5	1.5	1.22	1.25
Extreme Storm (100 years)	Max Compression	1.0	1.0	1.0	-	1.35	1.25
	Max Tension	0.9	0.9	0.8	-	1.35	1.25

Table 8.5 – Partial Action Factors for the In-Service Analysis

8.4.2 Partial Action Factors for Pre-Service Analyses

Partial Action Factors for Pre-Service analysis (e.g. Lifting, Transportation, Load-out) conditions are shown in Table 8.6 with reference to three design situations, where:

- G_T are the permanent actions during the transient situation.
- Q_T are the variable actions during the transient situation.
- T represents the actions from the transient situation being considered.

SITUATION	ACTION		
	Permanent (GT)	Variable (QT)	Transient (T)
1	1.3	1.3	1.0
2	1.1	1.1	1.35
3	0.95	0.9	1.35

Table 8.6 – Partial action factors for Pre-service Analyses, Ref. /7/

8.4.3 Partial Action Factors for Accidental Limit State Actions

All partial action factors for the abnormal / accidental limit state conditions are set equal to 1.00.

8.4.4 Partial Action Factors for Fatigue Analysis

In determining stress variations for fatigue analysis, all the partial action factors shall be taken as 1.0.

8.4.5 Partial Action Factors for Serviceability Limit States

All partial action factors for Serviceability Limit States conditions are set equal to 1.0.

8.5 ACCEPTANCE CRITERIA

The maximum member Unity Check (UC) will be less than unity.

8.6 SPAN LIMITS

Maximum allowable beam span limits are provided in Table 8.7.

Structural Component	Member Length / Member Height
Primary members	18
Members carrying sensitive machinery / vessels	15
All other members	22
Deck plate and stringers	30

Table 8.7 – Maximum Allowable Beam Span Limits

On the BLPH topsides, 1 m deep Plate Girders (PGs) will be used with a span to depth ratio of 16, which is considered acceptable as the PGs will be continuous over the long span. The TP arms will be Box Girders (BGs). BGs are preferred as they have better corrosion resistance, torsional strength and are robust in a potential wave slam load application.

8.7 SERVICEABILITY LIMITS

ISO 19901-3, Ref./4/ states structural member deflections under serviceability conditions must not impair the serviceability of the structure. Corresponding deflection limits are presented in Table 8.8.

Condition	Limit for Δ_{max}	Limit for Δ_2
Deck Beams	L / 200	L / 300
Cantilever Beams	L / 100	L / 150
Deck plate thickness	-	2t or b/150 whichever is smaller

Where:

L = span of the beam, length of the cantilever;

t = deck plate thickness;

b = stiffener spacing;

Δ_{max} = final deflected shape = $\Delta_1+\Delta_2-\Delta_0$;

Δ_0 = any pre-camber (hogging) prior to the addition of any permanent or variable actions;

Δ_1 = deflection from permanent actions after applying the actions;

Δ_2 = deflection from the variable actions and any time-dependent deformations from permanent actions.

Table 8.8- Limiting Values for Vertical Deflection

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8.8 HAND CALCULATIONS

Structural design of secondary and outfitting structures (e.g. equipment/piping supports) can be carried out by hand calculations. LRFD method, according to EC code (Refs./10/ and /11/), can be adopted for local structural checks.

9 STRUCTURAL DESIGN & LOADS

9.1 STRUCTURAL MODEL

The software SACS™, ref./57/, is used for all global analyses of the structures in the SoW.

The model geometry will be referred to a right-handed orthogonal Cartesian coordinate system with the origin located at the centre of the foundation structure at level +0.0 m relative to LAT. The X-axis will be directed toward East, the Y-axis will be directed toward North and the Z-axis will be upward.

The structural models of the Topside, TP and caissons will be representative of the actual structure and shall include the following:

- main structural components: all primary framing members such as columns, truss braces, topside girders, floor bracing and members supporting large and heavy equipment;
- structural components which presence generates significant environmental loads;
- deck plating will be modelled using plate finite elements with shear resistance. Flooring beams will not be included in SACS model and in grated areas no account will be taken of any additional stiffness due to the grating itself;
- secondary members will be modelled where they are significant in consideration of load paths and/or overall stiffness and/or contribute to structural capacity. For example: beams supporting heavy equipment and/or large vessels.

Other secondary and tertiary structures are generally not be modelled. Their effect on primary steel (such as local loading or buckling restraint) and corresponding loads will be considered within the modelled supporting structures.

Structural members like beams, columns, braces, etc. shall be modelled with proper cross-sectional properties correct lengths. Member ends' restraints and constraints, if any, will reflect the actual released / restrained degrees of freedom.

Material property assignments will account for varying member thickness and steel type.

Member offsets and eccentricities will be modelled to take into account all secondary moment effects at a beam/brace intersection.

The Topsides and TP joints will be modelled at the TOS elevation for each plan level. Vertical member offsets will be incorporated to adjust the ends of the plan members to the centrelines of the sections.

The member slenderness will be defined considering the following effective length factors for in-plane (Ky) and out-of-plane (Kz):

• Braced legs: Ky = 1.0, Kz = 1.0;

• X-bracings: Ky = 0.9, Kz = 0.8 of the total length or 0.9 of the longer segment length if the opposite ones are in tension;

- Floor beams: Ky = 1.0 Kz = 1.0;
- Cantilever beams: Ky = 2.0 Kz = 2.0.

Further and more detailed guidance on the selection of effective length factors for member stability checks is given in ISO 19902, ref./7/.

The deck beams out-of-plane effective length (Lz) will be set equal to 1.00 m for the beams supporting, and then restrained by, the secondary floor beams.

The beams unbraced length (Lb), governing the buckling length of the compressed flange, will be set equal to the beam length. Based on the moment diagram of the beam resulting from the analyses, this length Lb will be revised where resulting the discriminant parameter of the beam checks.

A stand-alone structural SACS model of the CF will be developed. The model includes members, joints and supports. Analyses will be performed considering fixed restraints at the top hang-off clamps and horizontal restraints at the guide clamps. Caissons' marine growth weight and buoyancy will be included in the structural self-weight, since automatically evaluated by SACS™ based on the input marine growth profile and density.

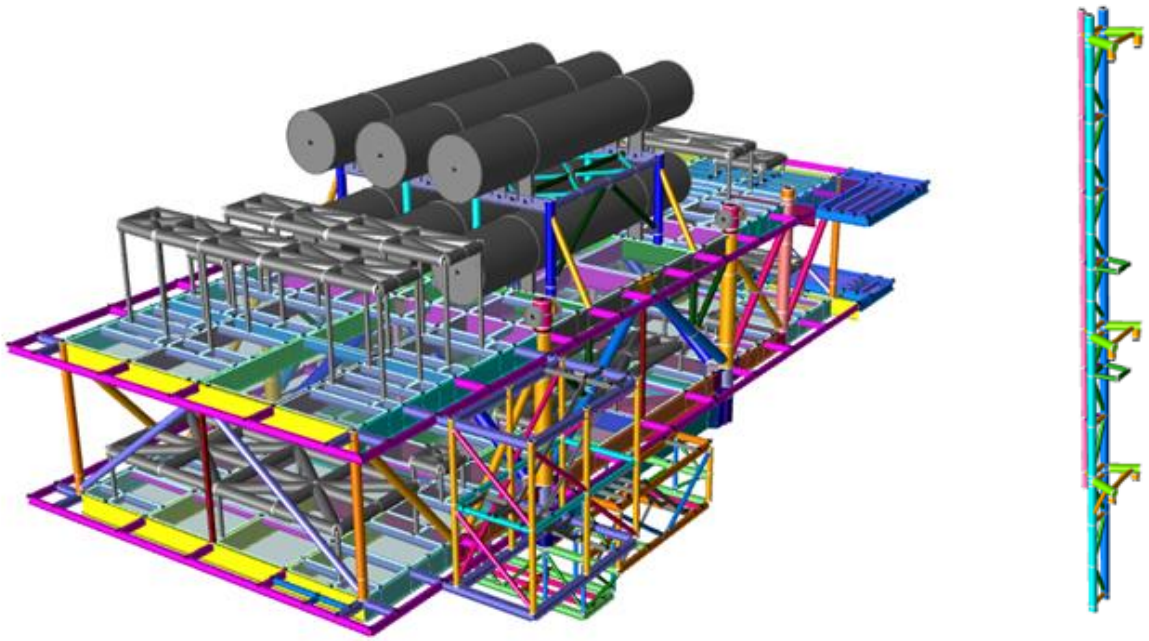


Figure 9.1 – Topside and TP (left) and Caissons Frame (right) SACS structural models

9.2 LOADINGS

All the design loads will be defined according to the Weight Control Report and Plot Plans, Ref./27/ and /29/, in terms of weights, CoG position and contingency factors.

The datum for weight reporting is different to that for the SACS structural analysis. The origin for the weight reporting is at gridline intersection C1 at LAT.

9.2.1 Permanent Actions

9.2.1.1 Structural dead weights

Dead loads will include the weight of all structural members either modelled or non-modelled. Dead weights of the modelled structures will be automatically evaluated by SACS considering the section properties while for the secondary and tertiary structures not included in the model it will be covered by the corresponding applied loads.

9.2.1.2 Buoyancy

Buoyancy is evaluated by SACS using the marine method, where gravity loads are calculated based on the submerged member weight and then applied to members as distributed load acting vertically. The net buoyancy will be calculated as the difference between the total buoyancy and the weight or water entrapped inside flooded members.

9.2.1.3 Equipment and Bulk weights

These loads include dry weight of all platform process/utility equipment and electrical & instrumentation equipment and other objects permanently mounted on the structure that do not change with the mode of operation. Equipment loads will be modelled in SACS as uniform or concentrated loads and Bulk loads will be applied as uniform loads.

9.2.1.4 CoG shift

According to DNV code. Ref. /12/, for the purpose of Topside and TP detail design analyses, the topsides CoG location is considered to be within a CoG tolerance envelope of 5% the distance between the structure length and breadth:

- Longitudinal Cog shift envelope (along global X axis; E-W direction) = 5% x 16 m = 0.8 m (±0.4m)
- Transversal Cog shift envelope (along global Y axis; N-S direction) = 5% x 40 m = 2 m (±1.0m).

CoG shift will be applied starting from the CoG obtained from WCR, with the restriction to remain within a radius of 1.5m with origin in the centre of the platform, see 10.1.

CoG shift will be obtained by applying suitable couples made by vertical upward and downward forces applied at the corners of the structure.

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9.2.2 Variable Actions

9.2.2.1 Equipment & Piping content weights

Equipment & piping contents weights will be modelled in SACS as uniform or concentrated loads.

9.2.2.2 Live loads

For global design analyses (e.g. In-Service) only superimposed area loads on laydown platforms will be considered in load combinations.

Area loads for local design are to be considered as per Table 9.1.

Area Use	Deck Flooring & Stringers	
	UDL (kN / m2)	CL (kN)
General Laydown and Storage Areas	15	22.5 (+10.0 for pipe support allowance)
Floor Areas in Deck	Actual equip. +5.0 in open areas or 15.0	22.5 (+10.0 for pipe support allowance)
Temporary Refuge	5	5
Walkways, Stairs and Access Platforms	4.5	4.5

- Concentrated Loads (CL) are applied to an area of 100 mm x 100 mm, and at the most severe position, but not added to wheel loads or distributed loads;
- Uniformly Distributed Loads (UDL) and CL are not considered to act together, except 10 kN pipe support allowance is assumed to act with any load combination.
- Wheel loads to be added to distributed loads, where relevant. Wheel loads can normally be considered acting on an area of 300 mm x 300 mm.
- Trolley route location and loads to be extracted from Mechanical Handling layouts.

Table 9.1 – Superimposed Area Loads for Local Checks

Primary deck beams are to be locally checked against same loads presented above, for both lower and upper deck.

For handrail the minimum service load applied horizontally to the top of the stanchion and then along the handrail, shall be:

Area Use	Load	Ref.
Service platforms	0.30 kN/m, per max distance in m, between the axes of two successive stanchions	/22/
Escape routes	0.50 N/m, per max distance in m, between the axes of two successive stanchions	FEED, /40/
Laydown areas	1.50 kN/m per max distance in m, between the axes of two successive stanchions	-

Table 9.2 – Handrail design loads for Local Checks

9.2.2.3 Ice and snow loads

Snow and ice build-up data on the platform are presented in Sec.6.8 and 6.9

Three cases are to be considered:

1) Snow load build-up on horizontal surfaces (0.7 kN/m2 as per sec. 6.8).
2) Ice build-up on horizontal surfaces due to sea-spray (850 kg/m3 as per sec. 6.9). Build up is only considered on the windward side of the platform.
3) Atmospheric build-up ice (30 mm thickness as per sec. 6.9), on horizontal and vertical surfaces, due to snow and rain at 700 kg/m3 .

According to FEED, Ref. /41/, snow loads build-up on horizontal surfaces (case 1) and atmospheric build-up of 30 mm thick ice, on horizontal and vertical surfaces, due to snow and rain (case 3) are considered concurrent, though not accumulative on horizontal surfaces.

Ice build-up on horizontal surfaces due to sea-spray (case 2) is only considered on the windward side of the platform and as a separate load case, not concurrent with Snow load (case1) and atmospheric build-up (case3).

9.2.2.4 Davit Crane hook loads

Davit Crane operating loads, including forces and moments, will be applied at the top of the pedestal. Davit crane is requested with1 ton at 10m radius and 3.5ton at 5m, fully revolving and suitable for offboard lift.

The structural checks will be performed according to DNV-ST-0378, Ref. /21/.

Considering the limited design life of the platform and the maintenance required every 6 months, the maximum number of cycles is estimated of about 1000 cycles including hook-up phase. The Ref. /4/ states in section 9.6 that fatigue analysis shall be based on the expected crane usage with a min. of 25000 lifting cycles. Being the expected cycles less than 5% of the minimum suggested by code in conclusion seems appropriate to skip the fatigue analysis.

9.2.3 Environmental loads

The following environmental loads shall be considered:

- Wind;
- Wave and current.

9.2.3.1 Wind loads

Operating wind loads will be defined considering 1-year return period. The 100-years return period will be used for the extreme environmental conditions.

Unfactored 100-yr wind loads at the monopile interface, El.+17.050 m LAT, should not exceed Ref. /40/ and Ref. /50/:

- Shear = 1247 kN;
- Moment = 13639 kNm.

Wind loads during detail engineering design will be checked to be within the aforementioned limits.

The equivalent static wind force, F_{wind}, acting on the structures may be calculated according to Refs. /2/and /7/:

$$F_{wind} = \frac{1}{2}\ \rho_a\ U_w\ C_S\ A_r$$

Where:

ρ = air density (1.225kg/m^3)

A = area of the object (m^2)

U = wind speed (m/s)

Cs = shape coefficient

As per Ref./2/ the wind speed in a 3 s gust is appropriate for determining the maximum quasi-static local actions caused by wind on individual components of the structure, whereas 5 s gusts are appropriate for maximum quasi-static local or global actions on structures whose maximum horizontal dimension is less than 50 m, and 15 s gusts are appropriate for the maximum quasi-static global actions on larger structures.

As per Table 9.3, a Shape coefficient Cs=1 will be considered for overall projected area of structures in global analyses. Wind loads will be concurrent with wave and current loads.

Component		Shape coefficients C_s
Flat walls of buildings		1,50
Overall projected area of structure		1,00
Beams		1,50
Cylinders	Smooth, $R_e > 5 \times 10^5$	0,65
	Smooth, $R_e \leq 5 \times 10^5$	1,20
	Rough, all R_e	1,05
	Covered with ice, all R_e	1,20
R_e Reynolds number		

Table 9.3- Shape Coefficients, Cs, for perpendicular wind approach angles, Ref./7/

9.2.3.2 Wave and current loads

According to /50/, the inertial loading due to wave and current actions on the monopile will be considered for TP and Topside structural design and check. Maximum accelerations at El.+28 m LAT reported in Table 9.4 will be applied to permanent loads to evaluate wave inertial forces.

Case	Location	Elevation (LAT) [m]	Sea State	Max acceleration [m/s2]
Operational	Topsides CoG	28	Normal	0.67
Extreme	Topsides CoG	28	Extreme	1.61

Table 9.4 - Topsides Accelerations from Wave Loading on Monopile

For Caissons' design and check NSS and ESS waves as per Sec.6.13 will be considered for operating and extreme wave and current conditions. ISS will be considered for design against slamming.

9.2.4 Transient Loads

As outlined in ISO19902, ref./7/, Transient loads represent the actions from the transient situation being considered, including:

a. when appropriate, environmental actions;

b. when appropriate, a suitable representation of dynamic effects;

c. for lifting, the effects of fabrication tolerances and variances in sling length;

d. for loadout, allowances for misalignment and support settlements;

e. for transportation, any hydrostatic and hydrodynamic actions on the structure, including any inertial actions resulting from accelerations of the structure;

f. for installation, the lifting actions and hydrostatic pressure actions on the structure.

9.2.5 Accidental Loads

9.2.5.1 Dropped Object

The BLPH laydown areas on the upper and lower deck, shall be designed to withstand the following cases:

A)	Small container	1.25m x1.25m	weight 3.5t	drop height 5m;
B)	Medium container	2.00m x 2.50m	weight 5.0t	drop height 5m.

The items are considered dropped from 5 m height over one beam of laydown area for case A), while two beams are involved by the drop of medium container.

9.2.5.2 Blast Loads

Blast pressure provided by Safety studies, ref. /33/, will be applied on the interested structural elements as distributed load. DAF will be defined based on the blast explosion duration and the natural period of the loaded structure (T_n).

Refer to report, Ref. /36/, for the structural checks.

9.2.5.3 Ship Impact

The effects of unintended (accidental) ship impact have been investigated for the BLPH, Ref. /52/.

For the Detail design of the structures in the SoW, it will be verified that Topside equipment is designed to cope with the accelerations that occur due to the ship impact. The accelerations are given in Table 9.5.

For the topsides design these accelerations will be compared against sea transport and in-service wave accelerations to evaluate which governs.

For the TP in-place condition the maximum acceleration at the topsides CoG is considered appropriate in order to mobilise all the topside mass. Operating wind loads shall be considered concurrently. Possible impact of larger ships directly on the TP support arms or topsides will not be considered.

Location	Elevation (LAT)	Min acceleration	Max acceleration	Min displacement	Max displacement
	[m]	[m/s^2]	[m/s^2]	[mm]	[mm]
Interface level	17.05	-2.3	3.0	-11	10
Topsides CoG	28.00	-1.1	1.2	-10	11

Table 9.5 - Topsides Accelerations and Displacements from Ship Impact

10 STRUCTURAL ANALYSES

10.1 NOT-TO-EXCEED WEIGHT (NTE)

According to FEED, Ref./40/, the following limits will be considered:

- Not-to-Exceed weight (Operating condition) : 2400 mT including TP;
- The centre of gravity shall remain within a 1.5 m radius of the monopile centreline;
- Maximum vertical CoG position El. +28.0 LAT

NTE weight includes the Topside operating weight and includes the snow and ice accumulations and live loads on laydown area. Individual component NTE weights may also be set for the installation operations when considering available installation vessels.

The NTE weight relates to the maximum supportable operating weight and is dictated by the monopile capacity.

The topsides and TP will be fabricated and lifted separately. 'Leg green' is provided in the TP receptacles which can be cut and removed so that the topsides is level, even if the monopile is not.

Regular weight reports, Ref./27/, will be issued during detail design, reporting the weight and CoG assuming the topsides and TP are fabricated and lifted separately.

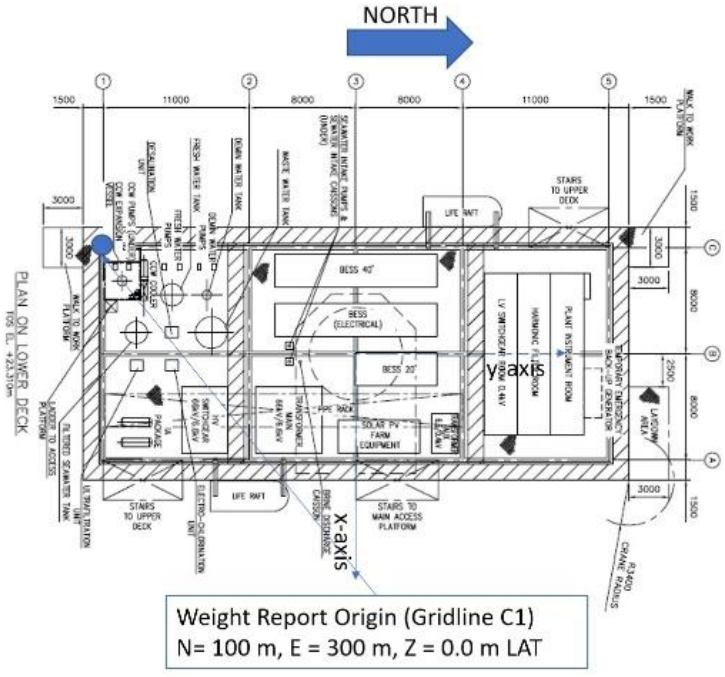


Figure 10.1 - Weight Reporting Origin

10.2 IN-SERVICE ANALYSIS

10.2.1 Topside and TP

The Topsides analysis will consider permanent, variable and environmental loads (operating and extreme wind and wave inertial loads).

Topsides support is provided by 4 leg supports on the ends of the TP arms. The pins have 'leg green' which is provided to ensure the topsides sits at the correct elevation and is level after installation – see Sec.6.12. Topside In-Service applicable design scenarios (operating and extreme) and respective LRFD partial action factors for both ULS and SLS are reported in Sec.8.

Wave run-up loads on the monopile are considered in /46/, /47/, /48/ and /53/. The loads and pressures on the WTG platform at Elevation (El.) +16.910 m Lowest Astronomical Tide (LAT) (top of grating) have been calculated. While these documents are detailed and comprehensive there is, unfortunately, insufficient information to extrapolate wave run-up loads and pressures to structural elements at higher levels. Hence, wave run-up loads on the support arms for Topsides (bottom of steel El.+19.210 m LAT) will be ignored in global analyses, considering the shield effect of main access platform at El.+17.00. The underside of these beams is considered sufficiently high that although they may be subject to wave pressure from wave run-up in extreme waves it will be relatively small compared to the Topside support load and it acts predominantly vertically, acting against gravity. Refer to Section 6.11.

Ship impact loads on the monopile generate accelerations on the Topsides – see Sec. 9.2.5.3. For the topsides design these accelerations and resulting inertial forces will be compared against sea transport and in-service wave accelerations to evaluate which governs. In case inertial load from ship impact acceleration will govern the design, a global static analysis with ship impact load combinations will be performed and included in the in-service design report. The ship impact inertial loads will be combined with Permanent, Variable and Operating Wind loads according to partial action factors reported in Table 10.1.

Similarly blast loads will be considered in conjunction with normal operating loads on the Topsides.

Load Condition FD	Permanent G	Variable Q	Operating Wind E_o	Inertia D
Ship Impact (ULS)	1.0	1.0	1.0	1.25

Table 10.1 – Ship impact Partial Action Factors

10.2.2 Caissons

A stand-alone structural SACS model of the CF will be developed, according to sec.9.1. Caissons' marine growth weight and buoyancy will be included in the structural self-weight, since automatically evaluated by SACS™ based on the input marine growth profile and density. Analysis will consider permanent, variable and environmental loads (extreme wind and wave inertial loads) according to ULS.

Pipes will be assumed fully corroded within splash zone and velocities enhanced to propre consider the presence of the monopile.

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10.3 LOADOUT

Loadout of the TP from the quayside to the barge will be by (SPMTs) supporting the TP by upper grillage, arranged to provide a statically determined support scheme.

Loadout of the topsides will be by hydraulic trailers (SPMTs) directly supporting, thought Spreader beams, the lower flange of Lower deck main beams arranged to provide a statically determined isostatic support scheme. Caissons will be loaded out by yard Crane.

10.3.1 Analysis Methodology

The structure to be loaded shall be designed to withstand the static and dynamic loads, support conditions (uneven support level), environmental loads experienced during the load out operation.

10.3.1.1 Weight and CoG

The loadout analysis shall consider the following contingencies depending on the weight reference:

- Loadout analysis shall be based on object weight and Center of Gravity derived from the weight control report;
- A global contingency corresponding to the accuracy of the weighing system will be applied to weighed weight (with a minimum of 3%);
- If the object will not be weighed (e.g. caissons) a contingency factor of not less than 1.10 shall be conservatively applied to the calculated weight.

The loadout manual shall be prepared and updated after final object weighing to consolidate the ballast procedure.

10.3.1.2 Design Cases

The SACS™ model, used for the load-out analysis, will be derived from the one used for the In-service analysis, properly modified to suite this temporary phase. To properly model the items restraint conditions during loadout, the item models shall include upper grillage if present. In order to take into account the proper behaviour of the items on the SPMT system, dedicated load cases for each hydraulic system will be defined. For each hydraulic circuit the total amount of load will be split in the contact points between the SPMT and the grillage.

The partial factor to be used are those listed in Table 8.6.

10.4 SEA TRANSPORTATION ANALYSIS

The transportation analyses will be performed as a two-step analysis with different restraint conditions, ensuring that the whole mass is reacted by the pre-installed vertical supports only, while dynamic loads both by sea fastening members and vertical supports.

The topside will be transported horizontally on 300 ft cargo barge together with Transition piece and caisson frame. It will be supported such as to minimize the effect of the barge displacement on the item and its supports. Therefore, the barge flexibility is neglected.

300ft barge has the following main dimensions:

- Length: 91.4 m
- Breadth 27.40 m
- Side height: 6.10 m

Partial action factors for pre-service analyses as per Table 8.6 will be considered.

The centre of rotation is assumed to be at the centre of the barge and eight (8) basic combinations of inertia loads will be considered in the design (± P ± H and ± R ± H).

Transient action considered for the transportation includes:

- Wind loads (1yr wind 5 sec gusts at platform location);
- Inertia forces coming from the motion criteria, defined in DNV, ref. /12/, for unrestricted world-wide tows (see /12/).

Item	Amplitude	Period
Roll	20 degrees	10 seconds
Pitch	12.5 degrees	10 seconds
Heave	0.20 g	

Table 10.2 – DNV - Default Motion Criteria

The Topsides SACS™ model, used for the transportation analysis, is derived from the one used for the in-service analysis, properly modified to suite this temporary phase.

Upper grillage and sea fastening will be included to represent the actual supports condition at the barge.

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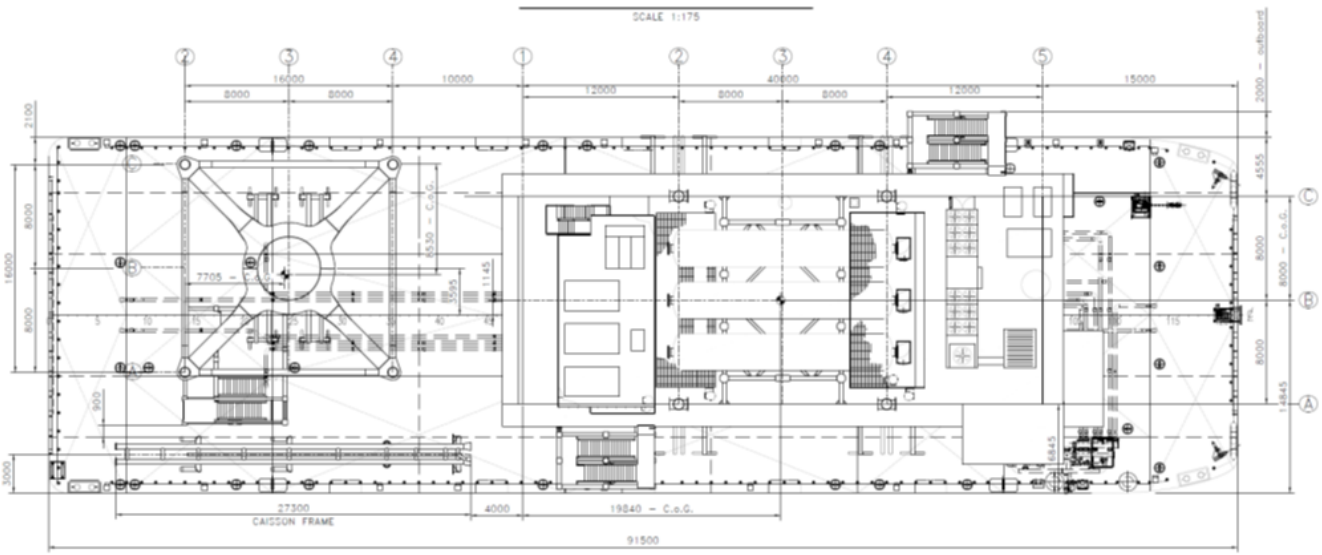


Figure 10.2 – 300 ft barge layout

10.4.1 Towing Route

The expected tow duration is about 30 days (including 3 days contingency) with departure date planned in mid of May 2025. The Metocean study performed by Fugro, Ref./56/, is used as a provision of the data. The report contains metocean data along 3 different routes to Tyra field, located in North Sea; one of these routes (North route 2) covers at 95% the route – Ravenna to WTG location and has been considered appropriate for the scope of the analysis. The route is shown in the following figure.



Figure 10.3 – Voyage Route – Sectors of North route – Route 2

The route is divided into 7 sectors with different wave data. For each sector corresponding DNV navigation zones were defined, Ref./56/.

Table 1.1.3 Transportation route 2 from Ravenna to Tyra field

Sector	1	2	3	4	5	6	7
Latitude	42.5°N	38.5°N	38.0°N	36.0°N	43.5°N	48.5°N	56.0°N
Longitude	16.0°E	19.0°E	5.0°E	10.5°W	10.5°W	5.5°W	2.0°E
DNV	27	27	26	25	16	17	11

Table 10.3 – Sectors coordinates and corresponding DNV zones

Voyage from Ravenna to Tyra Field						
Sector	Distance [nm]	Progres. Distance [nm]	Distance [%]	Average Speed [knots]	Duration [h]	Duration x 1.5 [h]
1	192	192	0.078	6.00	32.0	48.0
1	140	332	0.057	6.00	23.3	35.0
2	140	472	0.057	6.00	23.3	35.0
2	271	743	0.109	6.00	45.2	67.8
3	496	1239	0.200	6.00	82.7	124.0
4	255	1494	0.103	6.00	42.5	63.8
5	346	1840	0.140	6.00	57.7	86.5
6	279	2119	0.113	6.00	46.5	69.8
7	357	2476	0.144	6.00	59.5	161.3 (3 days added)
totals	2476		1.000		412.7	691.0

Table 10.4 – Sectors of the Voyage Route

10.5 LIFTING ANALYSES

Lifting analyses for the items will be performed to ensure that the structures are capable of satisfactorily withstanding all imposed loads during the offshore lifting. The linear analyses will be performed using the SACS software, Ref./57/.

Partial action factors for pre-service analyses as per Table 8.6 will be considered. Checks will be performed against ISO-19902, Ref./7/ for both structural members and joints.

The following DAF factors will be used for the lifting analysis to account for the related actions, according to DNV code, Ref./12/. The DAF's are dependent on the lifted weight as shown in Table 10.5.

Static Hook Load (SHL) tonnes			DAF
3	SHL	100	1+0.25√(100/SHL)
100	SHL	300	1.25
300	SHL	1000	1.20
1000	SHL	2500	1.15
	SHL>2500		1.10

Table 10.5 - DAF's for Offshore Lift (in Air)

The SHL should include the rigging weight. Limiting seastates are vessel dependent and will be defined by the Transport & Installation (T&I) contractors. The lifts are assumed to take place in calm conditions (low seastates and winds). Wind loading on the lifted structure will be ignored. The movements of the crane as a result of waves and wind will be included using a DAF.

Padeyes will be designed to take an additional lateral force of 3% of the sling force acting at the centreline of the shackle pin. In the case of doubled slings around a trunnion, a distribution of sling load of 45% / 55% between two slings will be applied to account for friction around the trunnion.

10.5.1 Topside Lifting Analysis

The Topsides is considered to be lifted with sheerleg crane single vessel, using spreader beam. The SACS models for the Lifting analysis is derived from the one used for the In-Service analysis, completed with hook joints and sling members to simulate the lifting configuration. The structural analysis will consider the pre-service gross weight, inclusive of structural weights, equipment dry weights and temporary installation aids (rigging, walkways, platform, etc.).

Lifting trunnion will remain on structure for decommissioning.

CoG shifting envelope as presented in 9.2.1.4 will be taken into account in structural analyses.

10.5.2 TP Lift Analysis

The TP will be lifted using the same crane as the topsides. It will be lifted using 4 slings rigged to the top of the TP. The padeyes will be left on the TP for decommissioning.

10.5.3 Lift factors

The following factors will be applied following the DNV code, ref. /12/.

10.5.3.1 2-hooks lift factors:

According to section 16.2.4.4 of ref. /12/ for multi-hook lift carried out by the same sheerleg crane vessel the more onerous condition of:

- Tilt of 1.5deg
- Hook elevation difference of +/- 0.5m

Will be used as static hook load.

10.5.3.2 YAW factor:

Assumed 1.0 according to section 16.2.4.5 of Ref. /12/.

10.5.3.3 SKL (skew load) factor:

- 1.10 for Topside based on dual crane lift configuration including 2 spreader bars.
- 1.25 for TP.
- 1.00 for caissons based on 3 sling arrangement.

10.5.3.4 Consequence factors:

The following factors will be applied following the DNV code, ref. /11/:

- 1.30 for lift points, spreader bars and rigging.
- 1.30 for non-redundant members framing into the lift point.
- 1.15 for load transferring redundant members framing directly into the lift point.
- 1.00 for all other members of the lifted structure.

10.6 TOPSIDE SETTING ON THE TP

10.6.1 Introduction

Landing of the topside onto the TP is not considered a governing design condition for the topsides as the transport is considered more onerous, being performed using the same 4 support points. Despite of that, for Topside and the TP design, two phases of installation are considered: the initial guide impact case and the landing case on the TP.

10.6.1.1 Primary Engagement (Guide forces)

The TP pin and Topside leg is considered a primary 'pin / bucket' guide, two long guides will be used at TP support on Row A, and two short guides on Row C.

Design loads are (primary guide):

- Fh = 5% of the static hook load (horizontal):
- Fv = 10% of the static hook load (vertical force to be combined with Fh to exacerbate applied loads).

For secondary guide forces can be reduced to 50%.

Nominal clearance between guide and bucket shall be <= +/- 15mm.

10.6.1.2 Set-down Forces

Forces acting on the supports due to placement of the topsides on the TP shall be calculated by distributing the module design weight proportionately to their distance from the CoG. Any two diagonally opposite supports shall be designed to withstand 100% of the design weight.

10.6.2 LRFD Partial Load Factors

In line with guidance in DNV-ST-N001, Ref./12/, for LRFD analyses relating to installation forces, a partial action factor of 1.3 shall be applied to all loadings.

10.6.3 Item movement

The maximum item movement relative to the target structure during installation should be according to Table 10.6:

Structure movements	
Vertical movement	+/- 0.75m
Horizontal movement	+/- 1.50m
Longitudinal tilt	+/- 2deg
Transversal tilt	+/- 2deg
Plan rotation	+/- 3deg

Table 10.6 - DNV module movements, section 16.14.2.1 ref. /12/.

10.7 TRANSPORTATION FATIGUE ANALYSIS

The Transportation Fatigue analysis will be performed for the Topside structure.

The spectral fatigue analysis will be performed using SACS, Ref./57/, considering the scatter diagrams, correlating the significant wave height and period with respect to transport heading for the route sectors, and the barge or vessel RAOs, sec.6.14.

The Topside structural model in SACS will be the same used for the structural Transportation analysis.

The structural analyses will be based on the computation of the stress transfer functions at each point of interest and assessment of fatigue damages.

The evaluation of fatigue cumulative damage related to each stress range will be calculated as the ratio of the occurrence of that stress range respect to the corresponding allowable number of occurrences given by the selected S-N curve.

The fatigue damage will be calculated following the Miner-Palmgren theory, i.e. as the sum of the cumulative damages due to all pertinent fatigue stress ranges. The fatigue life is then defined as the inverse of the fatigue damage.

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The Topside analysis will be performed considering DFF equal to 3.0 for the deck main joints and equal to 2.0 for the other joints.

Fatigue damages at tubular joint connections will be evaluated using the Efthymiou formulation for calculation of the stress concentration factors and the ISO TJ S-N curve, Ref/7/.

In-line of topside legs welds will be performed considering the ISO S-N curve F2 for one side weld, ISO E for double side weld and ISO C for weld cap grinded-flush, Ref/7/.

For beam connections, ISO E S-N curve will be used for double side welds, F2 curve will be used for single side welds, C curve will be selected for ground flush connections, Ref/7/.

F2, TJ or C curves are used also for pipe to beam connections (including leg to beam joint, bracing to beam and column to beam), Ref/7/.

In-line tubular joint and beam connections fatigue damages will be evaluated according to SCFs defined in DNV formulas, Ref./15/.

10.8 LOCAL DESIGN CHECKS

10.8.1 Joint Design

The checks of all the tubular connections will be covered by the global structural analyses as described in the previous sections (according to ISO 19902, Ref/7/) and they will be included in the corresponding analysis reports.

The not-tubular primary joint design will be performed using proven and sound engineering practice.

Connections between legs or columns and horizontal beams will be arranged with continuous plates in correspondence of the incoming beam flanges and webs and suitably reinforced with half or fourth pipe stiffeners, avoiding local checks. Where this arrangement is not applicable, the column will be the passing through elements.

Tubular braces will be fully backed up by elliptical stiffeners. Where required cover plates will be added to increase the shear capacity of the section between two adjacent braces. The shear checks of the beam joints will be covered by SACS including additional joints. All connections will be welded connections except where required for operating, handling or maintenance reasons.

For connections between legs and horizontal beams the leg will be reinforced by rings placed in correspondence of the flanges of the incoming beams and the check consists in the check of the ring. The checks will be performed considering the member end forces resulting from the topside structural analyses using a post processor program, including both the in-service and pre-service conditions and considering the joint geometry. Finite element models could be developed, if necessary, for main joints whose checks resulting from the above defined procedure will not be satisfied or that are not included in the above types (see next section).

10.8.2 FE Analyses

Finite element analyses will be developed, if required, to checks local details and/or joints whose geometry results very complex, with a multi-axial state of stresses than cannot be modeled with beam elements. FE analyses will be performed in ANSYS, Ref. /58/.

Materials will be defined considering linear or non-linear constitutive laws. Loads will be derived from the global structural analyses and the results will be evaluated in terms of Von Mises stresses considering a material factor of 1.0, according to ISO Ref./7/ :

$$\sigma_{vm} < \frac{F_y}{\gamma_{M0}}$$

Excursions in the plastic field will be accepted up to a plastic strain of 5%, ISO Ref./7/.

10.8.3 Miscellanea secondary structures

The local checks of the secondary structures will be performed according to EC3, ISO and API codes, Refs. /7/, /10/ and /11/, through handmade calculations or dedicated SACS models considering the applicable loads. They will include stairs and walkways, monorails, hatch covers. Walkways will be checked considering the live loads from Sec.9.2.2.2.

10.8.4 Dropped object

The structural checks will be performed according to DNV-RP-C204, Ref./16/ with the actions presented in section 9.2.5.

10.8.5 VIV checks

Vortex shedding checks will be performed for topside slender critical members only, where applicable, to assess that no unacceptable dynamic oscillation due to wind can occur.

Caissons will be checked for VIV with reference to extreme wave and current condition. The check will be performed according to DNV-RP-C205, ref. /17/.

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11 INSTALLATION OPERATIONS

The offshore operations shall be performed according to DNV–ST-N001 code, Ref./12/, considering the Company DEP 37.81.10.19-Gen and DEP 37.91.10.11 respectively for installation and mooring as additional guidelines, Ref./25/ and /26/.

11.1 MONOPILE

The monopile has been installed in Jan 2023 and surveyed by CPY on 29/05/2023.

11.2 TRANSITION PIECE

A lift assessment of the TP shall be performed using the Heavy Lift Vessels (HLVs) sheerlegs type using PS crane, with 4 slings running in a single hook configuration.

HLV will be on anchors.

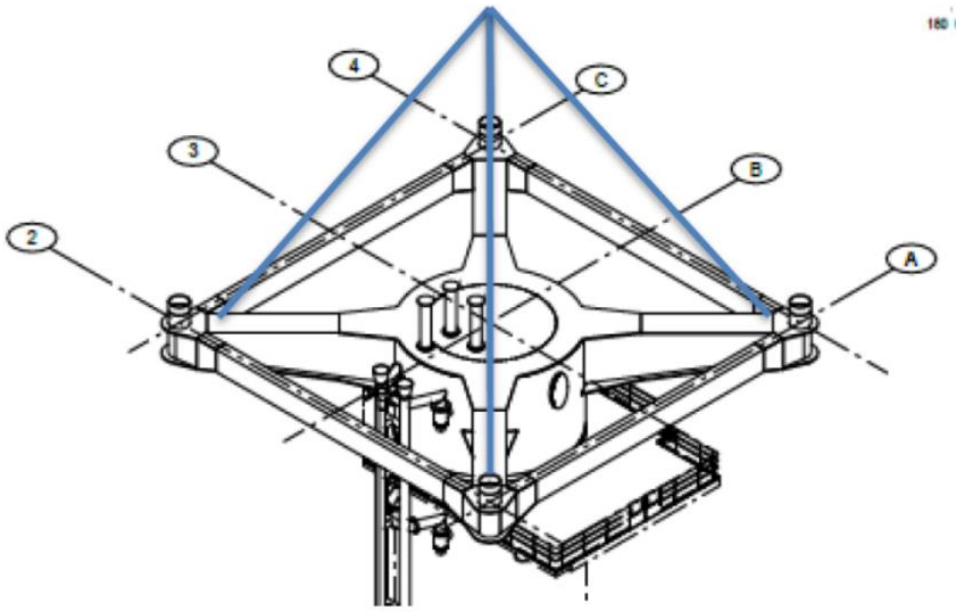


Figure 11.1– TP Offshore Lift Rigging Arrangement

The TP will be bolted to the top of the monopile.

11.3 CAISSONS

The caisson frame will be installed after the TP and before the Topsides using single hook lift point configuration in combination with steering lines and stubbing guides.

11.4 TOPSIDE

HLV will make use of its lifting beam in a dual crane lift configuration.

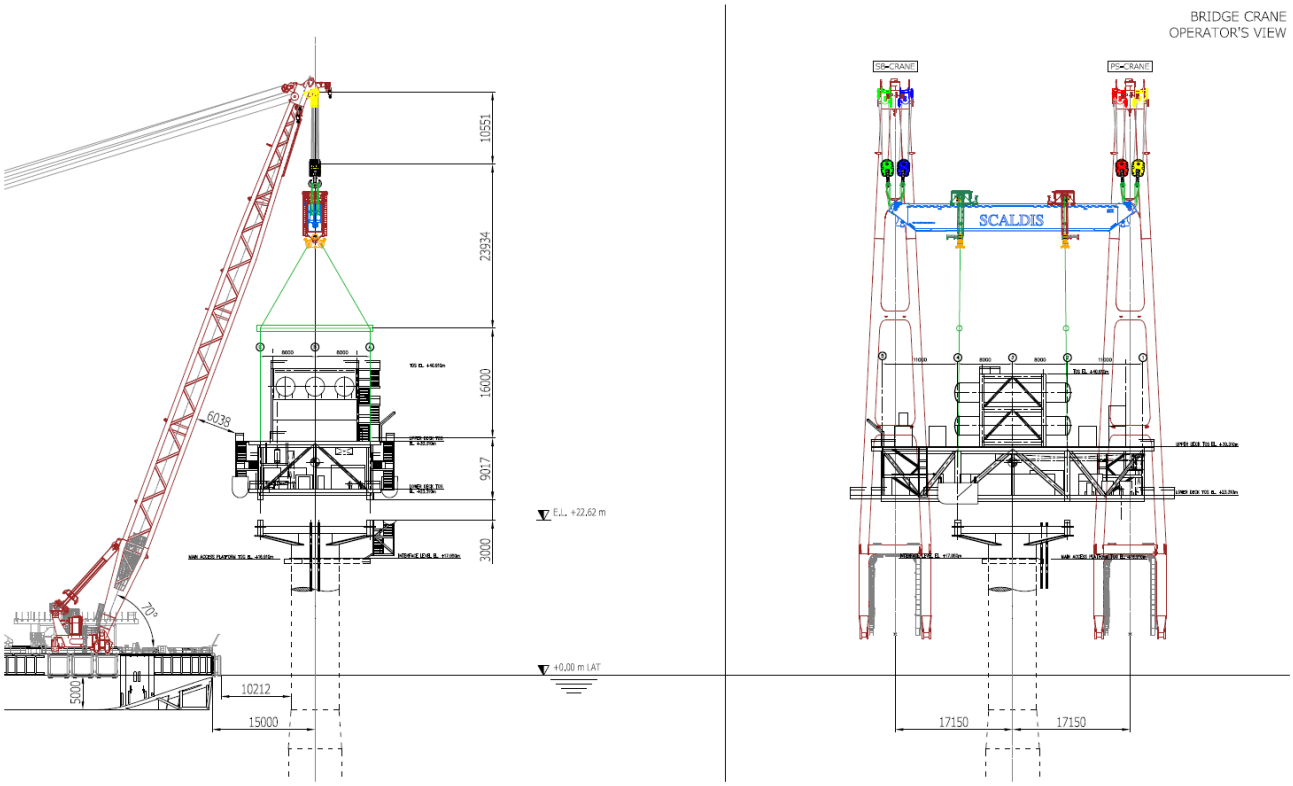


Figure 11.2– Topside Offshore Lift Rigging Arrangement

The BLPH topsides shall be constructed, commissioned, and tested onshore, and subsequently shipped and lifted onto the monopile TP at its offshore location. After the installation offshore is completed, the BLPH platform will be fully commissioned, preferably following connection to the floating solar PV.

11.4.1 Lift Assessment

Depending on the item to be lifted, the rigging configuration itself will be whether or not pre-rigged. The following pre-rigging conditions:

- TP: Rigging pre-rigged at the transport barge
- Caisson structure: Rigging pre-rigged at the transport barge
- Topside: Rigging pre-rigged on HLV by Contractor

Rigging weights shall be considered as follows, based on offshore contractor guidance:

- 5% of static hook load for single point lifts;
- 7% of static hook load for dual crane lifts with a spreader bar;

The T&I contractor shall consider the actual weight during the preparation of final Lift plans.

11.4.2 Installation Assessment Design Criteria

The maximum operational conditions for HLV Gulliver during heavy lifting operations are:

- Significant Wave Height ≤ 1.0 - 1.5m
- Swell period < 6 sec
- Wind speed ≤ 6Bft (12m/s)
- Visibility > 500m

The maximum operational conditions for HLV Rambiz during heavy lifting operations are:

- Significant Wave Height ≤ 0.75m
- Swell period < 5.5 sec
- Wind speed ≤ 6Bft (12m/s)
- Visibility > 500m

The operational criteria consider the clearance around lifted object and crane boom/vessel hull.

Crane Vessel	HLV to Monopile Underwater Clearance, m	HLV to Platform Above Water Hull / Crane Clearance, m	Crane Boom to Lifted Object Clearance, m
On Anchors	10.0	5.0	5.0

Table 11.1 – HLV Operational Clearances

ATTACHMENT A – MATERIAL SELECTION ACCORDING TO DNV CODE

The DNV code, ref. /13/, section 4.6 defines the steel materials, allowing for EN codes by using equivalent steel grades.

The table below has been prepared following the flow chart shown in Figure 4-1 of Ref. /13/.

			DNV-ST-0145, 2021				
Design Class	**Complexity**	**SQL**	**Classification of Structural members (DNV-OS-C101)**	**Hot rolled bar (DNV-ST-0126)**	**Plates and Welded tubular**	**Sections**	**SMLS pipes**
DC1 TP to monopile flange connection	High	**I (Z properties)**	**Special cat. "S"**	NV E420 (3)(4)			
DC1 Complex lift-nodes and element connection to topside, Lift points, main complex joints	High	**I (Z properties)**	**Special cat. "S"**		NV E420 / NV E460 (3)(4)		
DC2 simple joints, primary deck girders and beams of Topside and TP, column and diagonal to lift points, main columns, Davit crane support, Bolts	Low	**II**	**Primary category members "1"**		NV D420 (3)(4)		
DC3 all other columns and diagonals, stiffeners, Secondary deck beams, Grillage and sea fastening	High	**II**	**Primary category members "1"**		NV F36 (4)	NV F36 (4)	NV F36 (4)
DC4 Equipment and vessel supports, modules supports, equipment fixing bolts	Low	**III**	**Secondary category members "2"**		NV A36 / AW 36	NV A36 / AW 36	NV A36 / AW 36
DC5 floor plating, grating, Walkways, stairs, handrails	Any	**IV**	**Non structural members (NS)**		NV A NV A27S	NV A NV A27S	NV A27S

(3) Z35 option

(4) certification and test temp. for artic areas

ATTACHMENT B - FOUNDATION AS-BUILT SURVEY CW70 HNA6

144671-VOOW-GEN-SUR-REP-0074 Four

ATTACHMENT C - POSITIONING REPORT INNOVATION AT CW70 HNA6



144671-DEME-TF-IN
S-ITR-1270 Positioni

ATTACHMENT D – WTG COORDINATES AND CABLE ROUTING



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