Hornsea Project Four: Preliminary Environmental Information Report (PEIR)

Volume 5, Annex 1.1: Marine Processes Technical Report

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Glossary

Term	Definition
Amphidrome	A nodal point with minimal tidal range.
Cumulative effects	The combined effect of Hornsea Four in combination with the effects from a number of different projects, on the same single receptor/resource.
Far-field	An area remote from the near-field which is connected by a pathway
Hornsea Four	The proposed Hornsea Project Four offshore wind farm project; the term covers all elements within the Development Consent Order (i.e. both the offshore and onshore components).
Inshore	Between the nearshore and offshore. Generally, an area with more shelter than the offshore and where some coastal influences can still be expected
Isobath	A seabed contour commonly referencing chart datum.
Long-term	Of several years or decades, accounting for year to year variations
Maximum Design Scenario	A description of the range of possible elements that make up the Hornsea Four design options under consideration, as set out in detail in the project description. This scenario is used to define Hornsea Four for Environmental Impact Assessment (EIA) purposes when the exact engineering parameters are not yet known. This is also often referred to as the "Rochdale Envelope" approach.
Megaripples	A series of mobile bedform formations of sands with crest to crest wavelengths between 0.5 to 25 m
Mixed layer depth	Depth of surface mixed layer above density stratification formed by thermocline or halocline, if present
Near-field	The area immediately adjacent to a source of change, such as around the base of a wind turbine foundation
Nearshore	Generally, a shallow water area close to the coast
Offshore	Generally, a more exposed and deeper water area away from any coastal influence
Sandwave	A mobile bedform formation of sands with a crest to crest wavelength greater than 25 m, most likely interspersed with megaripples and with a higher crest height
Short-term	A sub-set of a repeating cycle, e.g. likely to be a few days, weeks or months but much less than a year

Acronyms

Acronym	Definition
ABS	Acoustic Back-Scatter
AfL	Area for Lease
AODN	Above Ordnance Datum Newlyn
BERR	(former) Department for Business, Enterprise and Regulatory Reform
BODC	British Oceanographic Data Centre
CBRA	Cable Burial Risk Assessment
CCO	Channel Coastal Observatory
CD	Chart Datum, the vertical datum of a navigation chart
COWRIE	(former) Collaborative Offshore Wind Research into the Environment
DECC	Department of Energy & Climate Change
D_{50}	Sediment diameter representing 50% by mass larger and 50% smaller
D_{90}	Sediment diameter where 90% of the sample by mass is smaller
EIA	Environmental Impact Assessment
EIA Report	Environmental Impact Assessment Report
EMODnet	European Marine Observation and Data Network
EPA	Environmental Protection Agency
ERYC	East Riding of Yorkshire Council
HAT	Highest Astronomical Tide
HDD	Horizontal Directional Drilling
HVAC	High Voltage Alternating Current
HVDC	High Voltage Direct Current
H_s	Significant wave height (m)
IECS	Institute of Estuarine and Coastal Studies, University of Hull
LAT	Lowest Astronomical Tide
LSO	Long Sea Outfall
MDS	Maximum Design Scenario
MFE	Mass Flow Excavator
MHWN	Mean High Water Neaps
MHWS	Mean High Water Springs
MLD	Mixed Layer Depth
MLWN	Mean Low Water Neaps
MLWS	Mean Low Water Springs
MNR	Mean Neap Range
MSL	Mean Sea Level
MSR	Mean Spring Range
NCERM	National Coastal Erosion Risk Mapping
NPS	National Policy Statement
OBS	Optical Back-Scatter
ODN	Ordnance Datum Newlyn
OWPB	Offshore Wind Programme Board
PEIR	Preliminary Environmental Information Report

Acronym	Definition
SAC	Special Area of Conservation
SMP	Shoreline Management Plan
SNL	Sandia National Laboratories
SPM	Suspended Particulate Matter
SWAN	Simulating WAves Nearshore, a third-generation spectral wave model
TSHD	Trailing Suction Hopper Dredger
Tz	zero up-crossing wave period (s)
UKCP18	United Kingdom Climate Projections 2018
UKHO	United Kingdom Hydrographic Office
WTG	Wind Turbine Generator
WWII	World War Two

Units

Unit	Definition
km	kilometre
l	litre
m	metre
mg	milligram
mm	millimetre
m/s	metres/second
s	Second
°C	Degrees Centigrade

1 Introduction

1.1 Project background

1.1.1.1 Ørsted Hornsea Project Four Ltd. (the Applicant) is proposing to develop the Hornsea Project Four Offshore Wind Farm (hereafter Hornsea Four). Hornsea Four will be located approximately 65 km offshore from the East Riding of Yorkshire in the Southern North Sea and will be the fourth project to be developed in the former Hornsea Zone. Hornsea Four will include both offshore and onshore infrastructure including an offshore generating station (wind farm), export cables to landfall, and connection to the electricity transmission network. The location of Hornsea Four is illustrated in **Figure 1**.

1.1.1.2 The Hornsea Four Agreement for Lease (AfL) area was 848 km^2 at the Scoping phase of project development. In the spirit of keeping with Hornsea Four's approach to Proportionate Environmental Impact Assessment (EIA), Hornsea Four is currently giving due consideration to the size and location (within the existing AfL area) of the final project that will be taken forward to Development Consent Order (DCO) application. This consideration is captured internally as the "Developable Area Process", which includes Physical, Biological and Human constraints in refining the developable area, balancing consenting and commercial considerations with technical feasibility for construction. The combination of Hornsea Four's Proportionality in EIA and Developable Area process has resulted in a marked reduction in the AfL taken forward at the point of PEIR (**Figure 1**). The evolution of the AfL is detailed in **Volume 1, Chapter 3: Site Selection and Consideration of Alternatives** and **Volume 4, Annex 3.2: Selection and Refinement of the Offshore Infrastructure**. The final developable area being taken forward to application for development consent may differ from that presented in **Figure 1** due to the results of the EIA, technical considerations and stakeholder feedback.

1.1.1.3 Cooper Marine Advisors Ltd was commissioned by the Applicant to undertake a Marine Geology, Oceanography and Physical Processes assessment of the marine areas being developed for Hornsea Four and the surrounding areas. The assessment is developed using an evidence base approach drawing on previous studies of comparable projects in comparable offshore settings.

1.2 Marine Processes

1.2.1.1 The topic of Marine Geology, Oceanography and Physical Processes is also commonly referred to as "Marine Processes", or when issues pertain to the nearshore and coastline then the term "Coastal Processes" is also frequently used. The use of either term is intended to be inclusive of marine geology, oceanography and physical processes at either location. For convenience, the term "Marine Processes" is used in this document.

1.2.1.2 A baseline assessment of marine processes provides an understanding of how the seabed and coastline respond to driving "metocean" conditions, such as winds, waves and tides. The morphological response of the seabed and coastline is also linked to understanding their potential erodibility as well as any geological constraints which may act as moderators to rapid change.

1.2.1.3 The impact assessment of Hornsea Four considers how the proposed development may modify the baseline during construction, operation and decommissioning periods.

1.3 Report Structure

1.3.1.1 This Technical Report is supplementary to **Volume 2 Chapter 1: Marine Geology, Oceanography and Physical Processes** of the PEIR, and serves to provide a more detailed description of the approach, evidence base and results of the impact assessment, as well as any supporting technical material. This approach allows for a more proportionate chapter, in line with Hornsea Four's proportionate approach to EIA.

1.3.1.2 The report is structured as follows:

- **Section 1** introduces Hornsea Four and the topic of marine processes;
- **Section 2** outlines the assessment approach and scope;
- **Section 3** provides a baseline review and identifies key receptors in the marine physical environment;
- **Section 4** considers the potential impact of the development; and
- **Section 5** offers a list of all the technical references informing this assessment.

1.3.1.3 The following technical appendices are also included for supporting information:

- **Appendix A** provides a review of comparable baseline conditions across the former Hornsea Zone;
- **Appendix B** sets out the primary baseline evidence; and
- **Appendix C** offers an evaluation of wave measurements from Hornsea Project One Offshore Wind Farm (hereafter Hornsea Project One) to investigate wave blockage across the constructed array.

2 Assessment Approach

2.1 Overview

2.1.1.1 The assessment is based on the "source-pathway-receptor" approach. This approach also helps confirm a relevant study area to extend across all locations with development activities that create potential sources of effects in the marine environment and the process influences which may link such effects via pathways to receptors. A baseline understanding is then established for this study area to act as the reference condition against which the scale of these potential effects can be determined. Both the baseline and the impact assessment are delivered using an evidenced-based approach

2.2 Source-pathway-receptor

2.2.1.1 A development activity which has the potential to create a physical change in the marine environment establishes a source; the origin of a potential impact. There are many different types of development activities which are planned to occur at different locations and at different phases in the Hornsea Four lifecycle (i.e. construction, operation and decommissioning periods). These sources are typically associated with site specific activities related to either seabed preparation, cable laying or the installation (and presence thereafter) of a large number of individual foundation structures. Each type of source may lead to a different type of local change in the near-field marine processes. All sources will occur within the PEIR boundary.

2.2.1.2 Where multiple options remain for project development activities the definition of sources for marine processes targets the relevant Maximum Design Scenario (MDS), determined from the relevant project parameters defined within the Project Description for Hornsea Four (**Volume 1, Chapter 4: Project Description**). The MDS envelope represents the conservative case of any of the design options with an alternative option to the MDS considered to have a lesser environmental effect.

2.2.1.3 Once a change in near-field marine processes has occurred (e.g. elevated levels of suspended sediment during seabed preparation activities) then the potential exists for that change to be transmitted beyond the source and to extend over a larger area; the far-field. The means by which any extended effect reaches a receptor sensitive to that change defines the pathway connecting source to receptor. The far-field can be expected to extend beyond the PEIR boundary. For example, the scale of tidal advection may have the potential to carry material in suspension past the PEIR boundary.

2.2.1.4 Receptors which are connected to a source effect via a pathway may be part of the marine physical environment, such as the Flamborough Front, or related to other ecological receptors associated with the marine environment. This report identifies the receptor features only related to the marine physical environment.

2.3 Establishing the study area

2.3.1.1 The marine processes study area encompasses the near-field sources created by any project activity that has a potential to disturb sediments or block waves and flow, and the pathways which have the capacity to extend effects from a source across a wider area (the far-field). In addition, where there are adjacent activities which may also create a similar type of effect over a similar period then this is also considered to be part of the study area in order that cumulative effects between such activities can be considered. In relation to Hornsea Four, these include the immediately adjacent offshore wind farms of Hornsea Project One and Hornsea Project Two Offshore Wind Farm (hereafter Hornsea Project Two) which may act cumulatively for potential blockage effects during the operational period and also possible landfall related issues with the installation of the Dogger Bank Creyke Beck Offshore Wind Farm (hereafter Creyke Beck) export cable.

2.3.1.2 Hornsea Project Three Offshore Wind Farm (hereafter Hornsea Three) is considered to be less relevant to the possible cumulative interactions because of the further distance from Hornsea Four, the flow and sediment pathways not passing between these two projects and that waves are mainly from the northerly sector. The additional moderation here is the final layouts and foundation types selected for both Hornsea Project One and Hornsea

Project Two utilise a fewer number of smaller diameter structures than the conservative cases considered in their respective EIA which reduces potential overlapping wake type effects between arrays. On this basis, Hornsea Three does not present a rationale for inclusion in the study area.

2.3.1.3 The study area is also divided into sub-areas which recognise the different types of project activity and different types of marine process environments. For example, the short-term installation activities at the cable landfall will occur in a shallow and sheltered nearshore environment which will be very distinct from construction activities required across the more exposed offshore array area where the placement of a large number of foundation structures may then create longer lasting blockage type effects. Accordingly, activities across the landfall, offshore export cable corridor (ECC) and the offshore array form the basis of describing sub-areas of the overall study areas.

2.3.1.4 **Figure 1** presents the marine processes study area for Hornsea Four, along with sub-areas established for the landfall, offshore ECC and offshore array. The offshore ECC and offshore array areas include buffer zones to represent a potential "zone of influence" for any sediment plumes that might be created within the main areas of activity. It is important to note that a zone of influence is not an area of impact. The buffer zones are scaled to represent the equivalent distance of tidal excursion on a mean spring tide, for the offshore ECC this is taken as a distance of 15 km based on the nearshore flows and for the offshore array area this is taken as a distance of 10 km representing weaker offshore flows. The wider study area aims to represent where changes in wave energy transmission might occur.

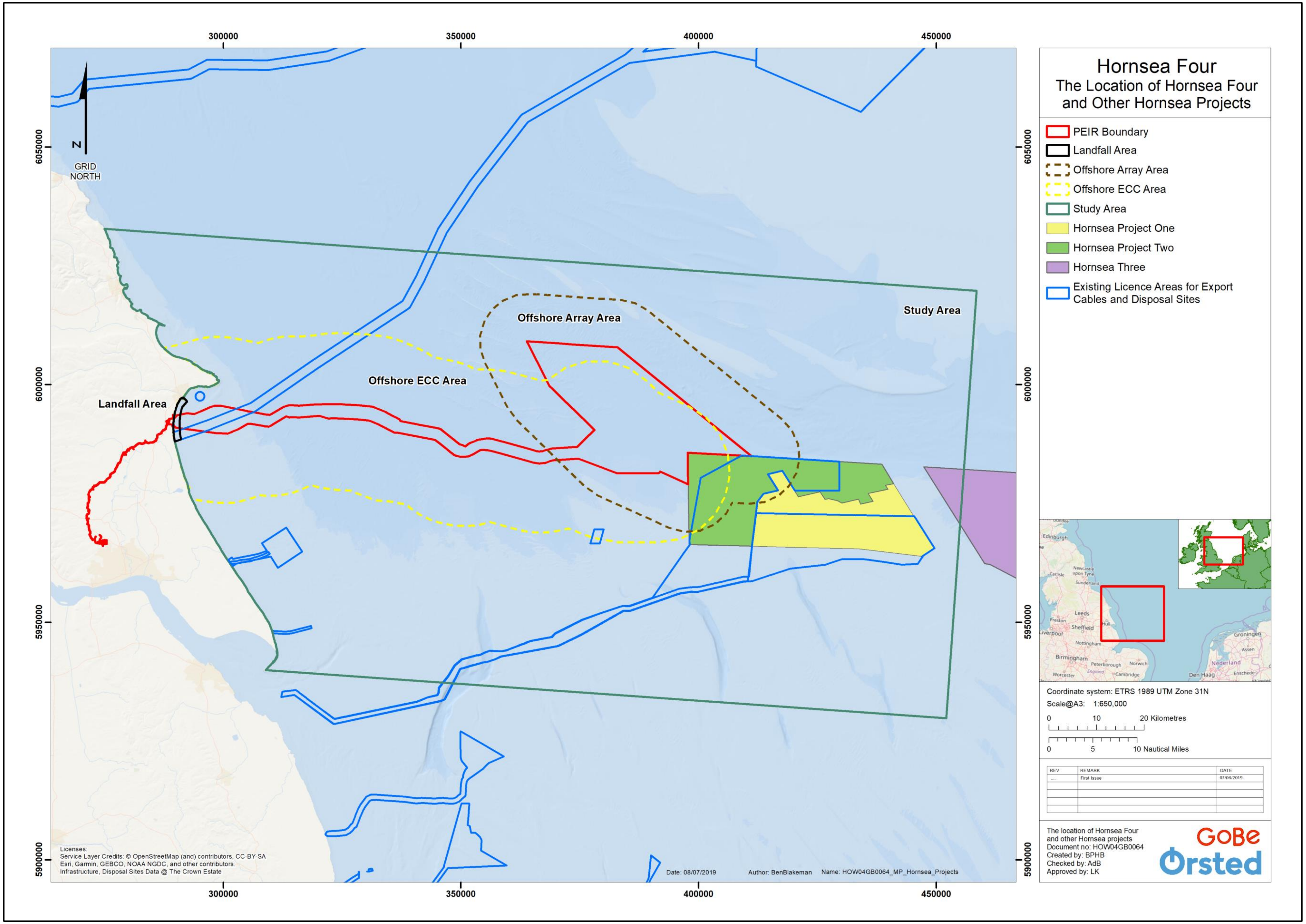

Figure 1: Marine processes study area and sub-areas (not to scale).

2.3.1.5 Importantly, sources which may occur in any sub-area may still have the potential to affect more remote receptors in other parts of the study area where there are connecting pathways.

2.4 Establishing the baseline

2.4.1.1 A baseline description of marine processes is established for the study area and each sub-area; landfall, offshore ECC and array area. This baseline represents conditions that are expected to prevail without any development taking place and with consideration of an equivalent period as the lease (i.e. 35 years). This description provides the reference conditions against which potential effects of the development are expected to occur. **Section 3** provides details for the baseline assessment.

2.5 Establishing the scope of the assessment

2.5.1 Issues scoped into assessment

2.5.1.1 The issues which have been assessed have been established from a full review of the Scoping Opinion received in November 2018 (The Planning Inspectorate, 2018) and are summarised in **Table 1.** These issues are identified as impact pathways and receptors and can be grouped by project phase and type of effect as either:

- Short-term (days to months) sediment disturbance events during construction, maintenance and decommissioning periods which may lead to sediment plumes of elevated suspended sediment concentration and the associated areas of the seabed with increased levels of deposition once the material settles out of the water column; and
- Long-term (several years) blockage related effects during the operational period of the windfarm which are due to foundation or rock berm structures being placed on the seabed which have a sufficiently large profile to individually and/or collectively interfere with waves or flows to develop wake effects, as well as interrupt sediment pathways.

Table 1: Summary of impact pathways and receptors.

Project Phase	Impact pathway and receptor
Construction	Sediment disturbance caused by seabed preparation activities (e.g. levelling around foundations, sandwave clearance for cable installation, etc.) which may lead to a requirement for removal of sea sediment and spoil disposal elsewhere creating elevated suspended sediment and potential smothering by deposition.
Construction	Sediment disturbance caused by activities that may lead to locally raised suspended sediment concentrations at source (drilling, cable laying, seabed levelling, etc).
Operation	Blockage of flows causing local (near-field) scouring around foundations (assumes scour protection is not pre-installed).
Operation	Blockage of flows from foundations interfering with far-field receptors, e.g. Flamborough Front.

Project Phase	Impact pathway and receptor
Operation	Blockage and modification to wave energy transmission and nearshore wave climate affecting coastal morphology, including cumulative effect with Hornsea Project One and Hornsea Project Two.
Operation	Blockage to nearshore sediment pathways from rock armouring over cables.
Decommissioning	Sediment disturbance during decommissioning activities that may lead to locally raised suspended sediment concentrations at source. Removal of foundations with cessation of blockage related effects on waves and tidal flows, reversing to a (future) baseline condition.

2.5.1.2 **Section 4** provides details of the impact assessment for each of the issues identified in **Table 1**.

2.5.2 Issues scoped out of assessment

Scouring around foundations

2.5.2.1 The option to place scour protection on the seabed prior to foundation installation would mitigate the scour process and the potential for seabed sediments to be locally eroded around any foundation. If this option is confirmed prior to DCO application, then assessment of local scour around foundations can be scoped out.

Changes to offshore sediment pathways

2.5.2.2 Previous impact assessments for Hornsea Project One (SMart Wind, 2013), Hornsea Project Two (SMart Wind, 2015) and Hornsea Three (Ørsted, 2018) have each indicated that impacts on sediment pathways are likely to be of minor adverse significance, at least for the offshore array areas. A subsequent moderation on this issue for Hornsea Project One and Hornsea Project Two is that the MDS option used in their respective assessments assumed a larger number of more closely spaced foundations with wider diameters (based on Gravity Base Structure (GBS) options in each case) than the final choices now installed for Hornsea Project One and planned for Hornsea Project Two which have both chosen a fewer number of smaller diameter monopiles spaced further apart.

2.5.2.3 Given the anticipated localised nature of the changes in tidal currents and waves for the Hornsea Four offshore array, there is expected to be an equivalent impact on offshore sediment pathways of minor adverse significance. Furthermore, Hornsea Four is situated updrift in the sediment pathway related to the Norfolk Banks Special Area of Conservation (SAC). On the basis of a proportionate approach, this issue of changes to offshore sediment pathways is therefore scoped out.

2.5.2.4 Changes to nearshore sediment pathways remains an issue for consideration for Hornsea Four, noting the specific comment from the Planning Inspectorate in the Scoping Opinion that sediment pathways should be scoped in from Smithic Bank inshore to the mean high water spring tide (MHWS) level (page 14 – 15 of The Planning Inspectorate (2018)).

2.6 Evidence-based approach

2.6.1.1 The assessment of Marine Processes adopts an evidence-based approach which is consistent with present best practice (COWRIE, 2009). The evidence-based approach was presented to the Marine Ecology & Processes Evidence Plan Technical Panel at Meeting 1 on 12th September 2018 (Ørsted, 2018). An overview of consultation carried out to-date is described in **Volume 1, Chapter 6: Consultation** of the PEIR, with consultation specific to marine processes set out in **Volume 2, Chapter 1: Marine Geology, Oceanography and Physical Processes**. This consultation included a series of three meetings with the Marine Ecology & Processes Evidence Plan Technical Panel.

2.6.1.2 The application of an evidence-based approach to offshore wind farms is now well-established and has been successfully demonstrated in several recent DCO applications, including Hornsea Three.

2.6.1.3 The evidence-based approach is most suited to an area of development which is already well provided with baseline data and information, and where assessment of comparable developments in comparable settings can be drawn upon to offer relevant evidence of the likely effects on the marine physical environment. In such situations, the need for additional baseline surveys and detailed numerical modelling is much reduced and existing assessments can be drawn on instead.

2.6.1.4 **Appendix A** provides a summary of the comparable conditions between Hornsea Four and the adjacent projects in the former Hornsea Zone to further justify the applicability of using the evidence-based approach. Of particular note is Hornsea Project Two which is the closest project to Hornsea Four with the most similar environmental conditions.

2.6.1.5 The baseline description is developed from existing data and information, as well as studies of equivalent projects (the evidence base). **Appendix B** identifies the main data and information sources which have informed this assessment and **Section 5** provides a reference list of the technical literature and reports. This evidence includes the extensive geophysical, benthic and metocean surveys which supported the characterisation of the former Hornsea zone (SMart Wind, 2012).

2.6.2 Baseline surveys

2.6.2.1 The evidence-base now includes the first phase of a new geophysical survey to supplement the previous zonal surveys and provide seabed mapping of sediments, bathymetry and bedforms. The first phase of the survey was completed in 2018 and included parts of the offshore ECC as well the offshore array. The second stage of this survey will complete the offshore ECC and is expected to be available to inform the EIA for submission of the DCO application.

2.7 Policy and Guidance

2.7.1.1 The assessment approach has been developed with consideration of the following policy documents, guidance notes and industry technical reviews:

- Turbidity due to dredging and dumping of sediments (van Rijn, 2019);

- Natural England Offshore wind cabling: ten years' experience and recommendations (Natural England, 2018);
- Technical Guidance – Environmental Impact Assessment of Marine Dredging Proposals (EPA, 2016);
- Environmental impact assessment for offshore renewable energy projects - Guide (BSI, 2015);
- Overview of the offshore transmission cable installation process in the UK (OWPB, 2015);
- Review of environmental data associated with post-consent monitoring of licence conditions of offshore wind farms (MMO, 2014);
- Offshore Wind Guidance Document: Oceanography and Sediment Stability. Development of a Conceptual Site Model (SNL, 2014);
- Guidelines for data acquisition to support marine environmental assessments for offshore renewable energy projects (Cefas, 2011);
- Overarching National Policy Statement for Energy (EN-1) (DECC, 2011);
- National Policy Statement for Renewable Energy Infrastructure (EN-3) (DECC, 2011)
- A Further Review of Sediment Monitoring Data (COWRIE, 2010);
- Coastal Process Modelling for Offshore Wind Farm Environmental Impact Assessment: Best Practice Guide (COWRIE, 2009);
- Assessment of the environmental impacts of cables (OSPAR, 2009); and
- Review of Cabling Techniques and Environmental Effects applicable to the Offshore Wind Farm Industry (BERR, 2008).

3 Baseline description of the study area

3.1 Overview

3.1.1.1 The baseline represents conditions that are expected to prevail without any development taking place and with consideration of an equivalent period as the 35-year lease. This baseline description provides the reference conditions against which potential effects of the development are expected to occur and to help determine the magnitude and duration of any impacts.

3.1.1.2 Of special interest in the study area are features of the marine environment which are regarded as potentially sensitive environmental receptors. Changes in the physical environment (sources) brought about by the development of Hornsea Four may create a pathway for an effect on a range of marine receptors, such as, benthic communities, fish and shellfish. There may also be pathways and effects on receptors associated with the marine physical environment. These receptors types are identified for each part of the study area, however, the presence of a receptor in either the landfall, offshore ECC or offshore array area does not limit that receptor from possible pathways communicating an effect arising from another part of the study area.

3.2 Landfall study area

3.2.1 General description

3.2.1.1 The landfall study area incorporates the proposed site for landfall works on Fraisthorpe Sands and extends north to the end of the beach at Bridlington Harbour. The area also extends south to incorporate the Creyke Beck landfall. The total longshore extent of the landfall study area considered for PEIR covers around 9 km. This extent may be reduced for the EIA phase where further refinements to landfall options are made.

3.2.1.2 The inshore extent is defined by the low-lying soft cliffs which tend to coincide with MHWS. The seaward extent is defined by the option for sub-tidal cofferdams to accommodate up to eight Horizontal Directional Drilling (HDD) exit pits (**Volume 1, Chapter 4: Project Description**). These pits may be around 1 km offshore and beyond the surf zone. Alternative options include open-cut trenching across the inter-tidal, a typical distance of around 200 km between high water and low water.

3.2.1.3 The general description of the landfall study area is an open inter-tidal sandy beach, backed by soft cliffs, gently shelving into a shallow sub-tidal environment. The sands can be thin in places exposing an underlying clay till. This environment mainly responds to wave driven processes which erode the cliffs and transport mobile sandy sediments along the beach. **Figure 2** provides a typical view of the intertidal area at the landfall.

Figure 2: View of intertidal area at landfall (from IECS, 2019).

3.2.2 Process description

Cross-shore profile

3.2.2.1 **Figure 3** presents a cross-shore profile for the landfall works area based on Channel Coastal Observatory (CCO) Lidar surveys (from 2017) and multibeam surveys (from 2014). A low-lying cliff backs the beach area with a height of around 7 to 8 m above Ordnance Datum Newlyn (AODN). The cliff has a near-vertical drop onto the beach with the MHWS effectively at the base of the cliff. The beach then shelves in a relatively uniform manner to low water over a distance of around 200 m. This gradient continues to around -10.35 m AODN (7 m below CD), a depth typically reached around 1.0 to 1.4 km from the base of the soft cliffs. From the seaward limit of the landfall study area, the seabed flattens out before shallowing over the southern part of Smithic Sands.

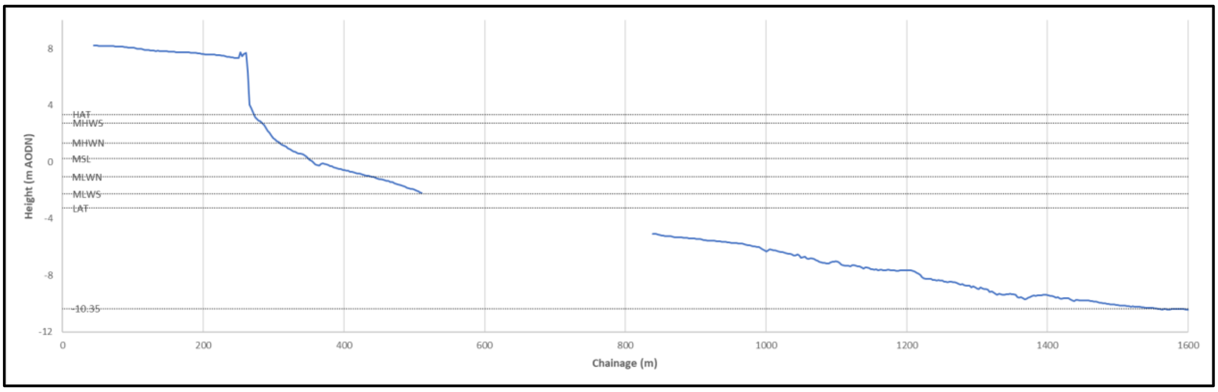

Figure 3: Cross-shore profile from landfall works area (based on CCO surveys).

Intertidal sediments

3.2.2.2 An intertidal walkover survey of the landfall works area was undertaken on 22nd March 2019 which qualitatively described beach material as coarse sands, and in places this thins to reveal hard boulder clay (IECS, 2019). Any trenching works on the beach are therefore likely to be into the underlying clay.

Subtidal sediments

3.2.2.3 The 2018 geophysical survey identifies the subtidal sediments as sand with patches of gravelly sand. In places, this cover of sand thins to expose underlying glacial till (stiff glacial till of Bolders Bank Formation) (Bibby HydroMap, 2019).

Water levels

3.2.2.4 The predicted tidal variation in water levels for the landfall study area is expected to be equivalent to Bridlington, a secondary non-harmonic port for tidal predictions, located to the north. Table 2 provides standard tide levels for this site based on Admiralty Tide Tables (UKHO, 2019). For reference, the correction between the sea datum of CD to the land datum of ODN is +3.35 m.

Table 2: Standard tidal levels for Bridlington.

Standard Tidal Level	Abbreviations	Relative to Chart Datum, CD (m)	Relative to Mean Sea Level, MSL (m)
Highest Astronomical Tide	HAT	6.7	3.1
Mean High Water Springs	MHWS	6.1	2.5
Mean High Water Neaps	MHWN	4.7	1.1
Mean Sea Level	MSL	3.6	0.0
Mean Low Water Neaps	MLWN	2.3	-1.3
Mean Low Water Springs	MLWS	1.1	-2.5
Lowest Astronomical Tide	LAT	0.1	-3.5
Mean Spring Range (m)	MSR	5.0	
Mean Neap Range (m)	MNR	2.4	

3.2.2.5 The tidal range on mean spring tides, and any higher tidal ranges, is sufficient to reach the base of the soft cliffs.

3.2.2.6 Over a 50-year period, mean sea level is expected to increase. UKCP18 provides climate projections for sea level rise up to 2100 based on different emission scenarios (representative concentration pathways). Based on the 50th percentile for low and high emission scenarios, an illustrative change in mean sea level after 50 years would be between 0.22 to 0.35 m. This effect would also redefine tidal levels presented in **Table 2** relative to CD and ODN, moving high waters further against the soft cliffs.

Water levels also vary under the influence of strong winds and atmospheric pressure variations leading to (non-tidal) surge effects. These effects can result in both positive and negative variations on the tidal level. The Environment Agency has produced a national dataset of design sea levels based on the analysis of Class A tide gauge data which incorporates the effect of surges (Environment Agency, 2011).

3.2.2.7

3.2.2.8 **Table 3** provides the extreme water level predictions up to the 200-year return period level for the landfall site (based on data for chainage 3796).

Table 3: Extreme water level return periods for landfall.

Return Period (years)	Extreme level, AODN (m)	95% confidence level (m) +/-
1	3.55	0.2
2	3.65	0.2
5	3.77	0.2
10	3.88	0.2
20	3.98	0.2
25	4.02	0.3
50	4.12	0.3
75	4.18	0.3
100	4.22	0.4
150	4.29	0.4
200	4.34	0.4

3.2.2.9 For context the HAT level of 3.35 m AODN (an event which is approximated by the vernal and autumnal equinox spring tides) is relatively close to the 1 in 1-year return period extreme water level of 3.55 m AODN. Positive surge events have the capacity to augment tidal levels and reach the base of the cliffs.

3.2.2.10 The last major storm surge influencing the North Sea occurred on 5 December 2013. This event produced a peak water level of 4.56 m AODN at Bridlington, comprising of a surge influence of 1.76 m above the predicted high tide level of 2.80 m AODN (ERYC, 2014). On the basis of the information provided in Table 3, this event would have a return period of around 1 in 1,000 years.

Waves

3.2.2.11 Waves shoal from the offshore ECC onto the shallowing sub-tidal and in very shallow water they typically break to form a surf zone. This process creates longshore (wave-driven) currents which are capable of transporting sandy material along the shore (longshore drift).

3.2.2.12 Smithic Sands and Flamborough Head both provide some local sheltering to the landfall area, especially for northerly sectors.

Sediment transport – longshore drift

3.2.2.13 The net annual longshore drift (sum of all drift rates and directions in a year) is effectively nil at the location of the landfall, with a balance of material transported to the north and south. South of Barmston, the coastline receives less sheltering from Flamborough Head (and Smithic Sands) leading to increased exposure to northerly waves which results in a progressively stronger net longshore drift towards Spurn Head (Pye & Blott, 2015). The area around Barmston can therefore be regarded as a drift divide for longshore sediment transport (**Figure 4**).

Pathways

3.2.2.14 The main process pathway in the landfall study area is wave-driven nearshore flows. Depending on the angle of approach, these wave-driven currents may drive sediment to the north or south along the beach.

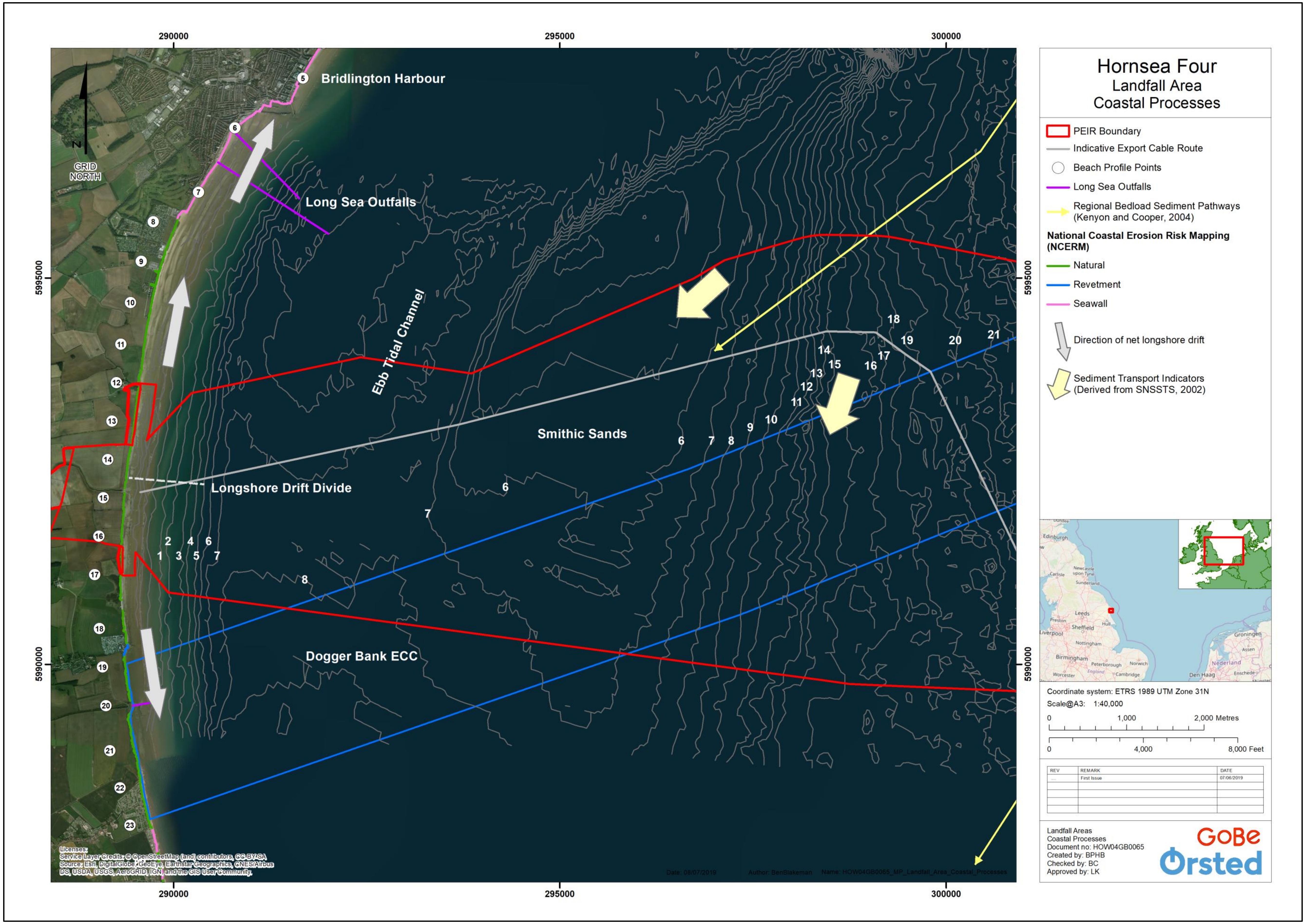

Figure 4: Landfall study area coastal process (not to scale).

3.2.3 Marine physical environment receptors – landfall study area

Holderness Coast

3.2.3.1 The main receptor extending north and south, and including the landfall study area, is the Holderness Coast. The coastline comprises of a sandy inter-tidal beach (Fraisthorpe Sands) backed by low-lying soft cliffs formed of a heterogeneous sediment mixture (from boulders to clay sized material) of Quaternary glacial till (Newsham, Balson, Tragheim, & Denniss, 2002). The toe of the cliff is close to the high water line making it susceptible to erosion during stormy periods with large waves. The greatest amount of cliff erosion would be expected when this process coincides with the high water period during positive storm surges. The cliffs are one of the fastest eroding coastlines in Europe (Sistermans & Nieuwenhuis, 2003; JNCC, 2007; ICES, 2016).

3.2.3.2 East Riding of Yorkshire Council (ERYC) undertake routine monitoring of the Holderness Coast which includes beach profiles from the top of the sea cliffs to low water. These profiles are surveyed in spring and autumn each year, notionally at 500 m spacing (shown on **Figure 4**). This survey record extends from 2003 to present and provides the basis of determining rates of cliff recession. **Table 4** provides a summary of cliff retreat rates for the beach profiles coincident with the immediate landfall area. Cliff recession rates vary along the entire coast, as well as year-to-year, but with a general increased rate towards the more southerly section of the coast, in line with increased exposure to northerly waves (i.e. less sheltering effects from Flamborough Head and Smithic Sands).

Table 4: Cliff recession rates at Profiles 14, 15 and 16.

Profile	Location	Height of cliff (m AODN)	Average cliff recession (m/year)	Maximum annual recession (m)	Year of maximum
14	North of Earls Dyke – Barmston	6.7	1.14	3.53	2017
15	South of Earls Dyke – Barmston	7.2	1.22	5.00	2005
16	Watermill Grounds – north of Barmston	8.3	1.57	6.54	2007

3.2.3.3 **Figure 5** provides an example of the survey record for Profile 15 which is mid-section of the landfall area (see **Figure 4** for location of profiles). Over the period June 2003 to May 2018, beach levels have varied but with a general reduction in vertical level of around 1 m over this 16-year period. The position of the base of the cliff has moved landward by around 20 m in this period. The apparent anomaly at chainage 370.00 m is a line of intact World War II tank traps. The generally, stable beach profile is likely to be indicative of a thin layer of mobile sands over a more resistant underlying glacial till (stiff glacial till of Bolders Bank Formation).

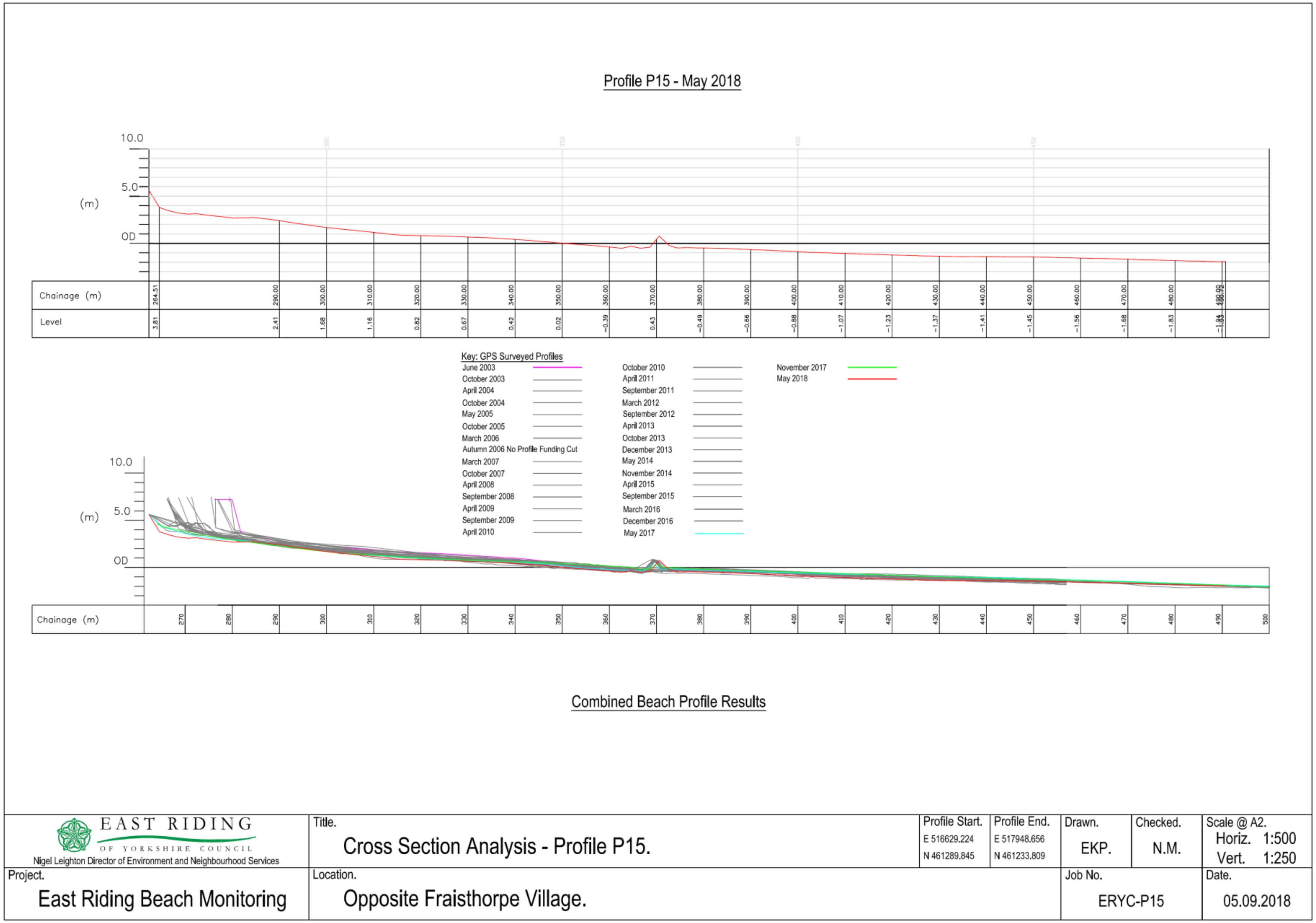

Figure 5: Example of beach profile monitoring record within landfall area (not to scale), from ERYC.

3.2.3.4 The regular tidal inundation of the beach between high and low water sweeps the finer material released from cliff erosion into the sea creating a visible nearshore plume. The coarser material produced by cliff erosion (sands and gravels) provide a primary source of beach material (Newsham, Balson, Tragheim, & Denniss, 2002). The sandy beach material is susceptible to longshore drift by wave-driven currents with the direction of drift determined by the oblique angle of approaching waves.

3.2.3.5 The Shoreline Management Plan (SMP) policy for the stretch of coast (Policy Unit C: Wilsthorpe to Atwick) which covers the landfall area is given as; "No Active Intervention" for the Short Term (present day to 2025), Medium Term (2025 to 2055) and Long Term (2055 to 2105) (Scott Wilson, 2010).

3.2.3.6 The National Coastal Erosion Risk Mapping (NCERM) identifies this frontage of coast as natural defence and erodible. Assuming the SMP policy remains unchanged, the predicted retreat distance for the short term (0 to 20 years) and medium (20 to 50 years) are given as 33 and 82 m (for the 50^{th} percentile confidence limit, +/- 30% for 5^{th} and 95^{th} confidence limits).

3.2.3.7 Sea level rise in this period would also expect to increase the rate of erosion[1] since the position of a higher mean sea level would translate landwards with a corresponding move of the high water line. Cliff erosion rates would also respond to any changes in the frequency and severity of storm surges.

Creyke Beck Landfall

3.2.3.8 The Creyke Beck landfall is around 1.5 km to the south of the Hornsea Four landfall. The anticipation is this installation is completed first and the Hornsea Four export cable will cross the Creyke Beck export cable east of Smithic Sands. Depending on the period between completion of the Creyke Beck landfall works and commencement of Hornsea Four landfall works there may be a potential for the beach to be in a state of partial recovery.

Earls Dyke

3.2.3.9 Earls Dyke (Earl's Dike) is a terrestrial feature located centrally in the landfall area. This is an artificial drainage channel serving a relatively small low-lying catchment (2,555 ha) south of Bridlington. The drain is not tidally locked which means that peak sea levels during surge tides can propagate inland and lead to periods of tidal flooding. The potential area of flooding is identified as part of the Environment Agency Flood Zone 3, derived on the basis of an event with a return period of 1 in 200 years.

Marine outfalls

3.2.3.10 Yorkshire Water operate two long sea outfalls approximately 3.5 km north of the landfall works. The 1.25 km long Bridlington Stormwater Outfall was installed in June 2014 and involved a 5 m deep open-cut trench inside a temporary cofferdam running 350 m down the beach at a location just to the south of Bridlington Harbour. The cofferdam was

[1] (https://www.eastriding.gov.uk/environment/sustainable-environment/looking-after-our-coastline/coastal-change-in-the-east-riding/)

subsequently removed with the trench backfilled (UK Water Projects, 2015). Presently, there is no visible evidence of beach or marine works at this location apart from the raised diffuser at the seaward end of the outfall.

Bridlington Harbour

3.2.3.11 Bridlington Harbour is around 4.5 km north of the landfall works and at the northern end of the landfall study area. The harbour is noted as being muddy (silts) and is considered as a sink for fine sediments. Estimates suggest that approximately 75% of the silts are from marine sources (principally the sediment plume created by cliff erosion) with the remaining 25% from terrestrial sources with material discharged into the back of the harbour from the Gypsey Race (HR Wallingford, 2005).

3.2.3.12 The estimated average build-up of silts in the harbour is nine inches (0.23 m) every year across the 10.5 acres (42,500 m2) area of the harbour bed. This creates a dredging requirement of between 12,000 to 14,000 tonnes per year (Maritime Journal, 2017). These sediments require dredging to maintain suitable depths with the spoil taken offshore for disposal at sea at disposal ground HU015, situated in the lee of Flamborough Head. The disposal ground is identified as a receptor within the offshore ECC study area.

3.2.4 Summary of marine physical environment receptors within the landfall study area

3.2.4.1 Table 5 summarises the receptors associated with the landfall study area. The potential sensitivity of each receptor is expressed prior to consideration of the scale of any impact related to the development.

Table 5: Marine physical environment receptors in the landfall study area.

Receptor	Potential sensitivity to marine processes
Holderness Cliffs	Changes in wave energy dissipation at toe of cliff that modify rates of cliff recession and supply of material to the beach.
Fraisthorpe Sands	Changes in sediment supply from cliff erosion. Changes in wave energy dissipation (wave height and direction) on the intertidal that alter the rate and direction of longshore drift.
Earls Dyke (terrestrial feature)	Long-term increases in sea level rise that increase severity and frequency of tidal flooding.
Creyke Beck Landfall	Beach lowering exposing export cables
Marine outfalls	High rates of deposition of coarse sediment onto diffusers which may block effective discharge of wastewater.
Bridlington Harbour	Increased suspended sediment concentrations in the nearshore leading to higher rates of harbour siltation from marine sources.

3.3 Offshore ECC study area

3.3.1 General description

3.3.1.1 The offshore ECC study area extends for around 107 km between the landfall and into the offshore array to provide a corridor for the six export cables. In addition, there is an option for a High Voltage Alternating Current (HVAC) booster area from around 34 to 42 km from the coast. The corridor is typically around 2.5 km wide, including the temporary works area, but widens in places to 4.8 km to accommodate the Dogger Bank- Creyke Beck cable crossing, to 3 km for the HVAC booster area and includes a widening fan onto the offshore array. With the inclusion of a 15 km buffer zone to conservatively accommodate the excursion of a mean spring tide, this develops an overall width of around 32 to 33 km. This buffer can be considered to represent an indicative zone of influence for the dispersion of fine sediments disturbed at the seabed (from foundation levelling, sandwave clearance or cable trenching) or disposed of as spoil at sites within the offshore ECC.

3.3.1.2 The HVAC booster area includes the option for three foundations to mount surface booster stations. The largest proposed foundation option here is a box-type gravity base with a dimension of 75 by 75 m. These structures have the potential to create blockage type effects on waves and currents that could extend into the wider study area.

3.3.1.3 The marine process environment varies from the shallow nearshore area in the lee of Flamborough Head to more exposed offshore conditions in deeper water towards the offshore array.

3.3.2 Process description

Seabed Profile

3.3.2.1 The offshore ECC commences from the seaward extent of the landfall area with depths approximately 7 m below CD onto a flattish seabed profile. This flat area is the seaward end of an ebb tidal channel that extends to Flamborough Head and defines the inshore flank of Smithic Sands. From this location, the offshore ECC gently shallows onto the southern part of Smithic Sands where depths reduce to around 5 m CD. Approximately 9 km from the coastline, the offshore ECC reaches the eastern edge of the bank, which also aligns with the seaward limit of Flamborough Head. Further to the east, the headland no longer provides direct sheltering from north and north-easterly waves, or strong tidal flows, and the seabed drops to around 20 m below CD. The profile of the seabed continues to deepen in an easterly direction and reaches around 50 m below CD at the HVAC booster area (approximately 34 km offshore), which is also the deepest section of the export cable route.

3.3.2.2 East from the HVAC booster area the offshore ECC passes just to the south of The Hills, a series of sinuous inter-related sandbank features with near symmetrical sandwaves. There are various undulations in depth along the route but also a generally shallowing profile to around 45 m below CD at the seaward end of the offshore ECC (**Figure 14**).

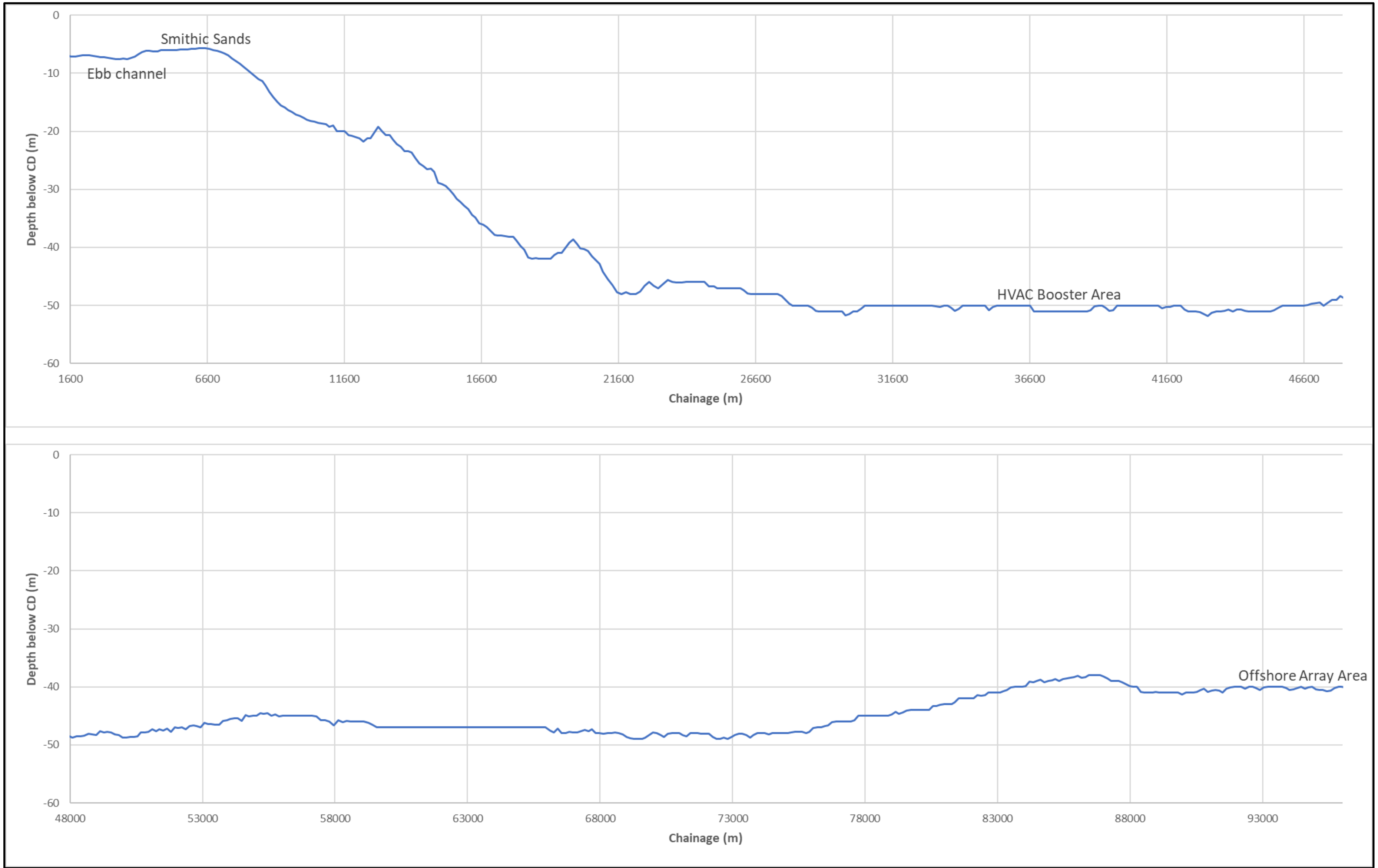

Figure 6: Seabed profile along offshore ECC, from landfall into the offshore array.

Sub-tidal sediments

3.3.2.3 Surficial sediment cover along the offshore ECC indicates an increasing sand content from inshore to offshore (**Figure 7**). From the landfall, the surficial sediments comprise of sands with patches of gravelly sand across Smithic Sands, then sandy gravels onto gravelly sands, slightly gravelly sands and finally sands across the majority of the offshore array.

3.3.2.4 Particle size information from available grab samples suggests the mud fraction is relatively low in surficial sediments and typically less than 1%. The highest content of muds is around 6% in a small area classed as muddy sandy gravel around 9 km to the west of the HVAC booster area.

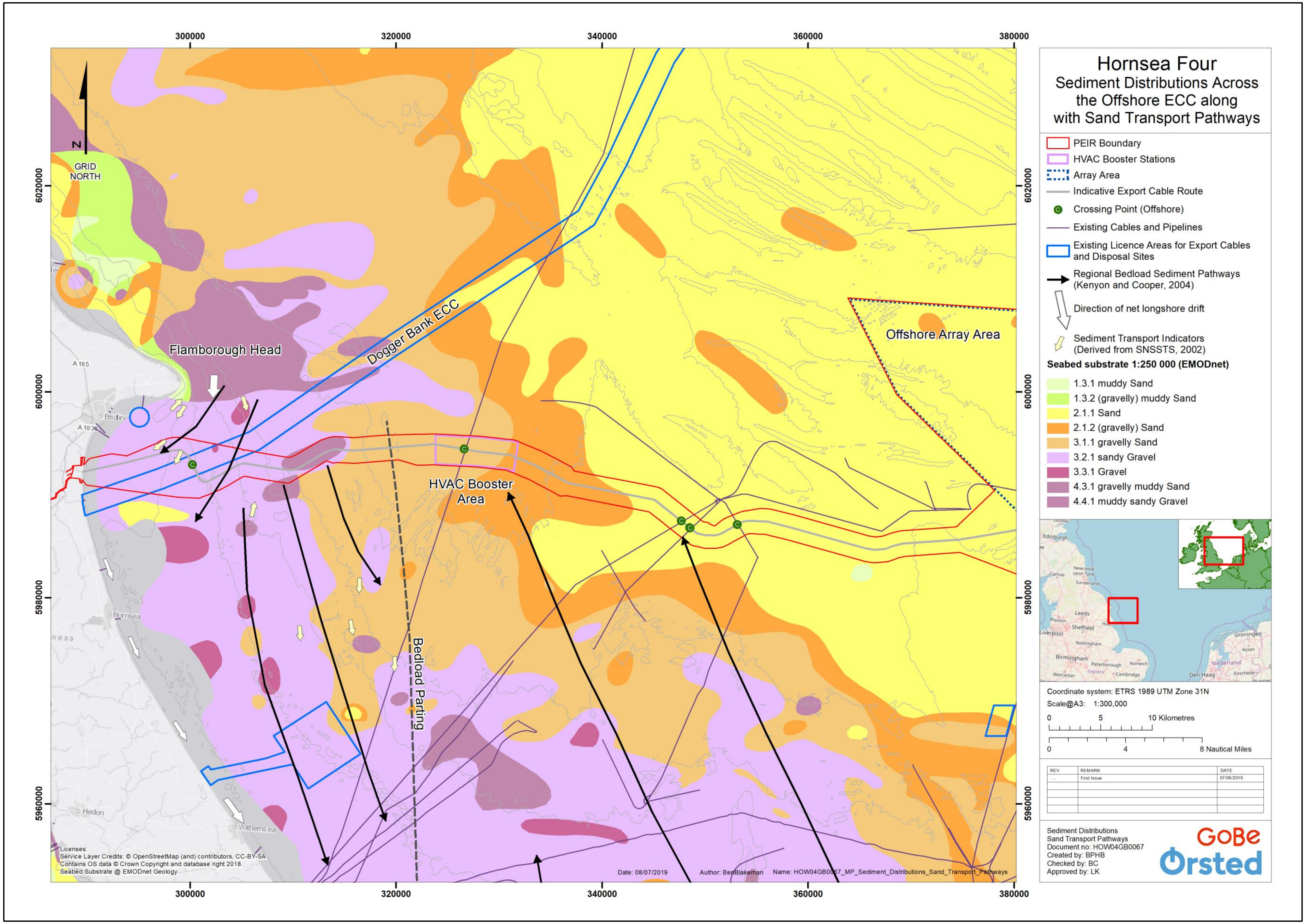

Figure 7: Sediment distributions across the offshore ECC based on descriptive classification by Folk (1954) (not to scale).

3.3.2.5 There is evidence of some bedforms (sandwaves) in existing bathymetric surveys (sourced from the United Kingdom Hydrographic Office (UKHO)) and most notably in the section of the offshore ECC from the HVAC booster area into the offshore array. The orientation of these features is flow transverse (90° to direction of flows).

Water levels

3.3.2.6 MSR varies from 5 m at the landfall area to around 3.3 m at the seaward limit of the offshore ECC within the offshore array (**Figure 8**). Equivalent MNR values are 2.4 and 1.6 m (DECC, 2008). The larger tidal range values at the western end of the offshore ECC are due to the greater distance from (two) tidal amphidromes in the Southern North Sea.

3.3.2.7 The combination of water depth plus tidal variation means that waves are unlikely to influence bedload transport, apart from the shallower inshore area approaching Smithic Sands and onto the shoreline (in the landfall area).

Tidal flows

3.3.2.8 In open water, tidal flows are generally to the south-east on the flood tide and north-west on the ebb. Closer inshore flows become more aligned with the orientation of the coastline, especially around Flamborough Head where flows are also strongest (peak of 1.2 m/s on mean spring tide). Regional mapping of tidal flows (DECC, 2008) shows flows tend to reduce from west to east along the offshore ECC, but the most sheltered conditions are in the lee of the headland (**Figure 9**). Peak flows on a mean spring tide for the HVAC booster area would be around 0.84 m/s.

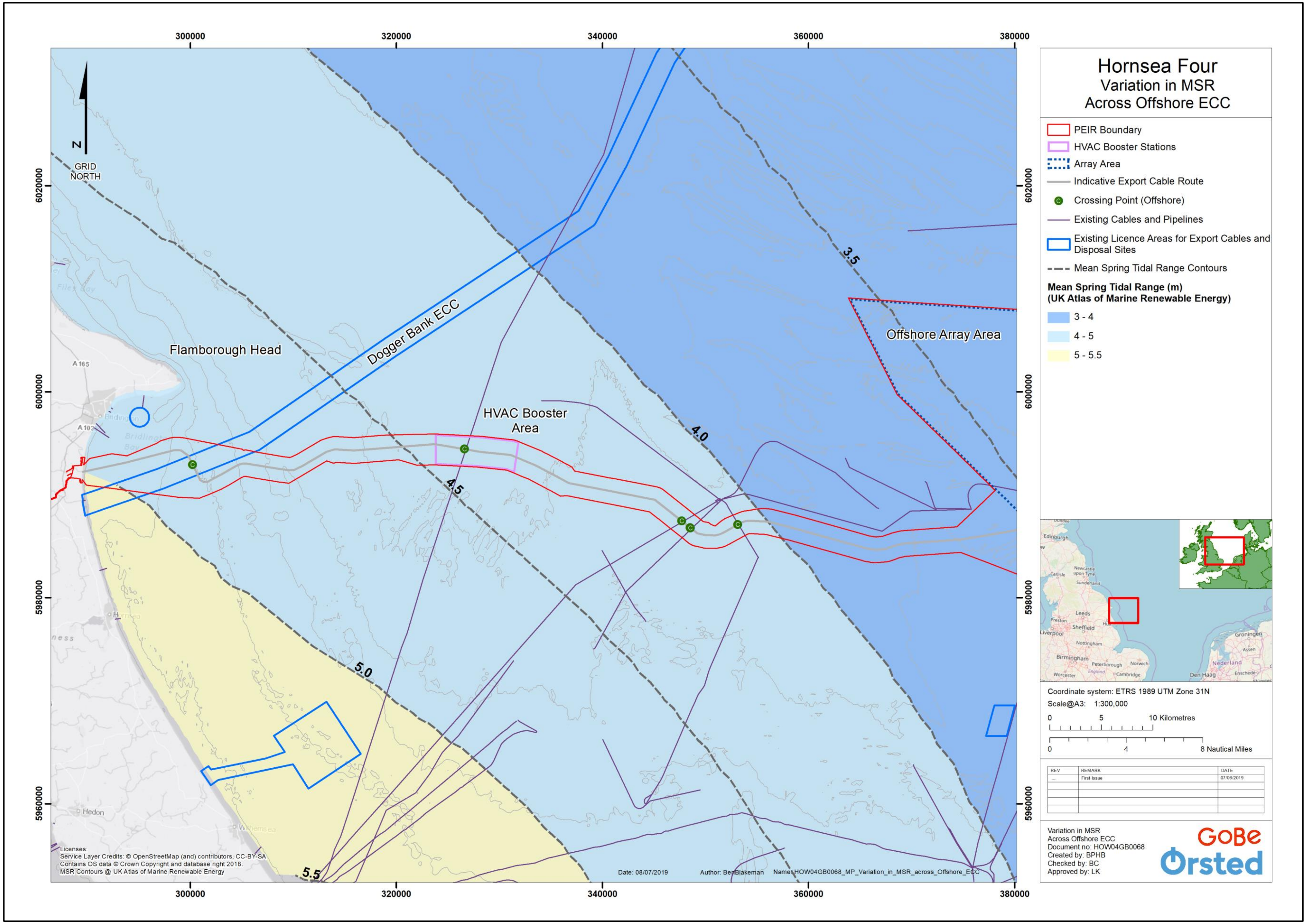

Figure 8: Variation in MSR across offshore ECC (not to scale).

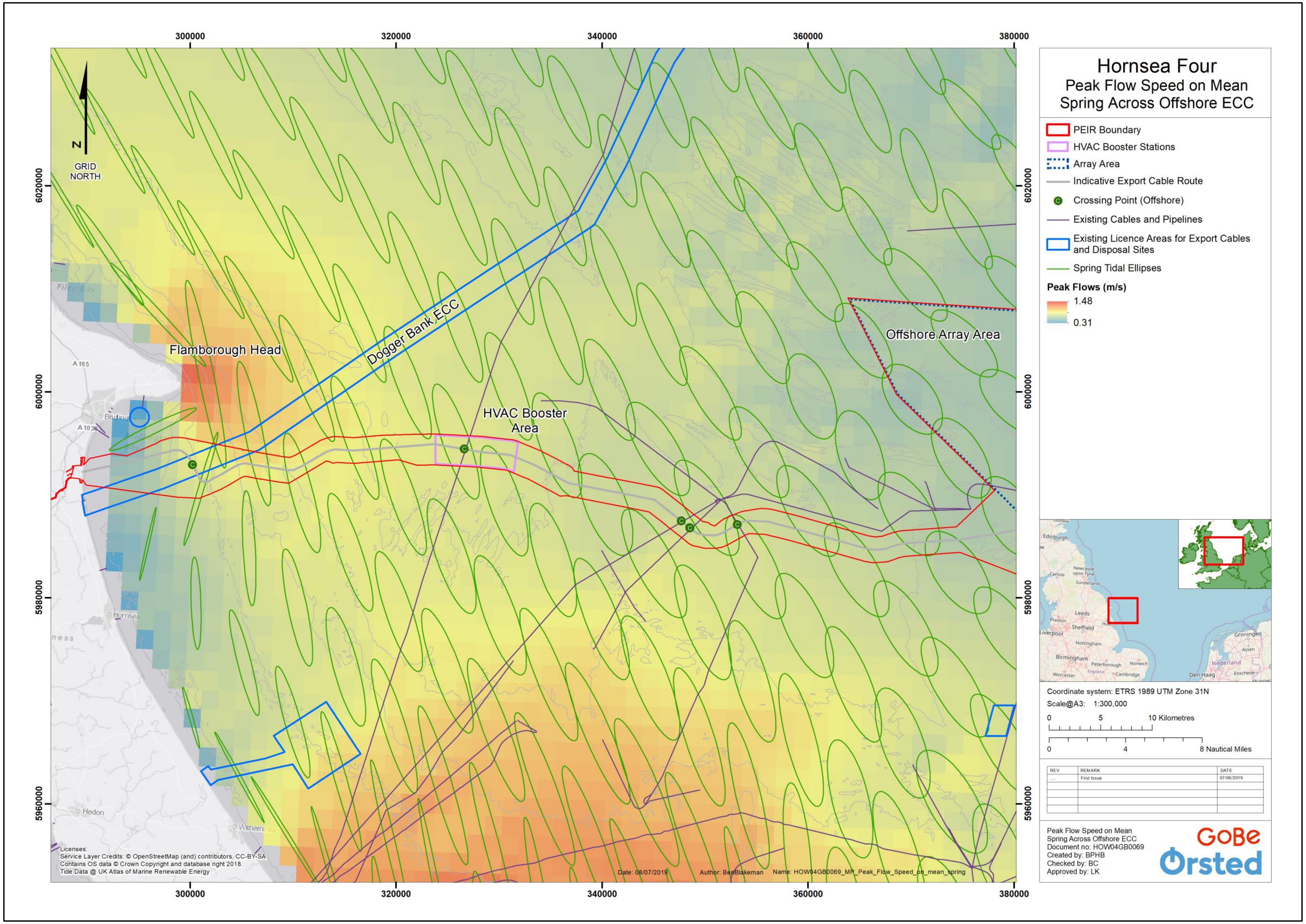

Figure 9: Mean spring tide, peak flow speed along with orientation of tidal ellipse scaled to represent the tidal excursion (not to scale).

Waves

3.3.2.9 **Figure 10** presents the spatial variation in winter wave heights across the offshore ECC area based on a regional wave model (DECC, 2008). The general pattern is for lower wave heights closer to shore, increasing in the offshore. Seasonal variation reduces wave heights during the summer period. **Table 6** provides summary wave height information for three locations along the offshore ECC from inshore to offshore.

Table 6: Summary wave height variability at sites along the offshore ECC study area.

Location	Winter Average wave height (m)	Summer Average wave height (m)
Inshore	1.20	0.79
HVAC booster area	1.84	1.06
Offshore	2.03	1.15

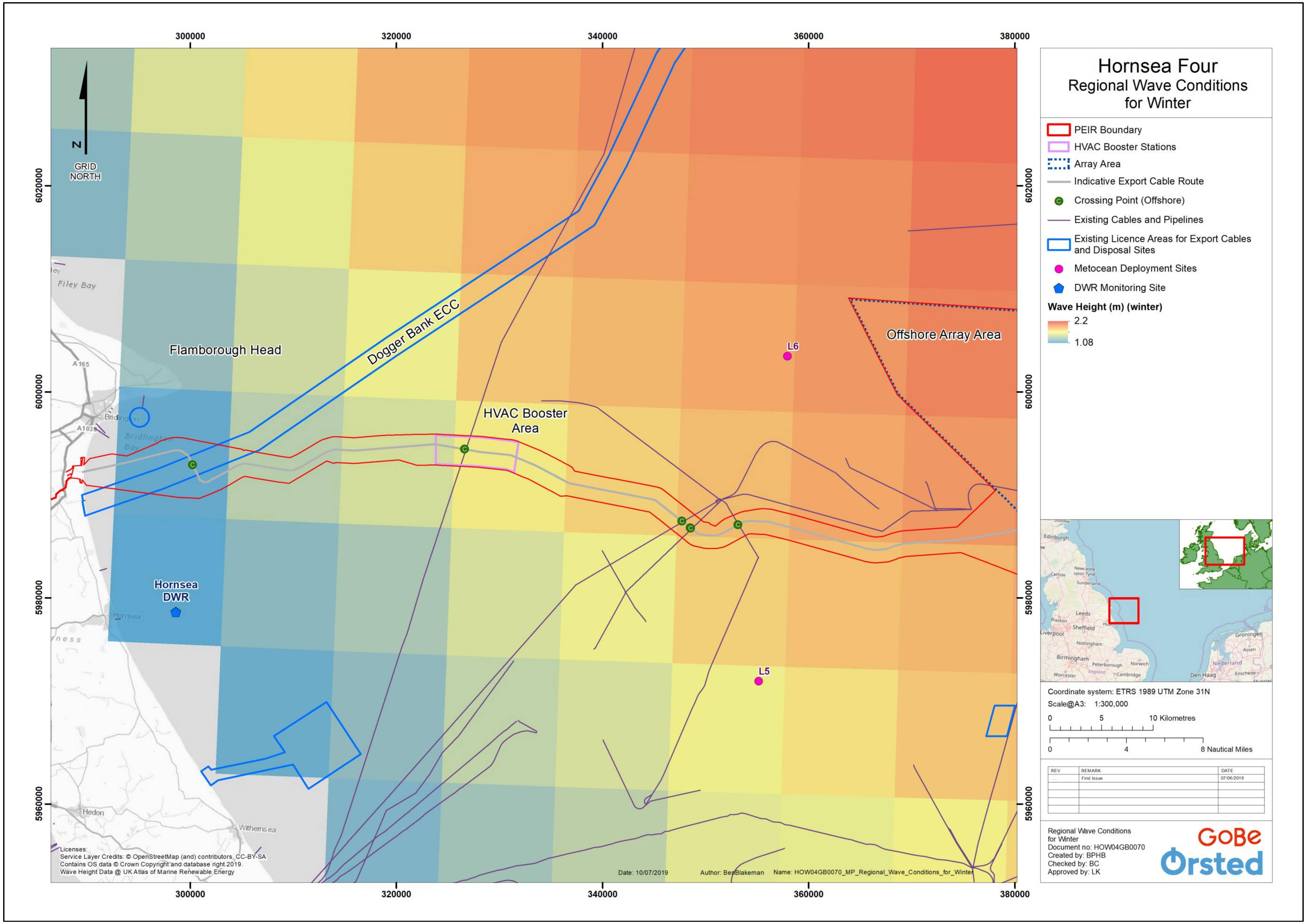

Figure 10: Regional wave conditions for winter (not to scale).

3.3.2.10 Wave observations are available at two locations slightly to the south of the offshore ECC; Hornsea DWR (local depths around 12 m below CD) and L5 - Off Grounds (local depths around 38.8 m below CD). Wave roses have been developed for the common period between these two locations; September 2010 to end of July 2011 (**Figure 11**). Site L6 also recorded waves but only for a 6-months period (from end of January 2011 to July 2011).

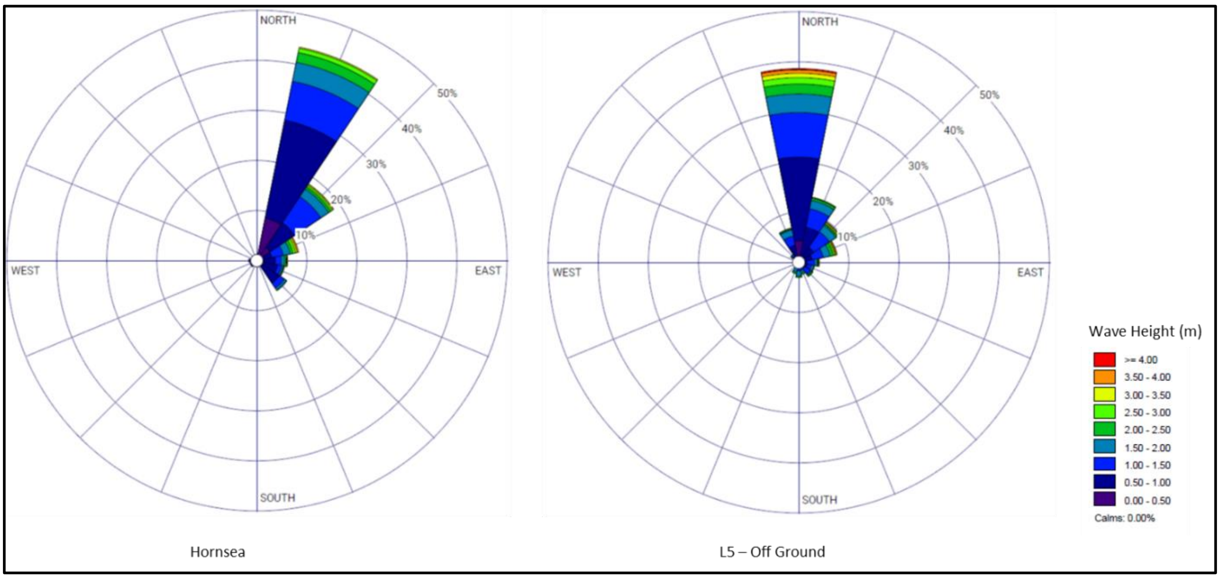

Figure 11: Wave roses for Hornsea DWR and L5.

3.3.2.11 Hornsea DWR (**Figure 10**) is slightly more exposed than waves conditions at the inshore end of the offshore ECC, but there is still evidence of local wave sheltering of northerly waves from Flamborough Head and some wave height reduction due to commencement of shoaling and refraction into shallow depths. Northerly waves approaching Site L5 are not influenced by such effects and are representative of offshore conditions.

3.3.2.12 Wave periods for both locations are typically in the region 3 to 6 seconds, and on a few occasions reached 7 to 8 seconds.

Bedload sediment transport pathways

3.3.2.13 Interpretations of regional sand transport pathways (Kenyon & Cooper, 2005) suggests that there is a net southerly transport for the area between the coast (from Flamborough Head) and the HVAC booster area and net northerly transport from the HVAC booster area onto the offshore array. A bedload parting zone separates these two areas (**Figure 7**).

3.3.2.14 Waves in deeper water (e.g. L5) have too short a wave period to exert any influence on the seabed, so these pathways are driven mainly by tides and tidal surge currents. In shallower water (e.g. Hornsea), waves begin to exert a stirring effect onto the seabed which can increase sediment mobility and rates of sediment transport. As an illustration, the largest measured wave for Hornsea considered in the observations was a significant wave height, H_s of 3.79 m and a mean wave period T_z of 6.7 s at 15:00 on 1 December 2010. At this time, the water depth was +4.1 m above CD. Based on shallow water wave theory, the maximum wave induced orbital velocity on the seabed would be 0.77 m/s, a magnitude similar to the peak flow on a mean spring tide.

Suspended particulate matter

3.3.2.15 Suspended particulate matter (SPM) comprises of suspended sediments and any other organic material held in suspension. The product of any material held in suspension is to reduce light penetration through the water column and lead to higher levels of turbidity. High turbidity levels can be detected by in situ sensors (such as OBS and Acoustic Back-Scatter (ABS) devices) as well as airborne sensors and satellites. The baseline description of SPM is applicable to assessing relative increases during sediment disturbance activities due to project development.

3.3.2.16 Spatial mapping of monthly mean non-algal SPM concentrations has been derived from satellite observations based on 18-years of data from 1998 to 2015 (Cefas, 2016). **Figure 12** presents SPM variations across the offshore ECC for the month of February which generally represents the maximum concentrations during the year. Surface concentrations are highest for around the first 10 km from the coastline and around Flamborough Head. July is typically the month with the lowest concentrations.

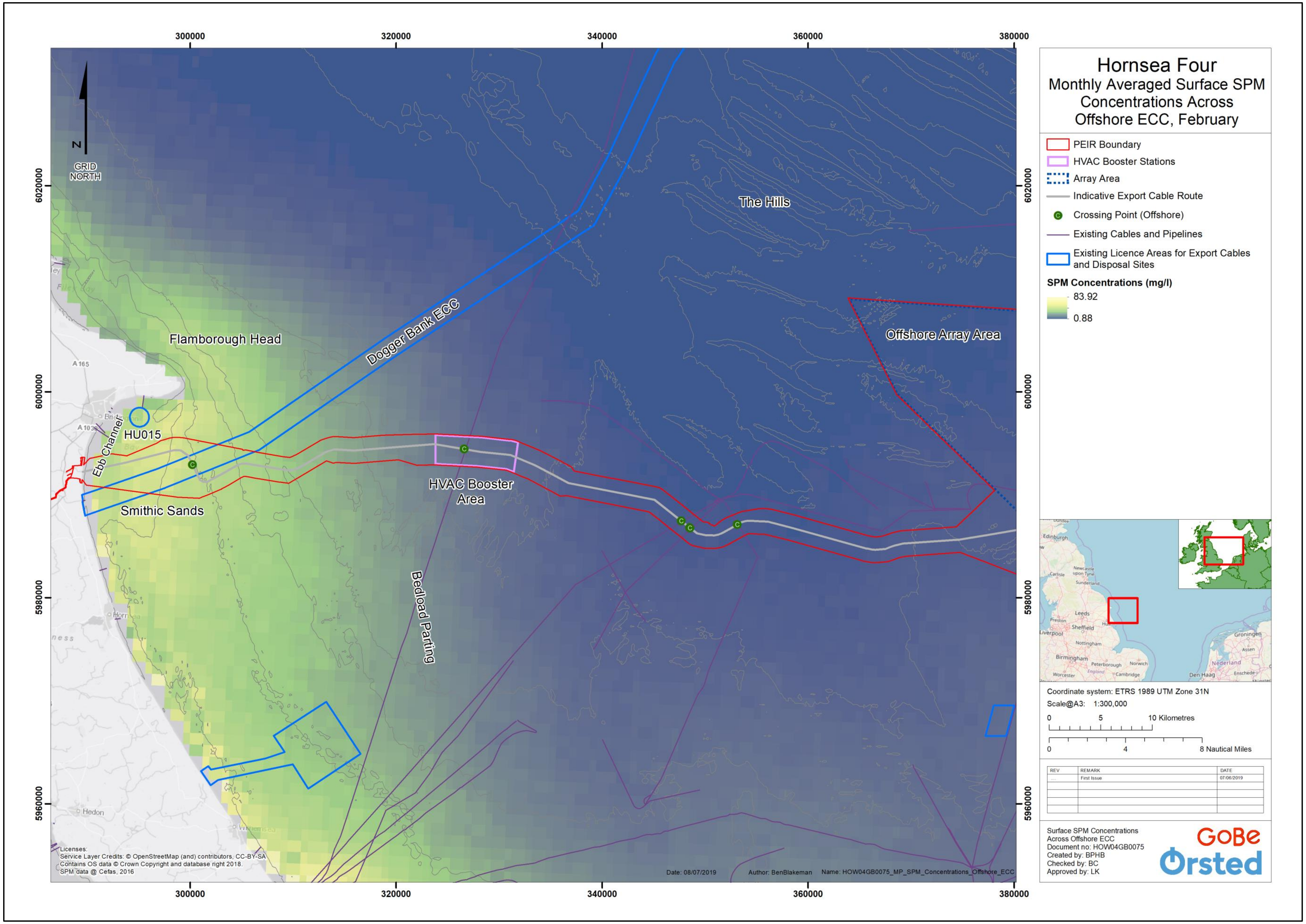

Figure 12: Monthly averaged surface SPM concentrations, February (not to scale).

3.3.2.17 Concentrations vary seasonally and are generally in the range 2 to 14 mg/l closer inshore (**Figure 13**). Concentrations reduce further offshore to levels around 2 to 3 mg/l. The larger variations and higher concentrations in the inshore region are mainly due to fine sediments eroded from the cliffs during winter periods, shallower water and locally stronger flows maintaining the material in suspension, preventing local deposition.

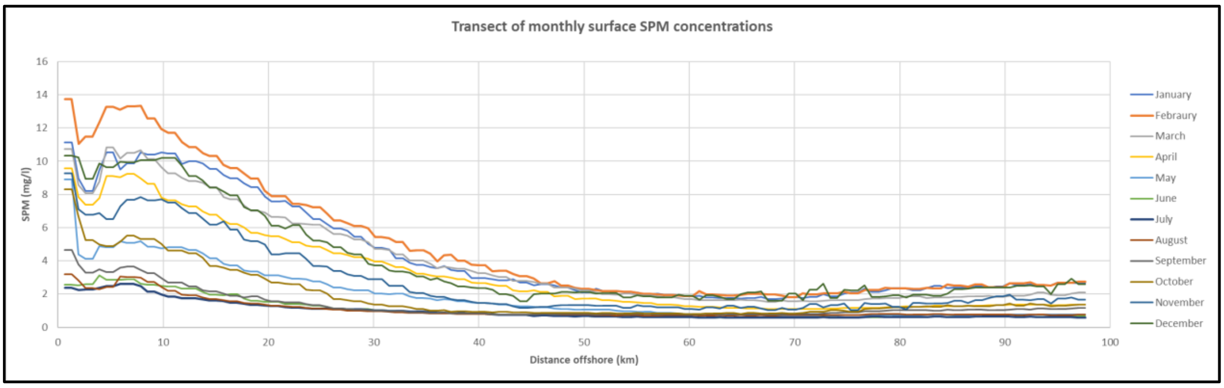

Figure 13: Transect along offshore ECC of monthly average surface SPM concentrations.

3.3.3 Marine physical environment receptors – offshore ECC study area

3.3.3.1 **Figure 14** shows the location of key features in the marine physical environment related to the offshore ECC study area.

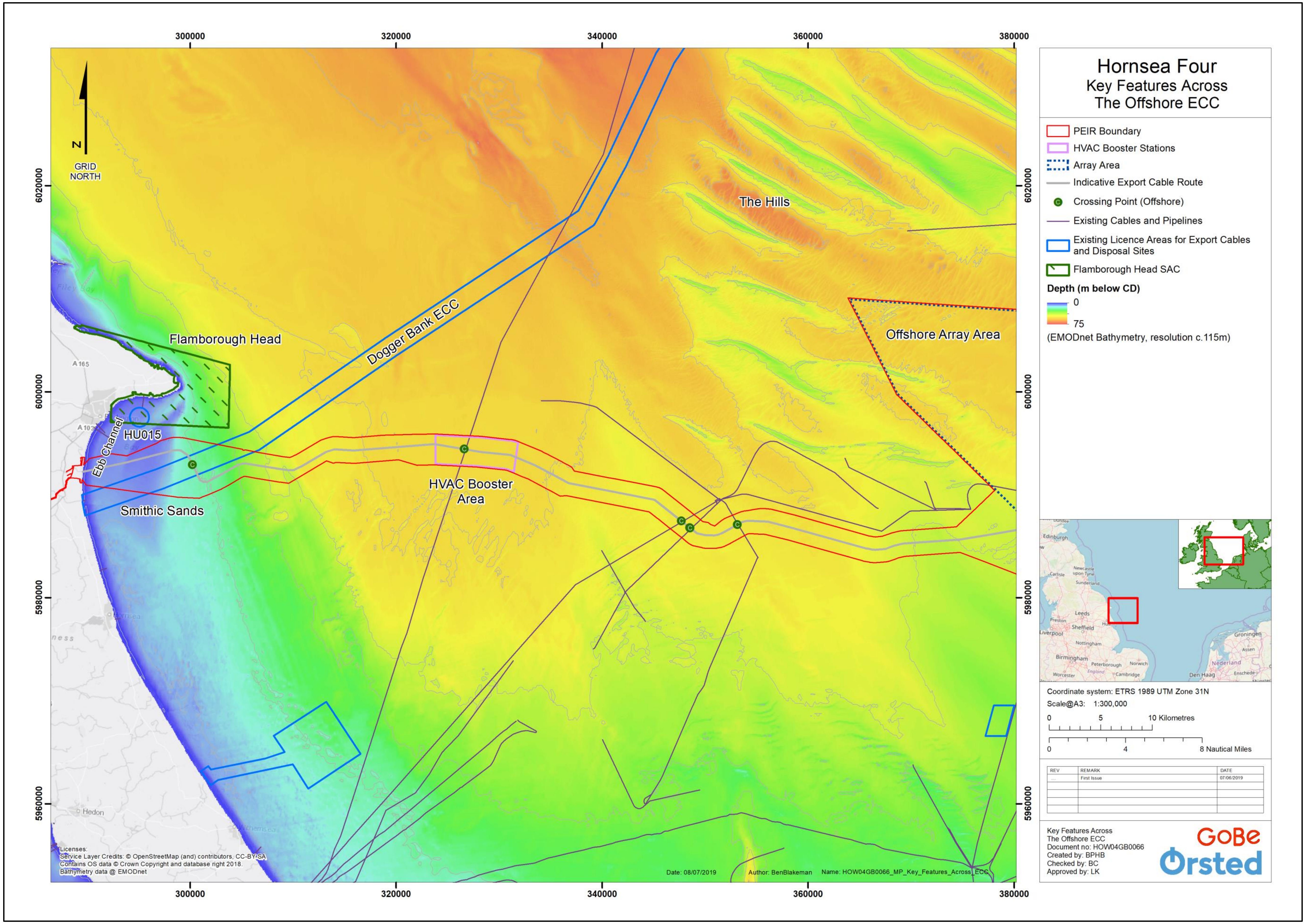

Figure 14: Key features across the offshore ECC (not to scale).

Spoil Ground HU015

3.3.3.2 The maintenance dredgings from Bridlington Harbour are disposed of at spoil site HU015 which is located approximately 2.3 km to the north of the offshore ECC and within the ebb tidal channel defining the western flank of Smithic Sands. The circular spoil site is 1.85 km wide with charted depths between 4.5 to 8.5 m below CD. Prior to 1985, a site 3.5 km to the south-west (HI010) was used for dredging disposals however, activity was shifted to the new site HU015 to accommodate concerns of the fishing industry that HU010 was accumulating silty material (Cefas, 2010).

3.3.3.3 The yearly maximum permitted disposal at HU015 is 30,000 tonnes of maintenance dredged material. The actual amount disposed of each year is far less, with dredging returns in the period 1999 to 2009 varying between 2,550 to 21,380 tonnes (Cefas, 2010), and averaging at 9,748 tonnes. These records show spoil disposal may occur at any month of the year, but not necessarily every month.

3.3.3.4 In 2017, Bridlington Harbour took possession of a new dredger, *Gypsey Race*, who has a capacity of up to 100 tonnes (Martime Journal, 2017). Based on past dredging records would suggest this capacity dredger would be used on average 97 times a year, with January typically being the month with most disposals.

3.3.3.5 HU015 mostly falls within the boundary of Flamborough Head SAC. Field investigations were undertaken in 2009 to address a concern by English Nature (now Natural England) that that spoil disposal activity could have the potential to impact the SAC. The field data indicated that there was currently little evidence that the disposal operation was affecting the integrity of the ecological features of the Flamborough Head SAC and no further monitoring was warranted in the immediate future unless significant changes to the disposal activities were anticipated. A repeat of the monitoring was advocated in five years' time (Cefas, 2010) but this recommendation does not appear to have advanced.

Flamborough Head SAC

3.3.3.6 Flamborough Head SAC encompasses the entire headland, and surrounding waters, and is around 1.6 km to the north of the offshore ECC at the closest point. The SAC is designated for various Annex I habitats, including reefs. This habitat may be susceptible to changes in suspended sediment concentration and high rates of sediment deposition, noting there is no evidence that maintenance dredgings (presumably fine sediments) disposed of at HU015 within the SAC has led to any significant impact at this time.

3.3.3.7 The seabed substrate around the headland is mainly rock (CCO, 2014), indicating an area scoured of mobile sediments by the locally faster flows.

Smithic Sands

3.3.3.8 JNCC identify Smithic Sands as a potential Annex I feature (subtidal sandbank) (JNCC, 2017). This feature extends south from the Flamborough Head SAC by over 12 km, with the southern part of the bank crossed by the offshore ECC.

3.3.3.9 The typology for Smithic Sands is a headland-associated banner type bank (HR Wallingford, CEFAS/UEA, Posford Haskoning, and Brian D'Olier, 2002), formed in the lee of Flamborough Head by clockwise tidal recirculation.

3.3.3.10 The sandbank is maintained by local sediment supply, with cliff erosion from the south likely to be a primary source of sandy material. This supply is initially transported by northerly longshore drift (for beach areas north of the drift divide at Barmston). The pathway is then deflected eastwards by the South Pier of Bridlington Harbour into the ebb channel running between the bank and Flamborough Head. The headland is regarded as divide between sediment cells (Cell 1 and 2) (HR Wallingford, 1993) and is classed as a one-way drift to the south, dominated by tidal flows (flood tide). Sands that may initially be transported on the ebb past Flamborough Head are returned on the flood tide, along with any additional material derived from sources north of the headland.

3.3.3.11 Evidence of active bedload transport is most prominent at the northern end of the bank (North Smithic) where large sandwaves are observed (CCO, 2014). This area is also associated with strongest tidal flows as water is forced past the headland. The asymmetric cross-sectional profile of these sandwaves offers supporting evidence for net clockwise directions of bedload transport around the bank. On the eastern outer flank, the sandwave asymmetry is with the flood tide, moving sands to the southwest and onto the bank, whereas for the western inner flank the ebb tide dominates through the distinct ebb channel between the bank and the headland to develop a net sediment pathway to the northeast (**Figure 15**).

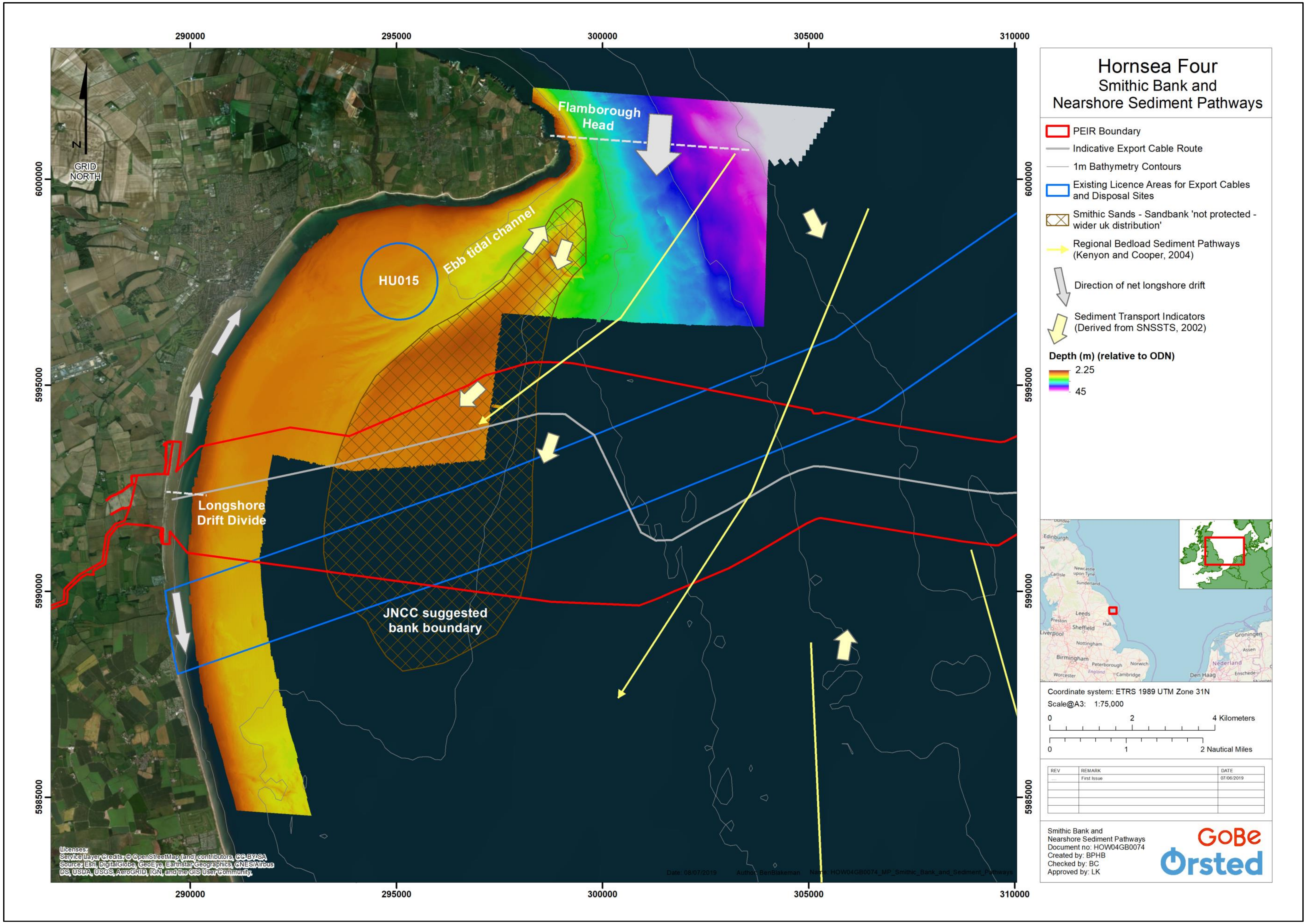

Figure 15: Smithic Sands and nearshore sediment pathways (not to scale).

3.3.3.12 The sediment cover through the ebb channel is described as a thin veneer over rock (CCO, 2014), scouring down to rock at the narrowest point at the northern end of the bank where the channel is around 1.3 km from the tip of the headland. Spoil site HU015 is located within this channel, suggesting any fines dumped onto this site are likely to be rapidly dispersed without becoming deposited either in the channel or on the bank. The bank acts as a local sediment store for sands.

3.3.3.13 The bank is shallowest (depths less than 3 m below CD) towards the northerly inshore flank where a steep slope drops around 6 m into the ebb tidal channel. The bank morphology shows evidence of responding to both waves and tides (CCO, 2014). Tidal flows are a key influence on driving sandwave migration whereas wave attenuation through refraction and shoaling are likely to be a main cause of smoothing and broadening the profile of the bank to the southern extents. The shallow profile of Smithic Sands provides some sheltering to the leeward coastline around Bridlington, especially during periods of stormy waves (Scott Wilson, 2010).

3.3.3.14 Over the longer-term any increase in mean sea level has the potential to place the profile of Smithic Sands lower in the tidal frame which would lead to a partial reduction in wave sheltering effects and potentially increased cliff erosion. If increased erosion lead to increased sediment supply to the bank, then the profile may be able to be maintained in a new dynamic equilibrium.

3.3.3.15 The offshore ECC crosses the southern part of Smithic Sands. At this location, the bank shoals on the seaward flank, from around 15 m below CD, to a relatively flat and wide surface with a shallow profile between 5 to 7 m below CD. The distance across Smithic Sands at this point is around 5 km. The geophysical survey conducted for Hornsea Four offers a seabed interpretation of sand with patches of gravelly sand across the southern part of Smithic Sands and reports depths of Holocene sediment cover of less than 6 m for this area (Bibby HydroMap, 2019).

3.3.3.16 The proposed Creyke Beck ECC also crosses Smithic Bank just to the south of the Offshore ECC. Geophysical surveys confirm sands and gravels across the bank and some areas with ripples and megaripples. Between the bank and the intertidal beach the surface layer of Holocene sand is recorded as < 1 m thick and in some places there is exposed glacial till. Over the southern end of Smithic Sands the depth of Holocene Sands extends to 6 m (ForeWind, 2013).

Cable and Pipeline Crossings

3.3.3.17 There are nine existing pipelines along the offshore ECC that require cable crossings. The Langeled gas pipeline crossing is within the HVAC booster area where depths are around 50 m below CD. This 44 inch diameter pipeline is discernible as a surface feature from the recent geophysical survey (Bibby HydroMap, 2019), noting that the pipeline appears to be on a gravelly seabed. The Cleeton to Dimilington pipeline crossing is east of the HVAC booster area in water depths of around 47 m below CD. A little further to the east, three crossings are required all within a distance of around 40 m. Neptune to Cleeton is the most easterly crossing in about 48 m depth below CD. All these further pipelines are not discernible in the geophysical survey as surface features. One explanation is that they have become buried within the sandy seabed where active bedforms (megaripples) are noted.

3.3.3.18 The offshore ECC also requires crossings with the proposed export cable from the Dogger Bank – Creyke Beck offshore wind farm. This crossing is planned at right-angles in an area seaward of Smithic Sands where local depths are around 20 m below CD. The orientation of the offshore ECC at this crossing would most likely be with the 20 m isobath meaning that any rock berms would also most likely be (near) parallel to local flows and where the seabed sediments are described as gravelly sand (ForeWind, 2013).

3.3.4 Summary of marine physical environment receptors within the offshore ECC study area

3.3.4.1 Table 7 summarises the receptors associated with the offshore ECC study area. The potential sensitivity of the receptor is expressed prior to consideration of the scale of any impact related to the development.

Table 7: Marine physical environment receptors in the offshore ECC study area.

Receptor	Potential sensitivity to marine processes
Spoil Ground HU015	Modification to local flows altering local dispersion characteristics, as a consequence of any large-scale changes in Smithic Sands morphology. The spoil site also has the potential to act cumulative during if disposal events of maintenance dredgings occurred in the same period as export cable laying activities in the nearshore region.
Smithic Sands	Insufficient sediment supply. Long-term increase in mean sea level (due to climate change) reducing sheltering effect to coastline if bank levels not sustained by sufficient sediment supply.
Flamborough Head SAC	Deposition of sediments onto Annex I reefs.
Pipeline and cable crossings	Local scouring around rock berms.

3.4 Offshore array study area

3.4.1 General description

3.4.1.1 The offshore array is located approximately 65 km seaward of Flamborough Head and covers an area of approximately 600 km^2 of seabed. Within this area there are provisions for up to 180 wind turbine foundations, nine offshore substation foundations and one offshore accommodation platform foundation. The general spacing of foundations for the indicative layout is around 1,100 m around the edge and greater for the majority of sites within the array at a typical value of 1,800 m, with a commitment for a minimum spacing of no less than 810 m between centres of each foundation location. Approximately 600 km of array cables will link up the wind turbines to the offshore substations. Due to two new pipelines expected to be installed through the array area, along with existing pipelines running through the site, there is also provision for up to 40 cable crossings.

3.4.1.2 The offshore array study area includes the offshore array and surrounding areas where any disturbed sediments from seabed levelling, sandwave clearance, array cable trenching or spoil disposal may reach due to plume dispersion of fine sediments. In addition, the offshore array study area also includes the areas where seabed structures may locally block passing waves, tidal flows or sediment pathways. Some of these effects have the potential of going beyond the offshore array study area.

3.4.1.3 Given the immediate proximity of Hornsea Project One and Hornsea Project Two, the offshore array study area also includes these adjacent wind farms and their associated foundation structures which might lead to a potentially larger cumulative blockage effect on waves, tides and sediment pathways.

3.4.2 Process description

Seabed profile

3.4.2.1 The general seabed profile across the array area shelves into deeper water in a northerly direction from around 40 m below CD towards the southern boundary to around 55 m below CD towards the northern boundary. Outer Silver Pit, a large geological "tunnel valley" depression, establishes the north-westerly / south-easterly alignment of the eastern boundary of the array area. There is a small area along the eastern boundary which coincides with part of Outer Silver Pit where depths reach around 60 m below CD (**Figure 16**).

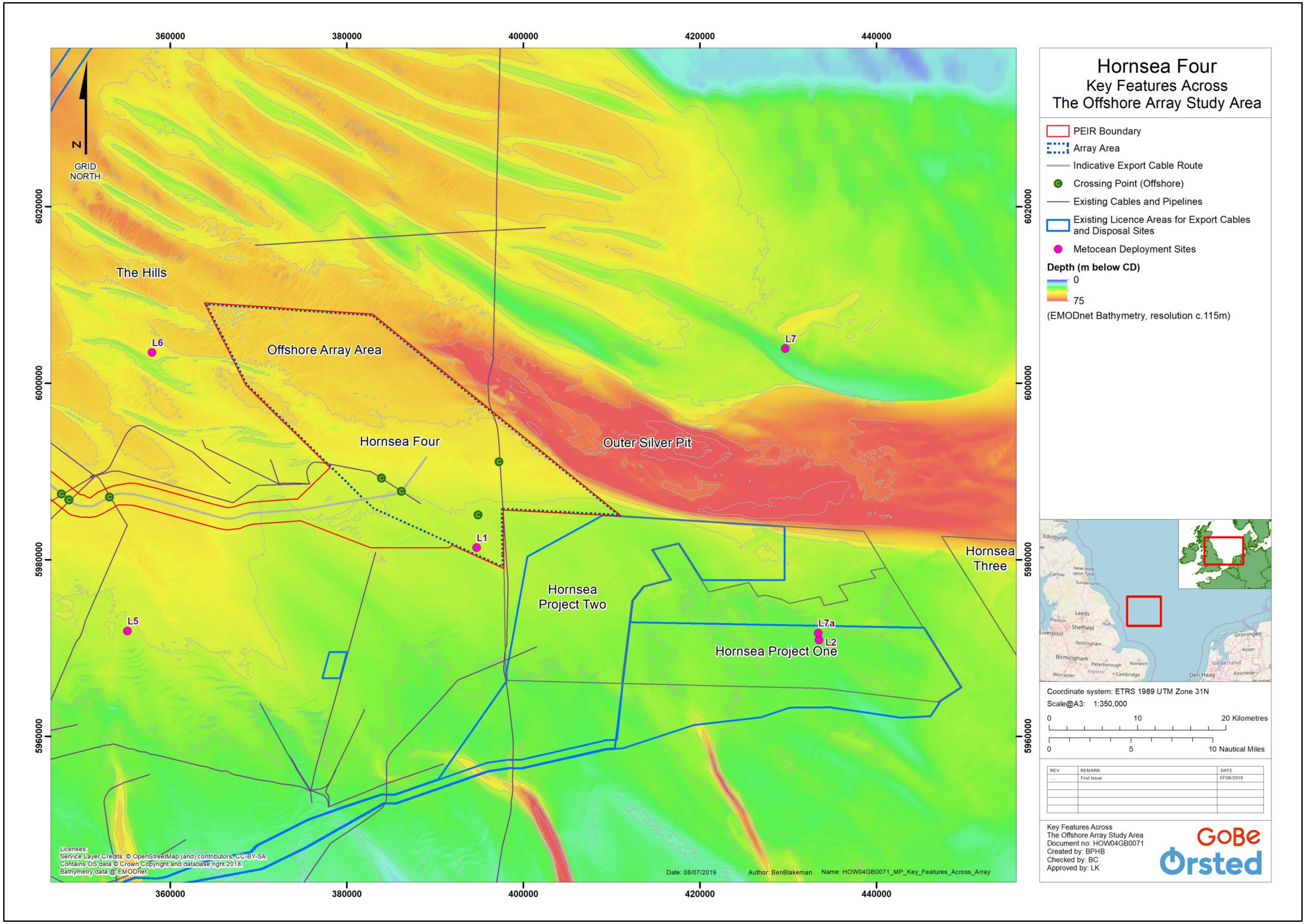

Figure 16: Key features across the offshore array study area (not to scale).

3.4.2.2 The shallowest parts of the offshore array area associated with large bedform structures, including sandwaves (wavelengths > 25 m), identified along the northern part of the western boundary. These bedform features are part of the larger series of sandwaves and open shelf sand ridges (sandbanks) that extend to the north and west of the array area and form an area known as The Hills (or Sand Hills). At this location, the shallowest depth is around 32 m below CD which is associated with the ridge of a sandbank feature (an area of convergence of sandwaves).

Sub-tidal sediments

3.4.2.3 Sands are the dominant surficial sediment type across the offshore array. Regional sediment mapping from EMODnet shows that there are also small patches of slightly gravelly sand and gravelly sand (**Figure 17**).

3.4.2.4 The compilation of grab sample evidence (including the 2018 geophysical survey data) supports the broader scale classification with a dominant sand content (medium sized), confirming the contribution of fine sediments (muds and silts) is generally low at < 5%, with a few exceptions, with a similar situation for the gravel content.

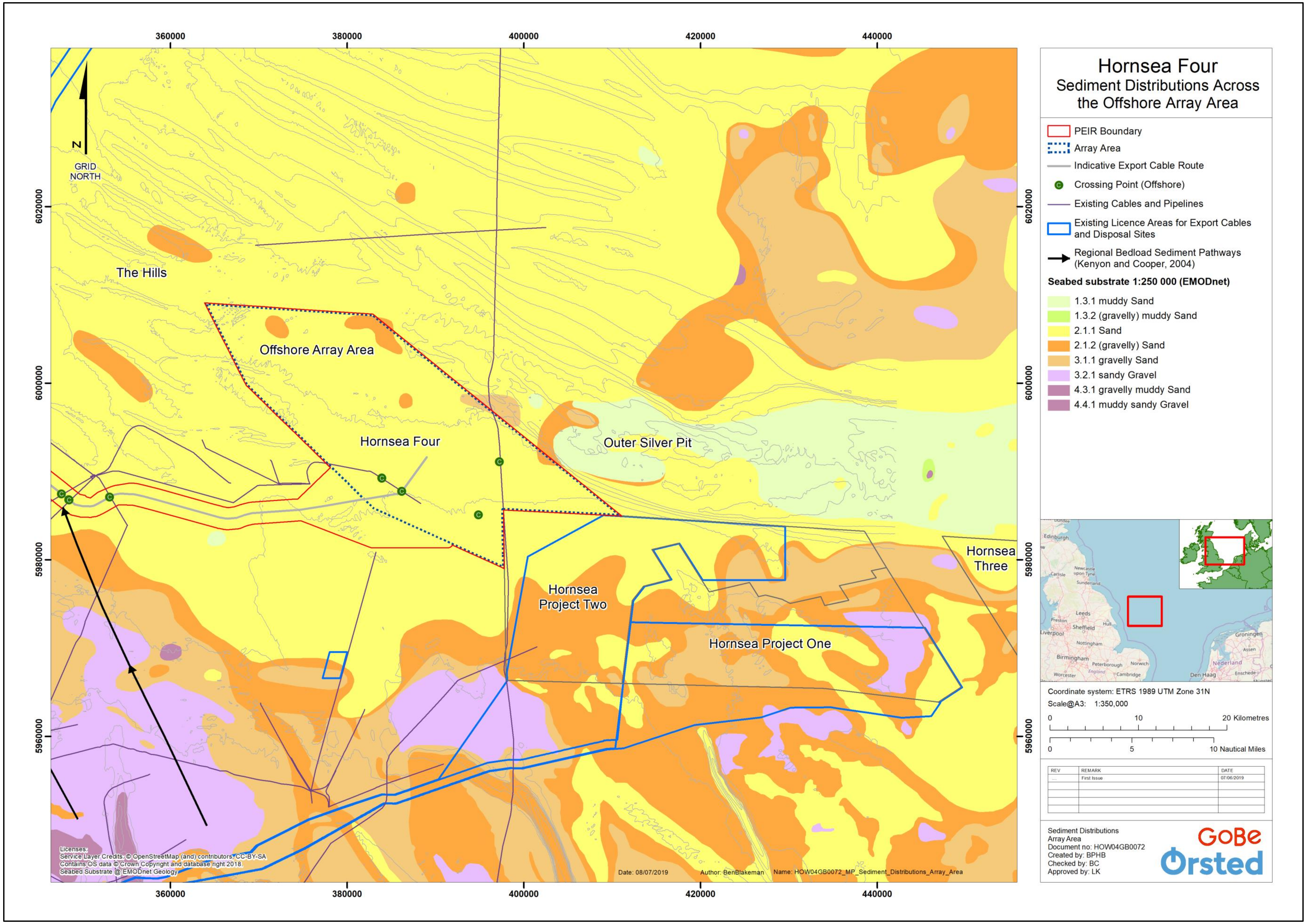

Figure 17: Sediment distributions across the offshore array study area (not to scale).

3.4.2.5 The base of the seabed (Holocene) sediments has been determined using sub-bottom profiling which suggests the majority of the area is less than 2 m thick. There are some local deviations with sediment thickness greater than 18 m over sandbank features towards the western boundary. Beneath the surface layer of Holocene sands is the firm to stiff clay till of the Bolders Bank Formation (Gardline, 2019). In the north-west corner of the offshore array there may be areas of exposed Cretaceous bedrock, or layers close to the surface, which are composed of chalk (SMart Wind, 2012). The presence of chalk has not yet been confirmed with recent geophysical surveys.

Water levels

3.4.2.6 The metocean survey for the former Hornsea zone included 12-months of observations from Site L1 – Well Bank Flat (29 June 2010 to 4 July 2011). This site is towards the southern part of the array area with a reported water depth of 37.5 m below CD (**Figure 18**). This dataset provides current velocity profiles, surface waves, water levels and near-bed backscatter (OBS and ABS) measurements. In addition, the spatial variance across the array is considered with reference to relevant details from the Atlas of UK Marine Renewable Energy, which are based on regional scale models.

3.4.2.7 Tidal range increase from east to west across the offshore array due to increasing distance from tidal amphidromes in the Southern North Sea. MSR is around 3.0 m at the easternmost extent increasing to around 3.5 m at the westernmost extent (DECC, 2008) (**Figure 18**). At Site L1 (around mid-way across the southern part of the array), MSR is assessed to be 3.28 m (EMU, 2013). Equivalent tidal range values for MNR are 1.50 to 1.77 m, and 1.61 m at Site L1.

Tidal flows

3.4.2.8 The most common sediment fraction present across the offshore array is medium sands (particle size in the range 0.25 to 0.50 mm) (Gardline, 2019). This sediment size requires flows in excess of 0.5 to 0.6 m/s to become mobilised, based on standard theoretical expressions (Soulsby, 1997). Tidal mapping from the Atlas of UK Marine Renewable Energy suggests this magnitude is generally limited to peak flows during spring tides (**Figure 19**) and is not attained during neap tides.

3.4.2.9 Tidal measurements show that times of peak flows occur at approximately high and low water.

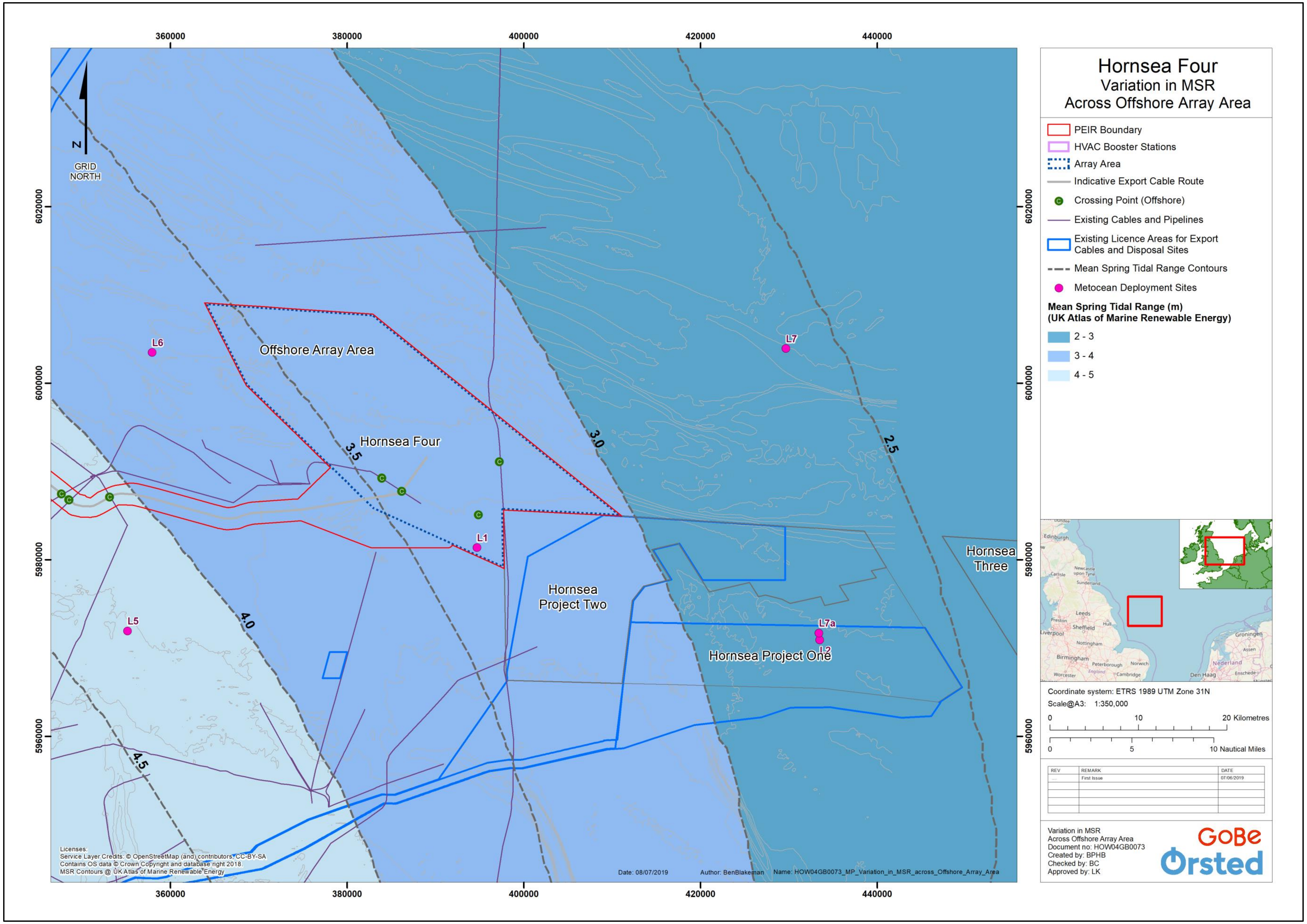

Figure 18: Variation in MSR across offshore array study area (not to scale).

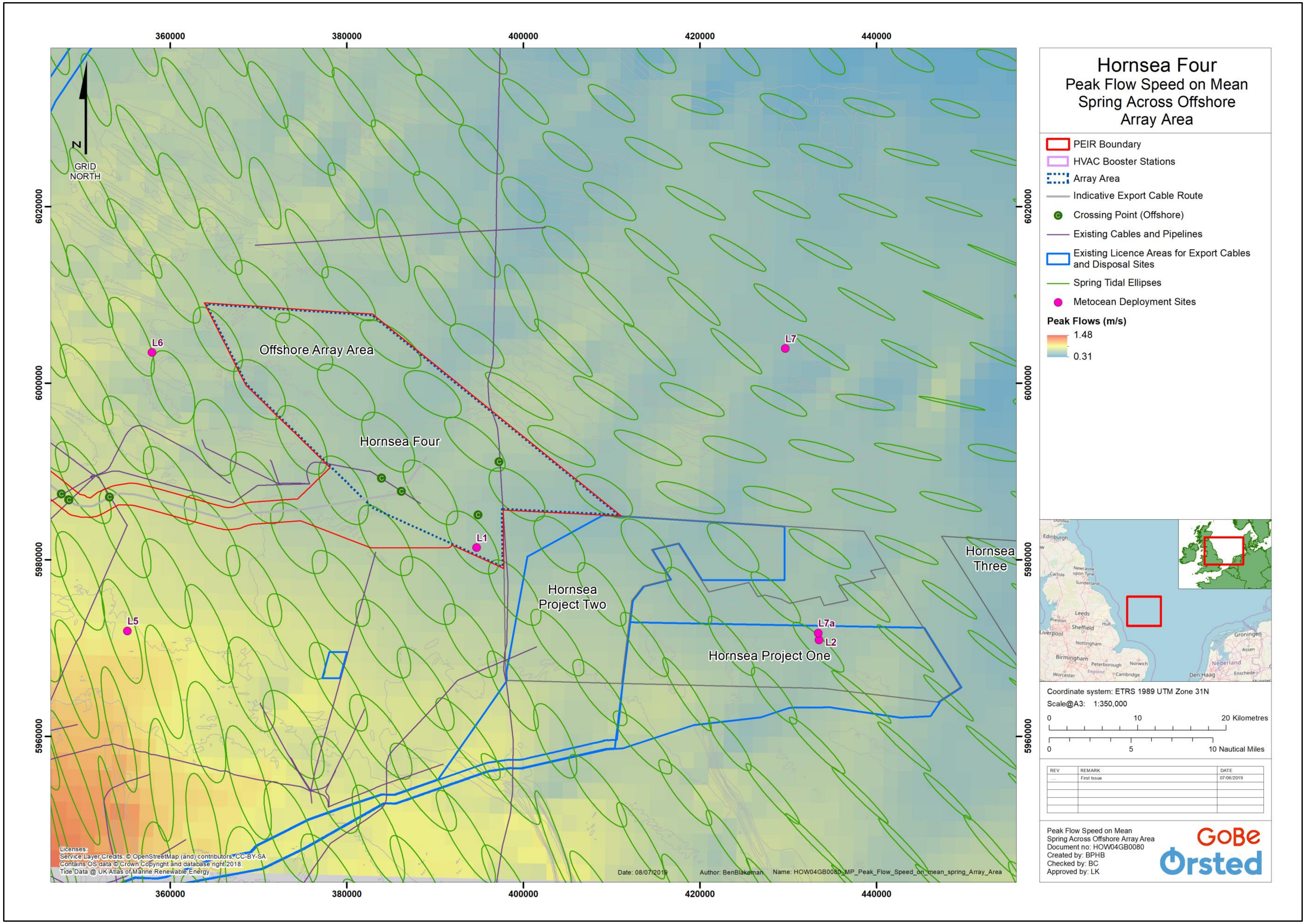

Figure 19: Peak flow speed on mean spring across offshore array study area (with orientation of tidal ellipse) (not to scale).

3.4.2.10 A current rose of depth-average flows at Site L1 (Figure 20) shows that the main tidal axis is south-east for the flood and north-west for the ebb. There is also a slight asymmetry between peak flood and ebb flows (i.e. peak flows > 0.6 m/s required for sediment transport) which supports a south-easterly (ebb) net transport direction for sands at this location. Site L6 – Ravenspurn Field is just westward of the array area, in a depth of 46.7 m below CD, but still offers a useful indication of likely flow conditions for the more northerly area. The asymmetry between peak flood and ebb flows at Site L6 indicates a slight north-westerly (flood) net transport direction, supportive of the sandwave asymmetry in this general area.

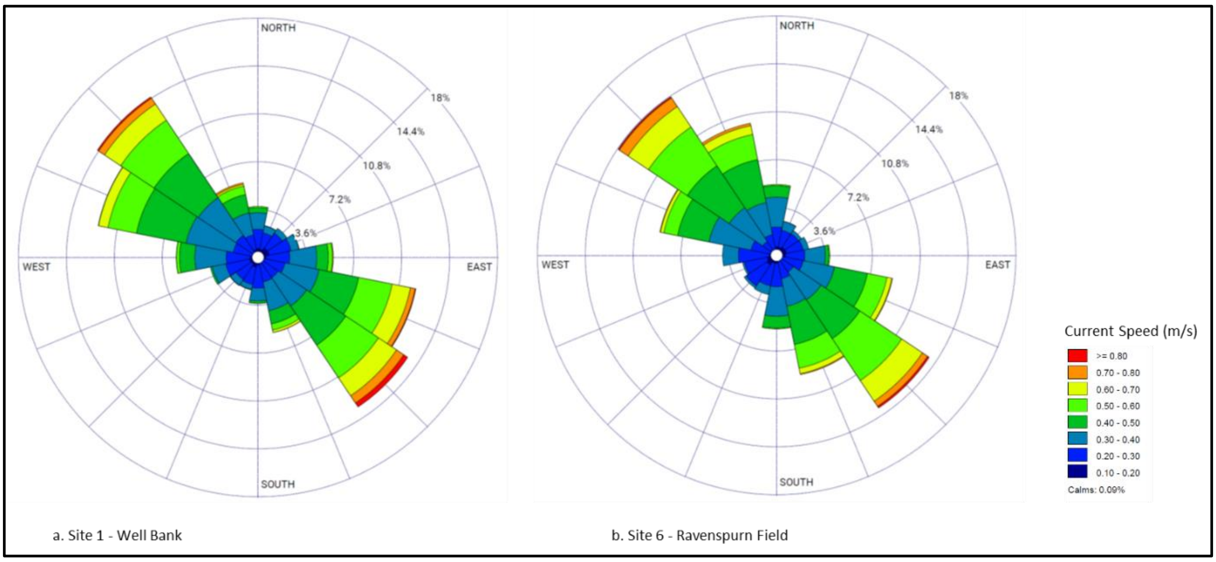

Figure 20: Current roses for sites L1 and L6.

Waves

3.4.2.11 Waves measured at Site L1 (southerly part of offshore array area) indicate wave periods (T_z, zero up-crossing period) in the range 3 to 6 s, and typically around 4 s. Wave heights (H_s, significant wave height) were typically less than 1.0 m but reached 4.5 m during a storm event in November 2011 (EMU, 2013). The wave period, T_z at this time was 6 s and from a south-westerly wave direction of 240°N. The equivalent maximum wave induced orbital seabed velocity would have been 0.07 m/s. If the same wave was in a deeper section of the array (around 55 m below CD) then the equivalent orbital velocity would be 0.01 m/s (these values ignore any additional water level contribution due to tidal influence). For the shallowest area at 32 m below CD, the equivalent wave orbital velocity would be 0.13 m/s. On this basis, even the largest measured wave event was incapable of stirring local sediments alone. This means peak tidal currents during spring tides are the main mechanism for developing sediment transport across the offshore array area.

3.4.2.12 The distribution of wave heights measured at L1 is presented in Figure 21 as a wave rose to demonstrate the prevalence of the north-north-westerly direction, which is also the direction which contains most of the largest wave events (43% of all waves > 3.5 m are from the north-north-westerly sector).

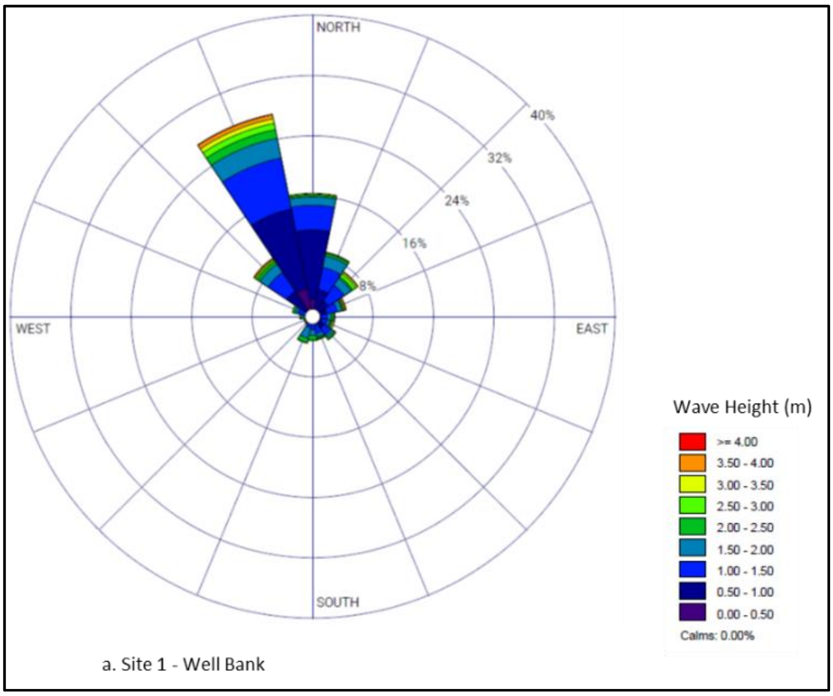

Figure 21: Wave rose for site L1.

Bedload sediment transport pathways

3.4.2.13 Sandwave crests are evident across much of the offshore array, apart from the southern extents. These crests are generally aligned perpendicular to the axis of tidal flows, a feature which is observed in both the zonal geophysical survey (SMart Wind, 2011) as well as the 2018 geophysical survey (Gardline, 2019).

3.4.2.14 Figure 22 provides an example of observed sandwaves along a transect line from the northern part of the offshore array. Zero chainage is at the north-west with increasing chainage to the south-east. Three large bedform features (sandwaves with heights of 3 to 4 m with megaripples on their stoss slope) are shown with steeper slopes to the north-west. Between these larger features are a series of smaller amplitude sandwaves with the same asymmetry. This asymmetry indicates a net direction of bedform migration towards The Hills.

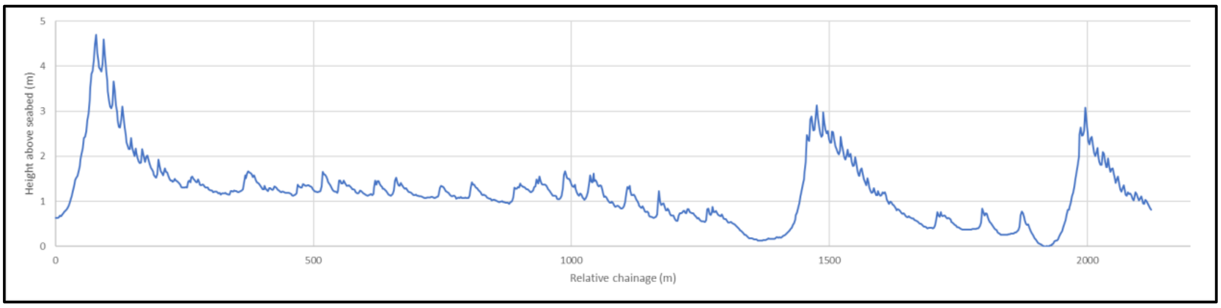

Figure 22: Example of sandwaves and megaripples within northern part of array area (based on data from Gardline, 2019).

3.4.2.15 Figure 23 provides an example of observed sandwaves along a transect line from the central part of the offshore array, at the southern extent of sandwaves. Zero chainage is at the north-west with increasing chainage to the south-east. In general, sandwaves appear to have lower amplitudes of 1 to 2 m with intervening megaripples. The sandwave asymmetry appears to be less distinctive (more symmetrical) than equivalent features to the north, although megaripples appear more prominent on the north-westerly facing slopes.

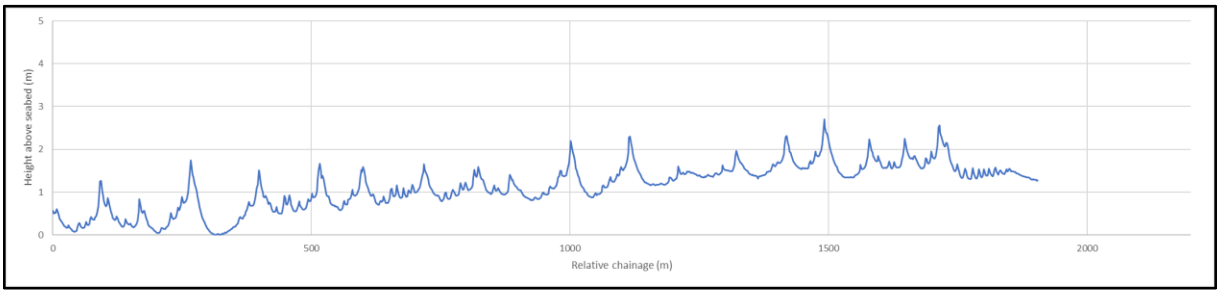

Figure 23: Example of sandwaves and megaripples within central part of array area (based on data from Gardline, 2019).

3.4.2.16 Previous investigators considered the area known as The Hills (Sand Hills) to be a convergence between ebb and flood transport pathways creating an area with near symmetrical sandwaves (Dingle, 1965).

Suspended particulate matter

3.4.2.17 The variation in derived suspended sediment concentrations for the near-bed ABS observations also correlates well with the observed peak flows, further demonstrating periods of sediment mobility (of finer material) being brought into suspension.

3.4.2.18 Surface turbidity (represented by SPM) is relatively low across the offshore array, with monthly averaged concentrations typically less than 5 mg/l across the whole year (Cefas, 2016), with minimal seasonal variation. The relatively low concentrations are due to both a low content of fine material in the seabed sediments and the area being distant from any terrestrial sources, such as the Humber Estuary and the Holderness Cliffs. Figure 24 provides a synoptic view of long-term SPM concentrations averaged for the month of February (generally the month with the higher SPM concentrations). The SPM values have been deduced from satellite data for the period 1998 to 2015.

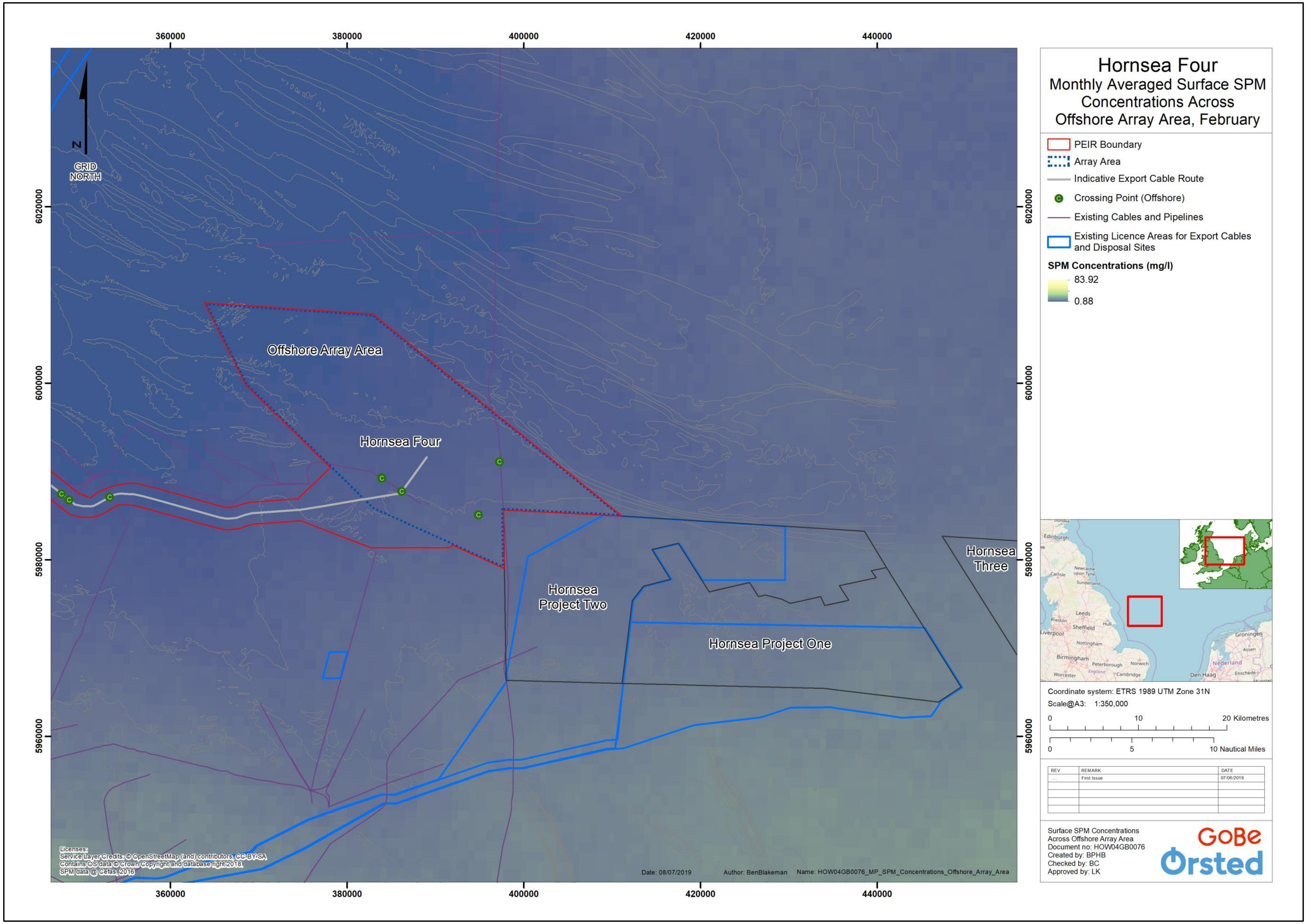

Figure 24: Monthly averaged surface SPM concentrations across offshore array study area, February (not to scale).

3.4.3 Marine physical environment receptors – offshore array study area

Pipelines

3.4.3.1 There are two existing pipelines which coincide with the offshore array; the Ravenspurn North Gas Field and the Shearwater – Bacton SEAL pipeline. In addition, a further three new pipelines are proposed which plan to pass through the offshore array. Assuming all new pipelines are in place at the time of construction this would require around 40 cable crossings for the array cables with an average of approximately 7,075 m^3 of rock being laid over an area of approximately 6,375 m^2 at each crossing.

3.4.3.2 The Shearwater – Bacton SEAL pipeline is detected in the geophysical data as a surface feature (Gardline, 2019).

Flamborough Front

3.4.3.3 The Southern North Sea is generally described as a well-mixed water body. These well-mixed conditions are mainly due to relatively shallow depths and the ability of winds and tides to continually stir water sufficiently to prevent the onset of any stratification (DECC, 2016). In contrast, the Northern North Sea is relatively deeper with slightly weaker currents, this helps temperature stratification develop from the spring into the summer months. During this period, a transition between these two water bodies develops from about 10 km offshore of Flamborough Head in the form of a temperature front. The deeper stratified water to the north tends to remain aligned with the 50 m isobath (Hill, et al., 1993). The surface waters of the front tend to move around this alignment with the scale of tidal advection. The front becomes nutrient rich and is considered to be ecologically important. During autumn / winter the front dissipates due to increased wind and wave related stirring effects which re-establish well-mixed conditions for this part of the northern part of the North Sea.

3.4.3.4 An assessment of the period of development and location of the Flamborough Front, relative to Hornsea Four, has been informed by the forecast model; NORTHWESTSHELF_ANALYSIS_FORECAST_PHY_004_013 (Tonani, et al., 2019). This model has a spatial resolution of 0.016° Latitude x 0.016° Longitude, equivalent to around 1.5 km north to south and 2.0 km east to west for the study area. The database of forecast outputs is held by the Copernicus Marine Environment Monitoring Service. The latest full year of daily mean values from 2018 has been assessed for variation in near-bed and near-surface temperature, as well as mixed layer depth (MLD).

Three locations in the North Sea have been examined for annual variation in water temperature (surface and near bed) and for development in MLD:

(a) A site 40 km to the north of the offshore array in a water depth of around 66 m below CD, representing the Northern North Sea water body (**Figure 25 a**). This area is expected to demonstrate temperature stratification over the spring and summer period.

(b) A site within the area to verify local conditions relative to the Flamborough Front

(**Figure 25 b**). Depths at this location are around 42 m below CD.

3.4.3.5 A site 40 km to the south of the offshore array in a water depth of around 37 m below CD, representing the Southern North Sea water body (**Figure 25 c**). This area is expected to demonstrate no temperature stratification and maintain well-mixed conditions over whole year.

Figure 25: Annual variation in water temperature and MLD at; (a) North Site, (b) Offshore array and (c) South Site.

3.4.3.6 **Figure 25** shows a clear representation of seasonal temperature rise and fall at all sites, with a peak summer temperature evident around the end of July. For the southern site, the near-surface and near-bed water temperatures appear equivalent throughout the year and the MLD is sustained over the full water column, demonstrating this location remains effectively well-mixed. In contrast, at the northern site there appears to be mainly warming of surface waters from around April which develops stratification in the water column, reducing the MLD. The largest difference between near-surface and near-bed temperatures is around the end of July with close to a 10 °C difference. From August to September the MLD is re-established due to reduced solar warming and increased wind and wave related stirring influences to establish well-mixed conditions with near-surface and near-bed water temperatures becoming equivalent again. Between these two locations there is an area where the Flamborough Front develops as a seasonal feature lasting approximately five months.

3.4.3.7 For the array area, a pattern similar to the northern site occurs, but with slightly less distinctive stratification between near-surface and near-bed water temperature. This is partly due to the array area being 24 m shallower than the northern site. This evidence locates the array area to the north of the Flamborough Front.

3.4.3.8 The location of the front can be defined as the spatial transition between mixed and thermally stratified water. **Figure 26** shows the MLD for the period of maximum stratification identified towards the end of July. Where the MLD is minimal the water is stratified, for well-mixed areas the MLD tends to represent the total water depth. On this date, the front is around 10 km offshore of Flamborough Head then appears to closely follow the 40 m isobath where an east-west alignment develops which continues to run south of the array area by approximately 3.5 km.

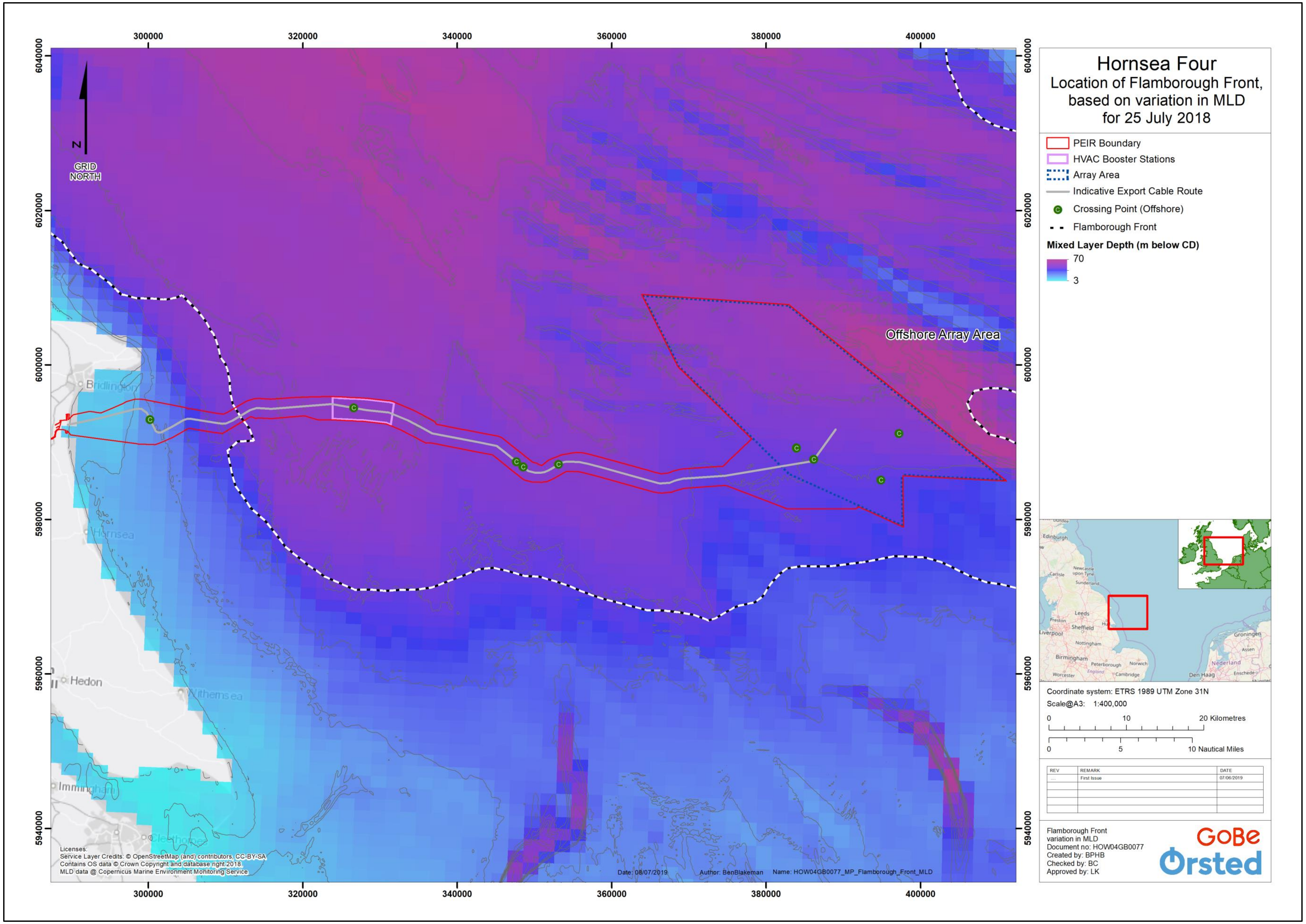

Figure 26: Location of Flamborough Front, based on variation in MLD for July 2018 (not to scale).

Hornsea Project One and Hornsea Project Two

3.4.3.9 At the time of producing the PEIR, all of the wind turbine foundations for Hornsea Project One have been installed. This project is located around 5.5 km to the south east of Hornsea Four, with the intervening area planned for Hornsea Project Two. Hornsea Three is located approximately 36 km to the east of Hornsea Four.

3.4.3.10 Hornsea Project One modifies the previous baseline by adding 174 8.1 m diameter monopile foundations across an area of around 407 km^2. These foundations individually create local blockage type effects on passing waves and tidal flows. Whilst the actual scale of any blockage effects on flows remains unknown at this time, the assumption remains that this is less than the conservative assumptions presented in the EIA report for Hornsea Project One and proportional to the smaller diameter structures. In this case, the assessment was based on a greater number (334) of larger diameter gravity base foundations assumed to be placed closer together. Offshore works for Hornsea Project Two are expected to commence in 2020 with the installation of 165 monopile foundations, also a reduction on the number (258) and scale (58 m diameter GBS) assessed in the respective EIA.

3.4.3.11 A review of array blockage effects on waves has been completed based on the comparison between pre- and post-construction observations to the north and south of the array. This review is presented in **Appendix C** and concluded no discernible changes in wave heights attributed to the presence of the monopile foundations of Hornsea Project One.

3.4.4 Summary of marine physical environment receptors within the offshore array study area

3.4.4.1 **Table 8** summarises the receptors associated with the offshore array study area. The potential sensitivity of the receptor is expressed prior to consideration of the scale of any impact related to the development.

Table 8: Marine physical environment receptors in the offshore array study area.

Receptor	Potential sensitivity to marine processes
Pipeline and cable crossings	Local scouring around rock berms
Flamborough Front	Changes in tidal mixing process which may inhibit formation of the front
Hornsea Project One (and Two)	Cumulative blockage effects with Hornsea Four

4 PEIR assessment

4.1 Overview

4.1.1.1 The Hornsea Four Scoping Report (Ørsted, 2018) together with the Scoping Opinion (The Planning Inspectorate, 2018) identify the issues for assessment of potential impact due to the proposed Hornsea Four development. The issues related to marine processes have been further clarified through the Evidence Plan discussions and are summarised in **Table 1**. These issues have the potential to create impact pathways on receptors in the marine environment. The receptors associated with the marine physical environment are summarised in **Table 5**, **Table 7** and **Table 8**.

4.1.1.2

4.2 Maximum Design Scenario for marine processes

4.2.1.1 Where multiple options remain for project development activities the definition of sources for marine processes is based on the Maximum Design Scenario (MDS). The MDS option represents the conservative case of any of the design options with an alternative option to the MDS considered to have a lesser environmental effect. The MDS for marine processes has been determined from a review of the Project Description for Hornsea Four (**Volume 1 Chapter 4: Project Description**).

4.2.1.2 The MDS is considered for activities that are planned for construction, operation and decommissioning phases.

4.2.2 MDS for Construction Phase

4.2.2.1 The MDS for construction related issues is defined by the greatest volumes of disturbed sediment occuring in the shortest period (highest rates of disturbance) from various seabed preparation activities which may create elevated levels of suspended sediment and subsequent deposition (smothering risk on a seabed receptor). These activities include:

- Levelling for foundations;
- Sandwave clearance for cable installation;
- Cable installation;
- Drilling for foundation piles; and
- Spoil disposal.

4.2.3 MDS for Operation Phase

4.2.3.1 During operation of the windfarm (the longest phase of the development) the main consideration for marine processes is blockage effects on waves, flows and sediment pathways from structures placed in the water column (including; foundations, subsea structures and rock armour at cable crossings), as well as consequential local scouring (if no scour protection is provided prior to installation of foundation).

4.2.3.2 Blockage can lead to the formation of wakes (retardation of flows with increased turbulence in a wake, flow seperation around large obstacles, diffraction and scattering of wave energy, etc) and the potential to modify sediment transport pathways in the far-field, including longshore transport.

4.2.3.3 The MDS for any array blockage effect is a product of the greatest number of closest spaced and widest foundations (with high solidity ratio[2]) that could potentially interfere with the normal passage of currents, waves amd sediment pathways.

4.2.3.4 During the operation phase there may also be various maintenance activities which have the potential to create short-term periods of distrubed sediments; however, these are considered to be minor in comparsion to those occuring during either the construction or decommisioning periods.

4.2.4 MDS for Decommissioning Phase

4.2.4.1 The MDS for decommissioning issues is relates to excavation activities which may lead to the greatest volumes of potential disturbed sediment in the shortest period (highest rates of distrubance), along with a consideration of subsequent seabed recovery to conditions which might have occured at this time in a baseline envrionment without the development.

4.2.5 Summary of MDS options for marine processes

4.2.5.1 **Table 9** provides details of the MDS options for marine processes. Where appropriate, the impacts have been split into sub-areas within the study area related to landfall, offshore export cable corridor (ECC) and offshore array activities.

[1] **Solidity ratio** is defined as the ratio of effective area (projected area of all the individual elements of a structure) of a frame normal to the wave, tidal flow or sediment transport direction divided by the area enclosed by the boundary of the frame. A solid structure will have a solidity ratio of 1, whereas an open frame lattice structure (e.g. jacket type) will generally have a much lower solidity ratio towards 0.2 (typical values between 0.1 and 0.3).

Table 9: Summary of MDS options for marine processes.

Impact and Phase	Embedded Mitigation Measures	Maximum Design Scenario
Construction **MP-C-1**. Seabed preparation activities (levelling, sandwave clearance, etc.) which may lead to a requirement for spoil disposal elsewhere creating elevated suspended sediment and potential smothering by deposition.	Primary: Co44, Co45	**a. Landfall area** Offshore cofferdam HDD exit pits require excavation of 2,500 m^3. **b. Offshore ECC** **Sandwave clearance** - Total sandwave clearance of 757,000 m^3 along a corridor of 99 km in length for 6 export cables. **HVAC foundations** Seabed preparation for Suction Caisson Jacket foundations requires removal of 171,735 m^3 for 3 * HVAC booster station foundations. **c. Offshore array area** **Sandwave clearance** –Total sandwave clearance of 961,000 m^3 which includes 77,000 m^3 for an additional 10 km of export cable within the offshore array. **180 WTG Foundations** Seabed preparation for Suction Bucket Jacket foundations requires removal of 2,134,440 m^3 for 180 wind turbine foundations. **9 OSS foundations** Seabed preparation for Suction Caisson Jacket (Medium OSS) & GBS (Large OSS) requires removal of 737,130 m^3 of spoil for 9 offshore sub-station foundations. **Offshore accommodation foundation** Seabed preparation for Suction Caisson Jacket (Medium OSS) requires removal of 57,245 m^3 of spoil for a single offshore accommodation platform foundation. **Total spoil in offshore array area = 3,889,915 m^3**
Construction **MP-C-2.** All direct sediment disturbance activities that may lead to locally raised suspended sediment concentrations at source (drilling, cable trenching, etc).	Primary: Co44, Co45	**a. Landfall area** Open cut trenching across the intertidal with tidal exchange (low water to high water to low water) flushing away lose materials determining a potential source of sediment from the trench and from any beach material cast aside. **b. Offshore ECC** **Cable trenching** – Cable installation along a length of 109 km for up to 6 cables releasing 3,903,000 m^3 into suspension by a Mass Flow Excavator (MFE). Values include the 10 km

Impact and Phase	Embedded Mitigation Measures	Maximum Design Scenario
		of export cable falling within offshore array area. Maximum duration of 24 months with a maximum trenching rate of 300 m/hr in soft soils. **HVAC Booster area** – drilling for Piled Jacket (Medium OSS) foundation option, releasing 4,618 m^3 for 3 foundations, representing 10% (of depth). **c. Offshore array area** **Cable trenching** - releasing 4,140,000 m^3 into suspension by MFE including the interconnector cables. Fastest excavation rate of 300 m/hr in soft soils. Single trenching vessel assumed for a sequential activity. **Drilling:** **180 WTG Foundations** – drilling for monopile foundation option, 127,235 m^3 for 180 foundations, representing 10% (of sites). Drilling activity considered to be sequential between sites. **9 OSS foundations** – drilling for Piled Jacket (Medium OSS) & Piled Jacket (Medium OSS), 13,854 m^3 for 9 foundations, representing 10% (of depth). Drilling activity considered to be sequential between sites. **Offshore accommodation** - drilling for Piled Jacket (Medium OSS) & Piled Jacket (Medium OSS), 1,540 m^3 for 1 foundation, representing 10% (of depth). **Total drill cutting arisings in offshore array area = 142,629 m^3**
Operation **MP-O-3** Scouring around foundations	Primary: Co82	**a. Landfall** Scouring around the base of cofferdams used to protect nearshore HDD exit pits. **b. Offshore ECC** **Rock berms at nearshore cable crossings** – Hornsea Four (up to 6 cables) will cross the export cable (up to 4 cables) for Creyke Beck seaward of Smithic Sands. **HVAC booster area** – risk for scouring in pre-scour protection period around three 75 m wide GBS (Box-type) foundations. **Rock berms at cable crossings** – 9 crossings over existing assets, potential for scouring dependent on rock size and grading to perimeter with heights up to 1.5 m. **c. Offshore array area** **180 WTG Foundations** – 3-legged suction bucket jacket with 20 m diameter buckets 5 m proud of seabed, with potential for group scour between legs.

Impact and Phase	Embedded Mitigation Measures	Maximum Design Scenario
		9 OSS foundations – 3 large box-type GBS of 150 m width and 6 medium box-type GBS width 75m. **Offshore accommodation** – 75 m wide GBS (box-type). **Rock berms at cable crossings** – 40 potential crossings over new pipelines, potential for scouring dependent on rock size and grading to perimeter. Some alignments may locally inhibit bedload transport.
Operation **MP-O-4** Turbulent wakes from foundations interfering with remote receptors, e.g. Flamborough Front		**a. Landfall** Wakes will form locally around the cofferdams used to protect offshore HDD exit pits. **b. Offshore ECC** **HVAC booster area** – largest solid structure in the vertical plane is the 75 m width GBS (Box-type). The wake formation may depend on the orientation of this structure to incident flows and waves as well as the minimum spacing between structures and the layout of structures. A minimum separation distance of 100 m is likely to result in wake-wake interactions and a larger cumulative effect between all 3 structures. **Rock berms** – all in water depths between 40 to 50 m CD. No likely wake effects. **c. Offshore array area** **180 WTG Foundations** – The foundation considered to have the greatest blockage effect for MDS is the 3-legged suction bucket jacket with 20 m diameter buckets 5 m proud of the seabed with a leg separation of 45 m at the seabed (and total effective width of 65 m including buckets) tapering to 25 m at the sea surface. **9 OSS foundations** –for the six OSS (small/medium) the 75 m box-type foundation has the greatest blockage. For the (3) large OSS foundations, the large 150 m GBS box -type foundation has the largest blockage. **Offshore accommodation** – 75 m box-type foundation has the greatest blockage. The total blockage effect for the whole array is also a function of the number, spacing and layout of all 190 foundations. The principles for the array layout are based on a minimum WTG separation of 810 m.
Operation **MP-O-5** Changes to waves affecting coastal morphology		**a. Landfall** Cofferdams used to protect offshore HDD exit pits will have a temporary effect on waves reaching the coastline

Impact and Phase	Embedded Mitigation Measures	Maximum Design Scenario
		b. Offshore ECC **HVAC booster area** – largest solid structure in the vertical plane is the 75 m width GBS (Box-type). These structures have the potential to block, reflect and scatter incident waves. A minimum separation distance of 100 m is likely to result in interactions and a larger cumulative effect between structures. **Rock berms** – 9 cable crossings further offshore in water depths between 40 to 50 m below CD. The additional Creyke Beck cable crossing is in around 20 m below CD and may have an influence on nearshore waves. **c. Offshore array area** **180 WTG Foundations** – The foundation considered to have the greatest blockage effect is the 3-legged suction bucket jacket with 20 m diameter buckets 5 m proud of the seabed with a leg separation of 45 m at the seabed (total effective width of 65 m) tapering to 25 m at the sea surface. **9 OSS foundations** –for the six (6) OSS (small/medium) the 75 m box-type GBS foundation has the greatest blockage. For the three (3) large OSS foundations, the large 150 m width GBS box -type foundation has the largest blockage. **Offshore accommodation** –75 m width box-type GBS foundation has the greatest blockage.
Operation **MP-O-6.** Changes to nearshore sediment pathways		**Rock berms at cable crossings** – Hornsea Four will cross the export cable for Creyke Beck seaward of Smithic Sands. Maximum berm height of 1.5 m placed in around 20 m CD. Rock protection assumed for 10% of offshore ECC cable length in addition to any cable crossings. **HVAC Booster area** – three (3) large BGS box-type foundations closely spaced at 100 m may dampen nearshore waves and modify nearshore sediment transport
Decommissioning **MP-D-2.** All direct sediment disturbance activities during decommissioning activities that may		The assumption is for comparable (or lesser) rates of sediment disturbance to the those described for installation of foundations and cables. Removal of structures will also remove their blockage effects.

Impact and Phase	Embedded Mitigation Measures	Maximum Design Scenario
lead to locally raised suspended sediment concentrations at source		

4.3 Seabed preparation activities

4.3.1.1 Seabed preparation is defined here as activities which may excavate material from source with a requirement for spoil disposal elsewhere. The excavation and disposal activities may each create elevated levels of suspended sediment and spoil disposal may lead to rapid smothering by large volumes of sediment falling to the seabed.

4.3.1.2 Seabed preparation activities planned for the construction phase include provisions for:

- Seabed excavation to install cofferdams at landfall for up to eight HDD exit pits;
- Sandwave clearance prior to cable laying along the offshore ECC and within the offshore array. This process will target mobile bedforms that are unfavourable to cable installation due to their gradient; and
- Seabed preparation (levelling) for foundations in both the HVAC booster area and offshore array. This activity aims to level the seabed to aid the installation of foundation bases that need an even surface. The levelling may require removing larger bedforms or excavating the consolidated surface sediments that are uneven.

4.3.1.3 The likelihood is that excavations in shallow water for the HDD exit pits will be achieved using a backhoe dredger, or similar.

4.3.1.4 The assumption for sandwave clearance and levelling is that a trailing suction hopper dredger (TSHD) would be used, this is also considered to the conservative option. Alternative methods are possible, such as jetting or mass flow excavator (MFE), but they would only disturb material from the seabed, and this would limit the time for any material to fall through the water column and become advected. In contrast, the dredging discharges overspill into surface waters which may create a larger sediment plume. In addition, the spoil dumping introduces a high volume of sediment in a more focused but separate location, rather than along the path of area being cleared. Given that some clearance operations are planned for completion a year before cable and foundations are installed, the advantage of a TSHD is that cleared features have potentially a lesser opportunity of being re-established, depending on where the spoil is disposed.

4.3.1.5 The TSHD process hydraulically sucks sediments from the seabed without creating any significant near-bed sediment spill layer (Bray, Environmental aspects of dredging, 2008). The sediment-water mix is pumped from the seabed into the hopper and the excess water is allowed to overspill (overflow) leaving the (relatively coarser) sediment to fill the hopper.

4.3.1.6 The overspill process is required to maximise the hopper load; the majority of overspill is likely to be seawater but some of the finer sediments will also be present. The overspill creates a sediment plume that would initially fall towards the seabed as a density flow due to the content of sediment, this is often described as the active phase of the plume. Once the overspill has been entrained by the receiving water then a passive phase of plume dispersion into the far-field will take place which is governed by advection and diffusion due to tidal flows. Eventually the sediment in the plume falls out of suspension and settles onto the seabed. **Figure 27** provides a schematic representation of the sediment plume during the dredging activity.

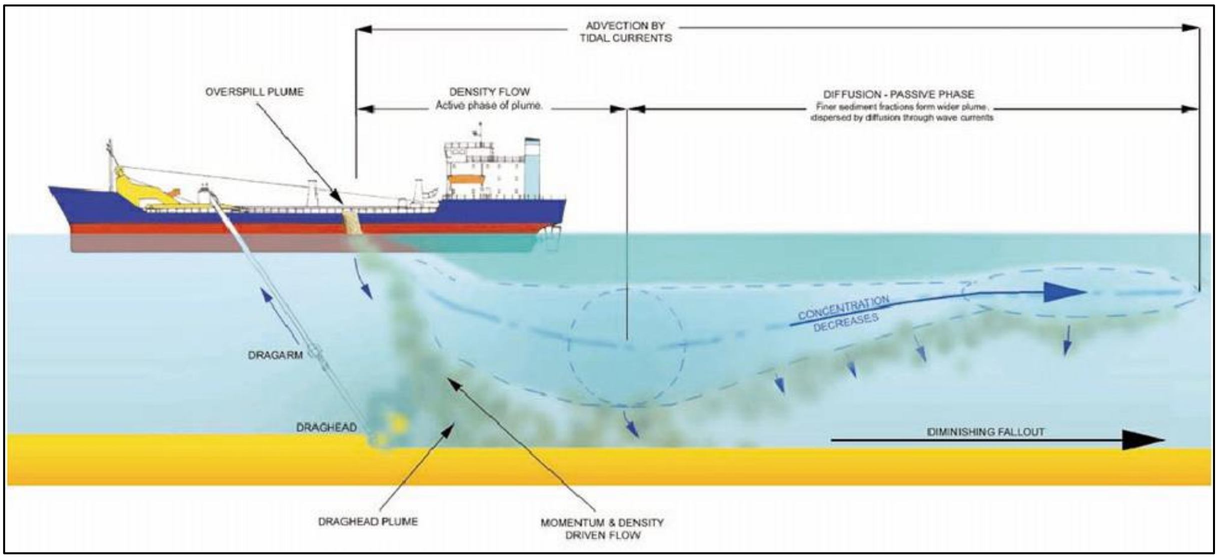

Figure 27: Schematic of sediment plume development during dredging activity (The Crown Estate and BMPA, 2009).

4.3.1.7 Once the hopper has sufficient sediment then the material removed from the seabed would be disposed of close by as spoil, and within the red line boundary defining Hornsea Four. This would be achieved as a surface release by opening the hopper doors. The same sediment types are therefore being returned to the same ambient seabed sediment environments.

4.3.1.8 The time taken to fully discharge the hopper load, the transit of the dredger during this time, the sediment volume, sediment grain sizes, local water depth and ambient flow conditions will all influence the formation of any sediment plume and the shape of any spoil deposit in terms of area of seabed involved and height of any mound, as well as the longevity of the feature thereafter.

4.3.1.9 For present purposes, the assumed hopper volume is 11,000 m^3, which is consistent with the assumptions used for Hornsea Project One, Two and Hornsea Three. This assumption for sand dredging with a conventional TSHD, discharge via the bottom doors takes approximately five to ten minutes[3].

4.3.1.10 The disposal of spoil replicates the active phase of the sediment plume described for the overspill but since the majority of the discharge is sediment rather than water, the active phase is more intense achieved over a shorter period. The likelihood is for any remaining finer sediments to form a passive sediment plume from the location of the release with the coarser material falling directly to the seabed.

4.3.1.11 Because the plume is made up of sediment particles which are denser than seawater then the plume will behave with negative buoyancy. The longevity of the plume in the water

[3] https://www.startdredging.com/dredging-cycle-trailing-suction-hopper-dredger/

column is determined by the particle settling velocity and the ambient flows which offer the potential for maintaining suspension or resuspension of the component sediment types.

4.3.2 Excavation of Landfall HDD exit pits

Sediment types

4.3.2.1 Depending on their precise location and distance offshore, the likelihood is that these shallow water excavations can be achieved using a backhoe dredger. The geophysical evidence suggests a thin surface layer of sands and gravels overlying glacial till of stiff clays. The target depth of exit pits at 5 m means that most of the excavated soil type is likely to be the glacial till.

Maximum sediment volumes

4.3.2.2 The MDS volume for cofferdam excavation is a total of up to 2,500 m^3 for 8 exit pits. This equates to an average volume of up to 312.5 m^3 per pit. A typical backhoe dredger can excavate up to 100 to 500 m^3/hr, but this depends greatly on the bucket size, soil types and configuration of the required excavation.

4.3.2.3 The excavation operation for each pit is likely to be sequential, limiting the chance for one large spill event. The time required to excavate each pit and the time between the next excavation are not known at this time.

4.3.2.4 The assumption is that the excavated material will be side-cast and left on the seabed. When back-filling of the excavated pit is necessary then sediment will be recovered from the surrounding area.

Local hydrodynamic conditions

4.3.2.5 The shallow water conditions are likely to be dominated by shoaling waves which eventually break along the beach to drive longshore drift for sands. The same process washes the fine sediments off the beach to create a nearshore plume.

4.3.2.6 Nearshore suspended sediment concentrations (to around 10 km from the coast) are also likely to remain higher than further offshore with background levels in the range 2 to 14 mg/l.

4.3.2.7 Tidal range is also an important consideration as this may modulate the effects local to the HDD exit pits, with strongest influences around low water.

Fate of excavated sediments

4.3.2.8 Depending on the method of excavation and the type of material being removed (consolidated or unconsolidated), the chances remain for some of the sediment to be spilled into the sea during excavation. Any fine sediments would be quickly dispersed away to become part of the nearshore sediment plume. Coarser sediments would drop back to the seabed.

4.3.2.9 The material that is cast aside of the excavated pit would be subject to wave and tidal action with unconsolidated fine sediments and sands becoming assimilated into the locally active sediment transport process. The amount of sediment loss from a side-cast mound would depend on the sediment composition, the area of placement, local wave and tidal processes, as well as the period until backfilling. Any gravels or consolidated clays would most likely experience the lowest amount of loss.

4.3.2.10 Once duct installation is complete the exit pits will be backfilled with sediments recovered from the surrounding seabed. This operation would expect to take place prior to cofferdam removal to assist compaction of the sediment in a dry environment, and this would also further limit any sediment spill. Once cofferdams are removed the sediments are expected to de-water and re-compact overtime.

4.3.3 Sandwave clearance

Sediment types

4.3.3.1 By definition sandwaves are formed of sand, with the grade of sand likely to be in dynamic equilibrium with the local environmental conditions that are able to move and maintain the bedform features.

4.3.3.2 The presence of sandwaves along the offshore ECC is being confirmed by the geophysical survey which already notes bedform features mainly towards the offshore array (Bibby HydroMap, 2019). The possibility is for further sandwaves to be present elsewhere along the offshore ECC so the clearance operations are not limited to any specific locations at this time and will be guided when the full set of geophysical information is available.

4.3.3.3 In contrast, sandwaves are quite evident in the offshore array, apart from the more southerly section (Gardline, 2019). Some of these sandwaves are relatively large, especially on the north-western side of the offshore array which merges into The Hills.

4.3.3.4 As a dynamic feature rather than a stable consolidated deposit, the regular turn-over of sands in motion on a sandwave also tends to create a well-sorted sandy sediment with relatively little fines. **Figure 28** provides an example particle size analysis for a grab sample coincident with an identified sandwave crest for a site close to the offshore array. In this example, there is a predominance of very well sorted medium sands.

SAMPLE STATISTICS

SAMPLE IDENTITY: **+54+000/211/GS/1** ANALYST & DATE: ,

SAMPLE TYPE: Unimodal, Very Well Sorted TEXTURAL GROUP: Sand

SEDIMENT NAME: Very Well Sorted Medium Sand

	μm	ϕ
MODE 1:	302.5	1.747
MODE 2:		
MODE 3:		
D_{10}:	233.0	1.522
MEDIAN or D_{50}:	294.4	1.764
D_{90}:	348.2	2.101
(D_{90} / D_{10}):	1.494	1.381
$(D_{90} - D_{10})$:	115.1	0.579
(D_{75} / D_{25}):	1.233	1.187
$(D_{75} - D_{25})$:	61.78	0.302

GRAIN SIZE DISTRIBUTION			
GRAVEL:	0.0%	COARSE SAND:	0.8%
SAND:	100.0%	MEDIUM SAND:	88.2%
MUD:	0.0%	FINE SAND:	9.9%
		V FINE SAND:	1.1%
V COARSE GRAVEL:	0.0%	V COARSE SILT:	0.0%
COARSE GRAVEL:	0.0%	COARSE SILT:	0.0%
MEDIUM GRAVEL:	0.0%	MEDIUM SILT:	0.0%
FINE GRAVEL:	0.0%	FINE SILT:	0.0%
V FINE GRAVEL:	0.0%	V FINE SILT:	0.0%
V COARSE SAND:	0.1%	CLAY:	0.0%

	METHOD OF MOMENTS			FOLK & WARD METHOD		
	Arithmetic μm	Geometric μm	Logarithmic ϕ	Geometric μm	Logarithmic ϕ	Description
MEAN ($\bar{x}$):	298.3	286.1	1.805	294.4	1.764	Medium Sand
SORTING (σ):	73.56	1.265	0.340	1.214	0.280	Very Well Sorted
SKEWNESS (Sk):	5.079	-1.174	1.174	-0.236	0.236	Fine Skewed
KURTOSIS (K):	74.12	11.82	11.82	1.587	1.587	Very Leptokurtic

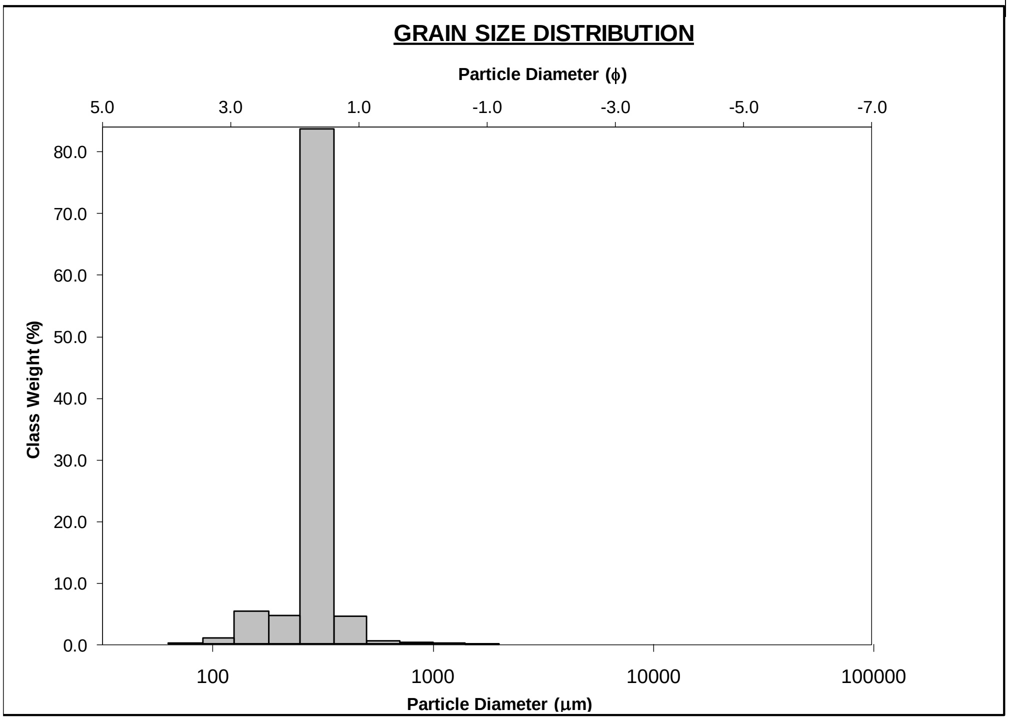

Figure 28: Illustrative particle size information coincident with sandwave feature.

Maximum sediment volumes

4.3.3.5 The MDS volume for sandwave clearance is a total of up to 757,000 m^3 across 6 export cables and sweeping a width of 30 m per cable. In addition, sandwave clearance in the offshore array accounts for up to 961,000 m^3, this includes a 10 km section of the export cable.

4.3.3.6 For uniform sand which is loose (i.e. not a consolidated deposit) a dry bulk density value would be around 1.43 tonnes/m^3 (Terzaghi, Peck, & Mesri, 1996), which suggests a mass equivalent of up to approximately 1,082,510 tonnes of sediment would need to be removed by sandwave clearance along the offshore ECC and up to approximately 1,374,230 tonnes within the offshore array.

4.3.3.7 The efficiency of the dredger discharging overspill means that less than 100% of the excavated material remains in the hopper and the rest is discharged with the overspill (overflow losses). Overspill losses depended on many issues, not least pumping rates and sediment types. For medium sand the overspill losses are likely to be relatively low and limited to any finer grained sands and silts present in the sediment. A conservative value of 5% has been assumed for average overspill losses over the loading cycle, noting that at commencement this value may be lower and when the hopper is reaching capacity this value may be higher.

4.3.3.8 Once sediment is excavated by a dredger and transferred into the hopper there is an immediate bulking factor which accounts for the volume difference between a consolidated deposit and a hopper load of loose material. In addition, a hopper capacity of 11,000 m^3 will be carrying a mixture of sediment and seawater. A bulking factor of 1.20 has been assumed for present purposes which is mid-range of 1.15 to 1.25 for sand, medium soft to hard (Bray, Bates, & Land, Dredging: A Handbook for Engineers 2nd Edition, 1996).

4.3.3.9 Based on these stated assumptions, there are expected to be up to approximately 78 hopper loads required to provide for sandwave clearance at any location required along the entire offshore ECC. This is equivalent to approximately one hopper load every 8.3 km for each of the 6 export cables. In reality, the chance for sandwaves to be present along the entire offshore ECC is quite low and not all of the dredging allowance is likely to be required.

4.3.3.10 For the offshore array, there are expected to be up to approximately 100 hopper loads required, although the chances for needing sandwave clearance in the southerly section of the array appear to be much reduced.

Local hydrodynamic conditions

4.3.3.11 The water depths and flow conditions along the offshore ECC into the offshore array show some variability which will influence both the fate of overspill and spoil disposal. Four representative sites have been selected to demonstrate this variability; nearshore towards Smithic Sands, mid-section around the HVAC booster area, an offshore section of the offshore ECC towards the offshore arrayand a site central to the offshore array.

4.3.3.12 **Table** 10 provides a summary of the representative hydrodynamic conditions at these locations.

Table 10: Representative hydrodynamic conditions for sandwave clearance activity.

Location	Water depth (m below CD)	Spring / neap tidal excursion (km)	Annual variability of average wave height (m)	Annual range of SPM (mg/l)
Nearshore	10 to 20	14 / 7	0.77 to 1.25	2 to 14
Mid-section	50	12 / 6	1.01 to 1.86	1 to 2
Offshore end of ECC	40	8.6 /4.3	1.1 to 2.09	1 to 3
Central to offshore array	50	8.3/4.1	1.1 to 2.14	1 to 3

4.3.3.13 The tidal excursion is normally defined as the limit of lagrangian movement of a water particle over one tidal cycle. This limit is normally attained during either the ebb or flood phase of the tide, a period which is typically around 6 hours and 10 minutes. Where the tide is asymmetric in magnitude and/or duration between flood and ebb phases then the excursion distances can also be slightly different.

Overspill considerations

4.3.3.14 The overspill is considered to occur local to the sandwave clearance operation along a section of the export cable corridor or within the offshore array. In the average dredging cycle, this is typically a 7.5 km section of the offshore ECC where sandwaves are present, although where such features are sparse this distance may increase, or if there are very large sandwaves this distance may shorten.

4.3.3.15 The finer sediment sizes of very fine and fine sands are more likely to be discharged in the overspill rather than the larger particle sizes of medium and coarse sand which will tend to be retained in the hopper, noting this hydraulic sorting cannot be considered as 100% efficient.

4.3.3.16 Overspill will form a plume largely made up of the finer sediment fractions which will be advected away by tidal flows. The duration of the overspill event per dredging cycle is likely to be comparable to the time required to fill the hopper. An indicative period of 4 hours is assumed to fill a 11,000 m^3 hopper.

4.3.3.17 The main axis of the plume trajectory will be governed by tidal advection with reduced concentrations around this axis due to dispersion and diffusion mixing processes spreading the plume. Plume concentrations will reduce over distance due to increased mixing and material falling out of suspension. During a neap tide the plume will be advected over a shorter distance than a spring tide, and since the rate of mixing will be less at these times due to weaker flows, then suspended sediment concentrations can be expected to be proportionally higher. On spring tides, the plume will spread further and have a proportionally lower concentration. Winds would expect to have some influence on surface material, either by increasing mixing and/or modifying the plume trajectory.

4.3.3.18 Table 11 provides a summary of representative sediment types expected to be present in the overspill, their individual fall velocities from the sediment plume phase and the time required to reach the seabed.

Table 11: Representative sediment types in sandwave overspill.

Sediment type	Size range (mm)	Representative size (mm)	Settling velocity (m/s)	Time to fall out of suspension 10 / 20 / 40 / 50 m (minutes)
Medium sand	0.25 to 0.50	0.38	0.050	3 / 7 / 13 / 17
Fine sand	0.125 to 0.250	0.18	0.017	10 / 20/ 39 / 49
Very fine sand	0.063 to 0.125	0.09	0.004	38 / 76 / 151 / 190

4.3.3.19 The fine and medium sand fractions will settle out of suspension relatively quickly and have limited time to advect and disperse. Even if this spill event occurred at peak spring flows the distance covered by fine sands in 49 minutes would still be restricted to around 1.8 km from source. The very fine sand may take over 3 hours to fall out of suspension in depths of 50 m at the representative site central to the offshore array. This is a period equivalent to half a tidal excursion from slack water to peak flows; however, this sediment size is generally the least abundant in the material to be dredged so concentrations will be limited.

4.3.3.20 As a general consideration, suspended sediment concentrations within sediment plumes can be in the order of hundreds of mg/l in the vicinity of the dredger, reducing to tens of mg/l with distance (CIRIA, 2000), but also quickly dissipating in time after release. Given the likely loading and dumping cycle each overspill event is expected to disperse away as a separate plume.

Hornsea Project One and Hornsea Project Two

4.3.3.21 Hornsea Project One and Hornsea Project Two use a common export cable corridor and represent the closest comparable projects to Hornsea Four. Sandwave clearance based on TSHD was modelled as part of their assessments (SMart Wind, 2013), (SMart Wind, 2015). These scenarios assumed a 1.5% fine sediment content in the sandwave. The results indicated a plume footprint of fine sediments aligned to the main tidal flow direction with some asymmetry due to the difference between flood and ebb flows. A depth-averaged suspended sediment concentration of up to 40 mg/l was predicted 200 m from the cable route (source). Due to the settling and re-suspension of fine sediments, the predicted maximum extent of the sediment plume (> 2 mg/l above background SSC levels) was 16 km northwest and 17 km southeast from the point of release (i.e. a full tidal excursion for this location). The peak predicted (depth-averaged) increase above background concentration was 37 mg/l along the dredge track.

4.3.3.22 The deposition of fine sediment under low flow conditions was predicted to be less than 2 mm. Based on minimum thickness of 0.5 mm, the area of deposition extended 60 m to the northwest and 250 m to the southeast of the cable route, however, under higher flow conditions this material was dispersed away.

Spoil disposal

4.3.3.23 Once the dredger moves to discharge a full hopper load close by, the majority of the finer sediments are expected to have already been lost as overspill. The remaining sediments in the hopper should be predominantly composed of the coarser sediment fractions, meaning that the disposal of the spoil is likely to have a lesser concern in the formation of any sediment plume. In contrast, the majority of the spoil will fall more quickly to the seabed with limited opportunity to disperse (but correspondingly leading to a greater depth of accumulation at the seabed and therefore a higher risk of smothering of any benthic receptors).

4.3.3.24 Table 12 provides a summary of representative sediment types expected to be present in the spoil, their individual fall velocities and the time required to reach the seabed.

Table 12: Representative sediment types in sandwave spoil disposal.

Sediment type	Size range (mm)	Representative size (mm)	Settling velocity (m/s)	Time to fall out of suspension 10 / 20 / 40 / 50 m (minutes)
Coarse sand	0.50 to 1.00	0.75	0.093	2 / 4 / 7 / 9
Medium sand	0.25 to 0.50	0.38	0.050	3 / 7 / 13 / 17

4.3.3.25 The depth of deposition and area covered will be determined by the course of the vessel in the period of opening hopper doors, the tidal flows at the time and the relative composition of the sediment being disposed. The vessel speed could also act as means to ensure the deposition of spoil is more widely dispersed than opening the hopper doors when the vessel is stationary.

4.3.3.26 Once deposited, the sand is likely re-join the same transport environment that created and moved the bedforms. This process may winnow down any spoil mound; however, in the offshore array sediment mobility is typically limited to the spring tides which may lead to a slower winnowing process. For the nearshore environment where flows are typically stronger, the mobility can be expected to be higher and will also become influenced by waves.

Hornsea One and Two

4.3.3.27 For spoil disposal, the coarser sands and gravels displaced by the dredging activity were considered not to disperse by tidal currents and were predicted settle rapidly in close proximity to the point of disposal. A single placement from a hopper with a volume of 11,650 m^3 was considered to lead to an area of deposition approximately 200 m in diameter and up to 1 m in height at the centre of the mound. The available disposal area was considered sufficient so that multiple placements of spoil could be separated out to avoid overlap (SMart Wind, 2013), (SMart Wind, 2015).

4.3.4 Seabed levelling in HVAC booster area

Sediment types

4.3.4.1 Based on regional sediment mapping from EMODnet, the generalised description of surficial sediment types for the HVAC booster area is mainly gravely sand with the eastern side becoming slightly gravelly sand. The same sediment types and distributions are largely confirmed by the recent geophysical survey. The Creyke Beck export cable route geophysical survey also crossed a similar area to the north and describes the seabed as a veneer of gravelly sand with occasion megaripples, cobbles and boulders and sub-cropping bedrock and till (ForeWind, 2013).

4.3.4.2 According to the Folk classification scheme for sediment types (Folk, 1954), both gravely sand and slightly gravelly sand sediment types can potentially contain up to 9% mud by content. Surficial sediment samples which are in the general area, and on the same substrate type, suggest the mud content is typically much lower and between 0.4 to 2%, although the mud content may be higher in material slightly below the surface of the seabed. The gravel content generally represents 1.2 to 26% of the sample with particle sizes mainly in the range 4 to 8 mm. The remaining sand content is characteristically fine to medium sized sands. No allowance is made here for variability of sediment types over the excavation depth.

Maximum sediment volumes

4.3.4.3 The MDS volume for seabed levelling within the HVAC booster area is a maximum value of up to 171,735 m^3 for the three six-legged Suction Caisson Jacket (Medium OSS) foundation option. The mass equivalent for this amount of sediment is dependent on the soil characteristics to be removed, such as; voids ratio, porosity and water content. Even for similar sediment types these values can vary dependent on the compaction and consolidation of the sediment. For dense, mixed-grained sand a dry bulk density value of 1.86 tonnes/m^3 is appropriate, for mixed-grained glacial till a dry bulk density value would be around 2.12 tonnes/m^3 (Terzaghi, Peck, & Mesri, 1996). A bulk density value of 2.00 tonnes/m^3 has therefore been applied for illustrative purposes which suggests a mass equivalent of up to approximately 114,900 tonnes of sediment is removed per foundation (343,470 tonnes in total for all three foundations).

4.3.4.4 The efficiency of the dredger discharging overspill means that less than 100% of the excavated material remains in the hopper and the rest is discharged with the overspill (overflow losses). Overspill losses depended on many issues, not least pumping rates and sediment types. For gravely sand and slightly gravelly sand the overspill losses are likely to be relatively low and limited to any finer grained sands and silts present in the sediment. A conservative value of 5% is assumed for average overspill losses over the loading cycle, noting that at commencement this value may be lower and when the hopper is reaching capacity this value may be higher. This equates to up to approximately 5,725 tonnes of the excavated material lost per foundation, with up to approximately 108,766 tonnes taken into the hopper for spoil disposal.

4.3.4.5 Once sediment is excavated by a dredger and transferred into the hopper there is an immediate bulking factor which accounts for the volume difference between a consolidated deposit and a hopper load of loose material. In addition, a hopper capacity of 11,000 m^3 will be carrying a mixture of sediment and seawater. A bulking factor of 1.25 has been assumed for present purposes which is mid- range of 1.15 to 1.35 for sand/gravel/clay mixtures (Bray, Bates, & Land, Dredging: A Handbook for Engineers 2nd Edition, 1996).

4.3.4.6 Based on these stated assumptions, there are expected to be up to approximately 19 hopper loads required in total.

Local hydrodynamic conditions

4.3.4.7 The water depths at the HVAC booster area are generally around 50 m below CD with predicted tidal flows reaching a peak of around 0.84 m/s on mean spring tides and 0.42 m/s on mean neaps. The equivalent tidal excursions are around 12 and 6 km, respectively.

4.3.4.8 Flow measurements at site B0592164, just to the west of the HVAC booster area, indicate speeds can exceed 1 m/s on the flood tide when the tidal range is greater than MSR. Equivalent ebb speeds tend to be slightly less suggesting a net transport to the south-west, in line with generalised sand transport pathways for this area. The equivalent maximum flood tide excursion is determined as 12.6 km.

4.3.4.9 The tidal excursion is normally defined as the limit of lagrangian movement of a water particle over one tidal cycle. This limit is normally attained during either the ebb or flood phase of the tide, a period which is typically around 6 hours and 10 minutes.

Overspill considerations

4.3.4.10 The overspill is considered to occur locally to the seabed preparation area, notionally over an area of 111 m by 111 m square for each foundation.

4.3.4.11 The finer sediment sizes of muds, very fine sands and fine sands are likely to favour being discharged in the overspill rather than the larger particle sizes of medium sand through to gravels which are likely to favour being retained in the hopper, noting this hydraulic sorting cannot be considered as 100% efficient.

4.3.4.12 Overspill will form a plume largely made up of the finer sediment which will be advected away by tidal flows. The duration of the overspill event per dredging cycle is likely to be comparable to the time required to fill the hopper. An indicative period of 4 hours is assumed to fill a 11,000 m^3 hopper.

4.3.4.13 The main axis of the plume trajectory will be governed by tidal advection with reduced concentrations around this axis due to dispersion and diffusion mixing processes spreading the plume. Plume concentrations will reduce over distance due to increased mixing and material falling out of suspension. During a neap tide the plume will be advected over a shorter distance than a spring tide, and since the rate of mixing will be less at these times due to weaker flows, then suspended sediment concentrations can be expected to be proportionally higher. On spring tides, the plume will spread further and have a proportionally lower concentration. Winds would expect to have some influence on surface material, either by increasing mixing and/or modifying the plume trajectory.

4.3.4.14 Table 13 provides a summary of representative sediment types expected to be present in overspill, their individual fall velocities from the sediment plume phase and the time required to reach the seabed, assuming a depth of 50 m below CD.

Table 13: Representative sediment types in HVAC booster area foundation overspill.

Sediment type	Size range (mm)	Representative size (mm)	Settling velocity (m/s)	Time to fall out of suspension (minutes)
Medium sand	0.25 to 0.50	0.38	0.050	17
Fine sand	0.125 to 0.250	0.18	0.017	49
Silts / muds	< 0.063	< 0.05	0.001	Remains in suspension

4.3.4.15 The fine and medium sand fractions will settle out of suspension relatively quickly and well-within the distance of a full tidal excursion, and most likely within 1 or 2 km of the discharge position. The short-lived and localised fate of this material will not create any concerns for reduced turbidity.

4.3.4.16 Only the finer material, represented here as silts and muds with a particle size < 0.063 mm, is unlikely to settle out of suspension quickly due to the weak settling velocities generally exceeded by the upward turbulent component of local flows. This material will rapidly disperse across a full tidal excursion leading to reduced concentrations over time and distance from source.

4.3.4.17 As a general consideration, suspended sediment concentrations within sediment plumes can be in the order of hundreds of mg/l in the vicinity of the dredger, reducing to tens of mg/l with distance (CIRIA, 2000), but also quickly dissipating in time after release. Given the likely loading and dumping cycle each overspill event is expected to disperse away as a separate plume.

Spoil disposal

4.3.4.18 Once the dredger moves to discharge a full hopper load close by, the majority of the finer sediments are expected to have already been lost as overspill. The remaining sediments in the hopper should be predominantly composed of the coarser sediment fraction, meaning that the disposal of the spoil is likely to have a lesser concern in the formation of any sediment plume. In contrast, the majority of the spoil will fall more quickly to the seabed with limited opportunity to disperse (leading to a greater depth of accumulation at the seabed and therefore a higher risk of smothering of any benthic receptors).

4.3.4.19 Table 14 provides a summary of representative sediment types expected to be present in the spoil, their individual fall velocities and the time required to reach the seabed, assuming a depth of 50 m below CD.

Table 14: Representative sediment types in HVAC booster area spoil.

Sediment type	Size range (mm)	Representative size (mm)	Settling velocity (m/s)	Time to fall out of suspension (minutes)
Fine Gravel	4 to 8	3	0.216	4
Coarse sand	1 to 2	1.50	0.147	6
Medium sand	0.25 to 0.50	0.38	0.050	17

4.3.4.20 The depth of deposition and area covered will be determined by the vessel track in the period of opening hopper doors, the tidal flows at the time and the relative composition of the sediment being disposed of between sands and gravels (which will determine the angle of repose, nominally 25 to 30° for sandy gravel). The vessel speed could also act as means to ensure the deposition of spoil is more widely dispersed than opening the hopper doors when the vessel is stationary.

4.3.4.21 Once deposited, the coarse sand and fine gravel are unlikely to be remobilised by the local tidal flows, whereas the medium sands are only likely to be remobilised when flows exceed mean neap tides and for material that is not covered and armoured by the immobile coarser sediment sizes.

4.3.5 Seabed levelling in offshore array area

Sediment types

4.3.5.1 Based on regional sediment mapping from EMODnet, the generalised description of surficial sediment types for the offshore array area is mainly sand with some patches of slightly gravelly sand and gravelly sand. The same sediment types and distributions are largely confirmed by the recent geophysical survey (Gardline, 2019).

4.3.5.2 The content of fines (material < 0.063 mm) determined by grab samples across the offshore array is generally low (0 to 10.1%, and typically < 5%) apart from two locations on the eastern boundary where the content of fines increases to 13.7 and 15.3%. These areas are described as gravelly muddy sand and represent an area without any cover of Holocene sands and are interpreted as firm to stiff glacial till of the Bolders Bank formation (Gardline, 2019).

4.3.5.3 The content of gravels (material > 2mm) determine by grab samples across the offshore array is generally low (0 to 9.1%, and typically < 5%) apart from the same two locations on the eastern boundary where the content of gravel increases to 15.4 and 23.8%.

4.3.5.4 No allowance is made here for variability of sediment types over the excavation depth.

Maximum sediment volumes

4.3.5.5 The MDS volume for seabed preparation (levelling) within the offshore array is a maximum value of up to 2,134,440 m^3 for 180 Suction Bucket Jacket (WTG-type) foundations, equivalent to up to 11,858 m^3 per foundation. In addition, levelling is also required for offshore substations and an accommodation platform. In this case, up to 794,375 m^3 for six Suction Caisson Jacket (Medium OSS), three GBS (Large OSS) and one Suction Bucket Jacket (Medium OSS) for the accommodation platform. This is a total amount of up to 2,928,815 m^3 of sediment removal.

4.3.5.6 The mass equivalent for this amount of sediment is dependent on the soil characteristics to be removed, such as; voids ratio, porosity and water content. Even for similar sediment types these values can vary depending on the compaction and consolidation of the sediment. For areas with dense, mixed grained sand a dry bulk density value of 1.86 tonnes/m^3 is appropriate, for loose mixed grained sand a dry bulk density value would be around 1.59 tonnes/m^3 (Terzaghi, Peck, & Mesri, 1996). For areas without sand, or where the sand layer is thin, a dry bulk density relevant to excavating glacial till, very mixed grained would be 2.12 tonnes/m^3. A bulk density value of 1.75 tonnes/m^3 has been applied for illustrative purposes which suggests a mass equivalent of up to approximately 5,125,426 tonnes of sediment would be dredged from across the array for seabed levelling.

4.3.5.7 The efficiency of the dredger discharging overspill means that less than 100% of the excavated material remains in the hopper and the rest is discharged with the overspill (overflow losses). Overspill losses depended on many issues, not least pumping rates and sediment types. For gravely sand and slightly gravelly sand the overspill losses are likely to be relatively low and limited to any finer grained sands and silts present in the sediment. A conservative value of 5% has been assumed for average overspill losses over the loading cycle, noting that at commencement this value may be lower and when the hopper is reaching capacity this value may be higher. This equates to up to approximately 256,271 tonnes of the excavated material lost as overspill and up to approximately 4,869,155 tonnes is taken away by hopper loads for spoil disposal.

4.3.5.8 Once sediment is excavated by a dredger and transferred into the hopper there is an immediate bulking factor which accounts for the volume difference between a consolidated deposit and a hopper load of loose material. In addition, a hopper capacity of 11,000 m^3 will be carrying a mixture of sediment and seawater. A bulking factor of 1.20 has been assumed for present purposes which is mid-range of 1.15 to 1.25 for sand, medium soft to hard (Bray, Bates, & Land, Dredging: A Handbook for Engineers 2nd Edition, 1996).

4.3.5.9 Based on these stated assumptions, there are expected to be up to approximately 304 hopper loads required in total.

Hydrodynamic conditions

4.3.5.10 Water depths across the offshore array are generally between 40 to 55 m below CD with predicted tidal flows reaching a peak of around 0.55 to 0.60 m/s on mean spring tides and 0.27 to 0.30 m/s on mean neaps. The equivalent tidal excursions are around 8 to 8.5 and 4 to 4.3 km, respectively.

4.3.5.11 Flow measurements at site L1, in the southern part of the offshore array, indicate speeds can exceed 0.8 m/s when the tidal range is greater than MSR. The equivalent maximum excursion at this time is determined as 8.8 km.

Overspill considerations

4.3.5.12 The overspill is considered to occur local to the seabed preparation area for each of the 190 foundations.

4.3.5.13 The finer sediment sizes of muds, very fine sands and fine sands are likely to favour being discharged in the overspill rather than the larger particle sizes of medium sand through to gravels which favour being retained in the hopper, noting this hydraulic sorting cannot be considered as 100% efficient.

4.3.5.14 Overspill will form a plume largely made up of the finer sediment which will be advected away by tidal flows. The duration of the overspill event per dredging cycle is likely to be comparable to the time required to fill the hopper. An indicative period of 4 hours is assumed to fill a 11,000 m^3 hopper.

4.3.5.15 The main axis of the plume trajectory will be governed by tidal advection with reduced concentrations around this axis due to dispersion and diffusion mixing processes spreading the plume. Plume concentrations will reduce over distance due to increased mixing and material falling out of suspension. During a neap tide the plume will be advected over a shorter distance than a spring tide, and since the rate of mixing will be less at these times due to weaker flows, then suspended sediment concentrations can be expected to be proportionally higher. On spring tides, the plume will spread further and have a proportionally lower concentration. Winds would expect to have some influence on surface material, either by increasing mixing and/or modifying the plume trajectory.

4.3.5.16 **Table 15** provides a summary of representative sediment types expected to be present in overspill, their individual fall velocities from the sediment plume phase and the time required to reach the seabed, assuming a depth of 50 m below CD.

Table 15: Representative sediment types in overspill across the offshore array.

Sediment type	Size range (mm)	Representative size (mm)	Settling velocity (m/s)	Time to fall out of suspension 40 / 50 / 55 m (minutes)
Medium sand	0.25 to 0.50	0.38	0.050	13 /17 / 19
Fine sand	0.125 to 0.250	0.18	0.017	39 /49 / 54
Silts / muds	< 0.063	< 0.05	0.001	Remains in suspension

4.3.5.17 The sand fractions will settle out of suspension relatively quickly and well-within the distance of a full tidal excursion, and most likely within 1 or 2 km of the discharge position. The short-lived and localised fate of this material will not create any concerns for reduced turbidity.

4.3.5.18 Only the finer material, represented here as silts and muds with a particle size < 0.063 mm, is unlikely to settle out of suspension quickly due to the weak settling velocities generally exceeded by the upward turbulent component of local flows. This material will rapidly disperse across a full tidal excursion leading to reduced concentrations over time and distance from source.

4.3.5.19 As a general consideration, suspended sediment concentrations within sediment plumes can be in the order of hundreds of mg/l in the vicinity of the dredger, reducing to tens of mg/l with distance (CIRIA, 2000), but also quickly dissipating in time after release. Given the likely loading and dumping cycle each overspill event is expected to disperse away as a separate plume.

Hornsea Project Two

4.3.5.20 Hornsea Project Two is the closest project to Hornsea Four with comparable seabed and tidal conditions for sediment dispersion. Seabed preparation was assessed for levelling requirements of GBF (GBS) foundations with up to approximately 23,892 m^3 of material to be removed per foundation using a TSHD (SMart Wind, 2015). For comparison, the equivalent volume per suction bucket foundation for each WTG in Hornsea Four is up to approximately 11,858 m^3 (although, this increases to 131,220 m^3 for each of the three large GBS foundations to support OSS).

4.3.5.21 Sediment plume simulations focused on the overspill with an assumed composition of silts, and fine sands. Coarse sediment (sands and gravels) were not simulated and were assumed to deposit in close proximity to the point of release as spoil. A simulation of eight hopper loads was simulated to cover four foundations and four disposal locations, each with a three hour cycle time. Based on these assumptions, predicted depth-average increases of SSC of > 2 mg/l were predicted around the dredging site with excursions of up to 16 km northwest and 14 km to southeast. At the disposal site the sediment plume showed SSC > 10 mg/l above background over an excursion distance of up to 12 km to the northwest and up to 13.5 km to the south east of each foundation. Peak concentrations of 500 to 800 mg/l were predicted at a site very close to the release of spoil. All peak concentrations were localised and short-lived. When concurrent preparation was simulated for two sites adjacent to each other comparable results were obtained.

4.3.5.22 The deposition of fine sediment (< 0.25 mm) was considered to be localised and in the region of a few millimetres.

Spoil disposal

4.3.5.23 Once the dredger moves to discharge a full hopper load close by, the majority of the finer sediments are expected to have already been lost as overspill. The remaining sediments in the hopper should be predominantly composed of the coarser sediment fraction, meaning that the disposal of the spoil is likely to have a lesser concern in the formation of any sediment plume. In contrast, the majority of the spoil will fall more quickly to the seabed with limited opportunity to disperse leading to a higher risk of smothering of any benthic receptors.

4.3.5.24 Table 16 provides a summary of representative sediment types expected to be present in the spoil, their individual fall velocities and the time required to reach the seabed.

Table 16: Representative sediment types in spoil disposal across offshore array.

Sediment type	Size range (mm)	Representative size (mm)	Settling velocity (m/s)	Time to fall out of suspension 40 / 50 / 55 m (minutes)
Fine Gravel	4 to 8	3	0.216	3 / 4 / 4
Coarse sand	1 to 2	1.50	0.147	5 / 6 / 6
Medium sand	0.25 to 0.50	0.38	0.050	13 / 17 / 18

4.3.5.25 The depth of deposition and area covered will be determined by the course of the vessel in the period of opening hopper doors, the tidal flows at the time and the relative composition of the sediment being disposed of between sands and gravels (which will determine the angle of repose, nominally 25 to 30° for sandy gravel). The vessel speed could also act as means to ensure the deposition of spoil is more widely dispersed than opening the hopper doors when the vessel is stationary.

4.3.5.26 Once deposited the coarse sand and fine gravel are unlikely to be remobilised by the local tidal flows, whereas the medium sands are only likely to be remobilised when flows exceed mean neap tides and for material that is not covered and armoured by the immobile coarser sediment sizes.

Hornsea Project Two

4.3.5.27 For disposal, the placement of a single hopper load of spoil was considered to develop a 120 m diameter and 1.5 m high mound, assuming a stationary release. For all placements required for a single foundation site the area required was assessed as being up to 250 m in diameter, (SMart Wind, 2015), but this scale of deposition was not considered to affect marine processes.

4.4 Seabed installation activities

4.4.1.1 Seabed installation activities planned for the construction phase include provisions for:

- Open cut trenching across the intertidal (landfall area);
- Cable trenching along offshore ECC (for export cables) and through offshore array(for array cables); and
- Drilling for foundation options requiring piles to be inserted into the seabed in the HVAC booster area (3 foundations) and offshore array (190 foundations).

4.4.2 Open cut trenching at the export cable landfall

Overview

4.4.2.1 The option for open cut trenching across the intertidal (an alternative to the HDD option) has the potential to temporarily interrupt longshore drift for a short period (up to 32 weeks), before the beach is re-instated.

4.4.2.2 The potential disturbance corridor from plant movements, excavation, etc. for each cable is an approximate width of 60 m, with a separation of approximately 30 m between cables, a total disturbance width of approximately 210 m. Each trench would be dug to a depth of between 1 to 3 m and take approximately two weeks to complete.

4.4.2.3 The configuration of landfall works across the intertidal, the method of trenching and location of stockpiles for excavated material will determine the types of impacts that might occur.

Sediment types

4.4.2.4 The beach is composed of relatively mobile sands and gravels overlying a more stable glacial clay till of the Bolders Bank formation. The likelihood is that excavation will be required into the more stable till.

Potential impacts

4.4.2.5 The assumption is that these works will take place after any similar landfall works required for Creyke Beck. This assumption removes the opportunity for cumulative impacts of cable trenching between two activities occurring in a similar timescale and close together (Creyke Beck landfall is around 1.5 km to the south of Hornsea Four landfall).

4.4.2.6 Sediment disturbance will occur during the trenching works and any loosened fine material remaining in an open trench will tend to be transported away when the tide washes in and out of the trench. This is likely to introduce a relatively low volume of sediment into the marine environment.

4.4.2.7 If installation works create a barrier effect between high and low water (e.g. due to equipment, vessels, spoil mounds, etc), then there is a potential for longshore drift to be interrupted (e.g. due to excavated volumes being cast aside to create a mound). Given that the landfall works will be completed in a relatively short period, and generally in periods of calmer weather, the potential to (block) interrupt periods of longshore drift during most active winter periods will tend to be mitigated. In addition, longshore drift at the landfall is relatively low, given that this location is essentially a drift-divide with a balance between up and down-drift rates.

4.4.2.8 Once the works are completed, and beach levels are re-instated then a normal beach level is expected to return relatively quickly and most likely within a 6-month period. Corresponding works to lay a new long sea outfall (LSO) south of Bridlington Harbour involved more intrusive works with a temporary cofferdam holding open a larger single trench across the intertidal (**Figure 29**).

Figure 29: Installation of the Bridlington LSO, from UK Water Projects (2015).

4.4.2.9 Shoreline monitoring at this location shows beach levels fully recovered within 6 months (Figure 30).

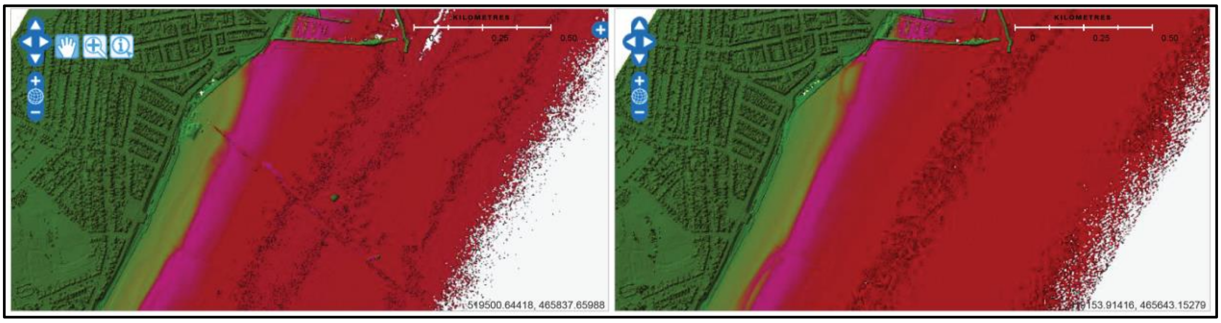

Figure 30: Lidar monitoring at intertidal works area of Bridlington LSO (left – May 2014 during works, right – November 2014 after works).

4.4.3 Cable trenching

<u>Overview</u>

4.4.3.1 Cable trenching will occur after sandwave clearance is completed. Six export cables will be laid along the 109 km offshore ECC, and 600 km of array cables and 90 km of interconnector cables will be laid within the offshore array.

4.4.3.2 Cables will also need to cross other existing pipelines and cables which will introduce cable protection measures. Rock berms up to 1.5 m high are expected to represent the conservative case in terms of introducing the most intrusive option which may change the local profile of the seabed as well as locally increasing seabed roughness with rocky substrate for the mainly sandy and gravelly seabed substrate.

4.4.3.3 The trench depths will be confirmed by a cable burial risk assessment (CBRA). Present assumptions are based on typical trench depths of 1 to 2 m into the levelled seabed. This trench will be V-shaped with a surface width of up to 6 m.

4.4.3.4 The optimal method to achieve trenching generally corresponds to soil strength characteristics and may require jetting and / or ploughing. In addition, consideration is being given to the use of MFE, which is similar to jetting in that both use hydraulic forces to push away mobile sediments. MFE is considered as the conservative installation option because of the greater volume of sediments likely to be disturbed and the type of disturbance which injects material into suspension in comparison to ploughing which will simply cast material to the side.

4.4.3.5 Trenching rates determine how much material is released per second. Trenching rates depend as much on the trenching tool as the soil characteristics, however, some general rates can be suggested:

- 55 m/hour for hard soils;
- 125 m/hour for medium soils; and
- 300 m/hour for soft soils.

4.4.3.6 Up to three cable laying vessels may be operating at the same time creating the potential for these vessels to be operating in a similar area which may compound concentrations of suspended sediment from sediment disturbance effects.

4.4.4 Offshore ECC trenching

Sediment types

4.4.4.1 Sediment types along the offshore ECC are expected to vary in line with the distribution of the regional mapped surficial sediments (Figure 7). The exception here is Smithic Sands were more detailed surveys suggest sands.

4.4.4.2 In some areas where the depth of surficial sediments is less than the trench depth then the composition of material being trenched will vary, however, sediment variability over the depth of trenching remains unknown at this time.

4.4.4.3 Based on this assumption, the main variability in sediment types is summarised in Table 17. Where grab sample evidence coincides with the offshore ECC then the percentage content of muds (irrespective of the sediment type indicated in the generalised mapping) is recorded as a guide to the amount of fine sediment likely to be available to be dispersed away from the trenching process.

Table 17: Expected variation in surficial sediments along offshore ECC.

Location	Distance offshore	Sediment type	% Muds from grab samples
Nearshore	Ebb channel	gravelly muddy sandy	1.7 to 48
Nearshore	Smithic Sands	sand	1.8
Offshore	To 20 km	sandy gravel	0.4 to 2.5
Offshore	To 25 km	muddy sandy gravel	4.2 to 6.4
Offshore	To 41 km	Gravelly sand	0.9
Offshore	To 52 km	Slightly gravelly sand	0.2 to 0.6
Offshore	To 99 km	sand	0.6 to 5.2

4.4.4.4 For the nearshore location in the ebb channel, a high mud content of 48% is suggested to represent the sediment composition of the glacial till, given than the surface layer of sands and gravels in this area is reported to be relatively patchy and thin.

Maximum sediment volumes

4.4.4.5 The maximum sediment volume expected to be displaced by MFE along the export cable route is approximately 3,903,000 m^3. The assumption is this amount of sediment is apportioned between each of the 6 cables which equates to an average sediment volume of 6 m^3 per metre of excavation.

4.4.4.6 The rate of trenching will determine the release rate of sediments into the water column, with higher trenching rates releasing the most amount of sediment per unit time and developing the highest source concentrations. A conservative trenching rate of 300 m/hour is used, irrespective of sediment type. In a one hour period the release would be 1,800 m^3 over a distance of 300 m. If this trenching was through a soil type of "mixed grained sand, dense" then the dry bulk density would be 1,860 kg/m^3 (Bray, Bates, & Land, Dredging: A Handbook for Engineers 2nd Edition, 1996) and the release rate would be 930 kg/s.

4.4.4.7 In general, the majority of the excavated material is expected to be coarse sediments (sands and gravels) which will drop back to the seabed relatively quickly and close to the point of disturbance. The content of fines (fine sands, silts and muds) is generally expect to be low and in line with the variations identified in Table 17. Apart from the trenching required within the ebb channel, where underlying till appears to have a higher % mud content. The percentage of fines (fine sands, silts and muds) which can be dispersed away from the point of disturbance is considered here to be no more than 15% of the total release for the majority of the offshore ECC, which infers a sediment plume would form with a source rate of no more than 140 kg/s of fines.

4.4.4.8 For the main section of the offshore ECC the estimated sequential time required for a single vessel to complete all trenches is 91 days.

4.4.4.9 The exception to this assumption is the ebb channel area where mud content is reported as 48%. The stiffer soils expected here will reduce trenching rates to 125 m/hr, or less, and this equates to a release rate of up to 221 kg/s for fines (assuming 50% content) in this section of the trench.

Hydrodynamic conditions

4.4.4.10 Three locations have been examined for potential plume formation; ebb channel trench, Smithic Bank, area of muddy sandy gravel at 25 km offshore. These sites have been selected on the basis of adjacent receptors; Brislington Harbour, HU015 and Flamborough Head SAC. Sites further offshore are unlikely to lead to any greater impact than that already described for sandwave clearance and levelling.

4.4.4.11 Table 18 provides a summary of anticipated hydrodynamic conditions at the three selected sites. The excursion distance for the ebb channel is offered as indicative only as this site falls outside of available information. Further to this, the operating depth for MFE is likely to mean the site is only accessible during high waters.

Table 18: Representative hydrodynamic conditions for three locations along Offshore ECC.

Location	Water depth (m below CD)	Spring / neap tidal excursion (km)	Annual variability of average wave height (m)	Annual range of SPM (mg/l)
Nearshore – ebb channel	1 to 7 m	9.3 / 4.9	0.77 to 1.25	2 to 14
Smithic Sands	5 to 10	9.3 / 4.9	0.77 to 1.25	2 to 14
Offshore to 25 km, area of muddy sandy gravel	40 to 50	12.7 / 6.4	0.98 to 1.76	1 to 7

Sediment plumes

4.4.4.12 Sediment plumes are likely to form by the advection and dispersion of finer sediment from the point of release. Coarse sediments will fall to the bed relatively quickly.

4.4.4.13 The MFE process uses high volumes of seawater at relatively low pressure to displace sediments away from under the device. This hydraulic force is likely to mobilise the finer sediment fractions relatively high into the water column to form a suspension. In this case, the assumption is made for 10 m above the seabed.

4.4.4.14 Table 19 offers theoretical settling velocities for fine sand, very fine sand and silts and muds along with time for material to fall out of suspension for calm conditions (no waves). When wave influences are included then added stirring effects may prevent all finer sediments from settling out in the shallower nearshore sites and a wider, longer and more dispersed plume is formed.

Table 19: Representative sediment types in overspill plumes formed by MFE trenching at selected sites along offshore ECC.

Sediment type	Size range (mm)	Representative size (mm)	Settling velocity (m/s)	Time to fall out of suspension 1 / 5 / 10 m (minutes)
Fine sand	0.125 to 0.250	0.18	0.017	1 / 5 / 10
Very fine sand	0.063 to 0.125	0.09	0.004	4 / 21 / 42
Silts / muds	< 0.063	< 0.05	0.001	Remains in suspension

4.4.4.15 The main axis of any plume trajectory will be governed by tidal advection at the point of release with reduced concentrations around this axis due to dispersion and diffusion mixing processes spreading the plume. Plume concentrations will reduce over distance due to increased mixing and material falling out of suspension. During a neap tide the plume will be advected over a shorter distance than a spring tide, and since the rate of mixing will be less at these times due to weaker flows, then suspended sediment concentrations can be expected to be proportionally higher. On spring tides, the plume will spread further and have proportionally lower concentrations. Winds would expect to have some influence on surface material, either by increasing mixing or modifying the plume trajectory.

4.4.4.16 During the ebb phase of both a spring and neap tide, there is a theoretical pathway for the sediment plume formed during nearshore trenching activity through the ebb channel (mainly) to reach Bridlington Harbour. On springs tides only this plume could extend to disposal site HU015 and Flamborough Head SAC. The conditions at HU015 and the SAC are highly dispersive for muds and silts, so there is no expectation for material to settle in this location, however, within the harbour, the water conditions are expected to be calm and conducive to settle for any material reaching this location. The harbour already has an existing exposure to siltation from marine sources.

4.4.4.17 Since the ebb channel is around 1 km wide, trenching across this channel at a rate of 125 m/hr would take approximately 4 hours. In this period, the amount of muds brought into suspension could be around 3,000 tonnes. This activity is expected to occur six times but most likely on separate occasions which would mitigate the chance for a larger amount of sediment release and risk of material being deposited within the harbour.

4.4.5 Offshore Array trenching

Overview

4.4.5.1 Trenching for array cables will occur after sandwave clearance and seabed levelling is completed. The composition of the seabed at this time may be different to the present baseline, especially where the depth of surface sands has been cleared away to the underling till to ready for trenching.

4.4.5.2 The 2018 geophysical survey has resolved a depth of Holocene sand which shows that the majority of the offshore array is covered by surficial sediments to a depth < 2 m. Deeper sediment depths are mainly along the western, northern and southern boundaries, but the central area and along the western boundaries only have a thin cover of sediment which in some places is less than 0.5 m (**Figure 31**).

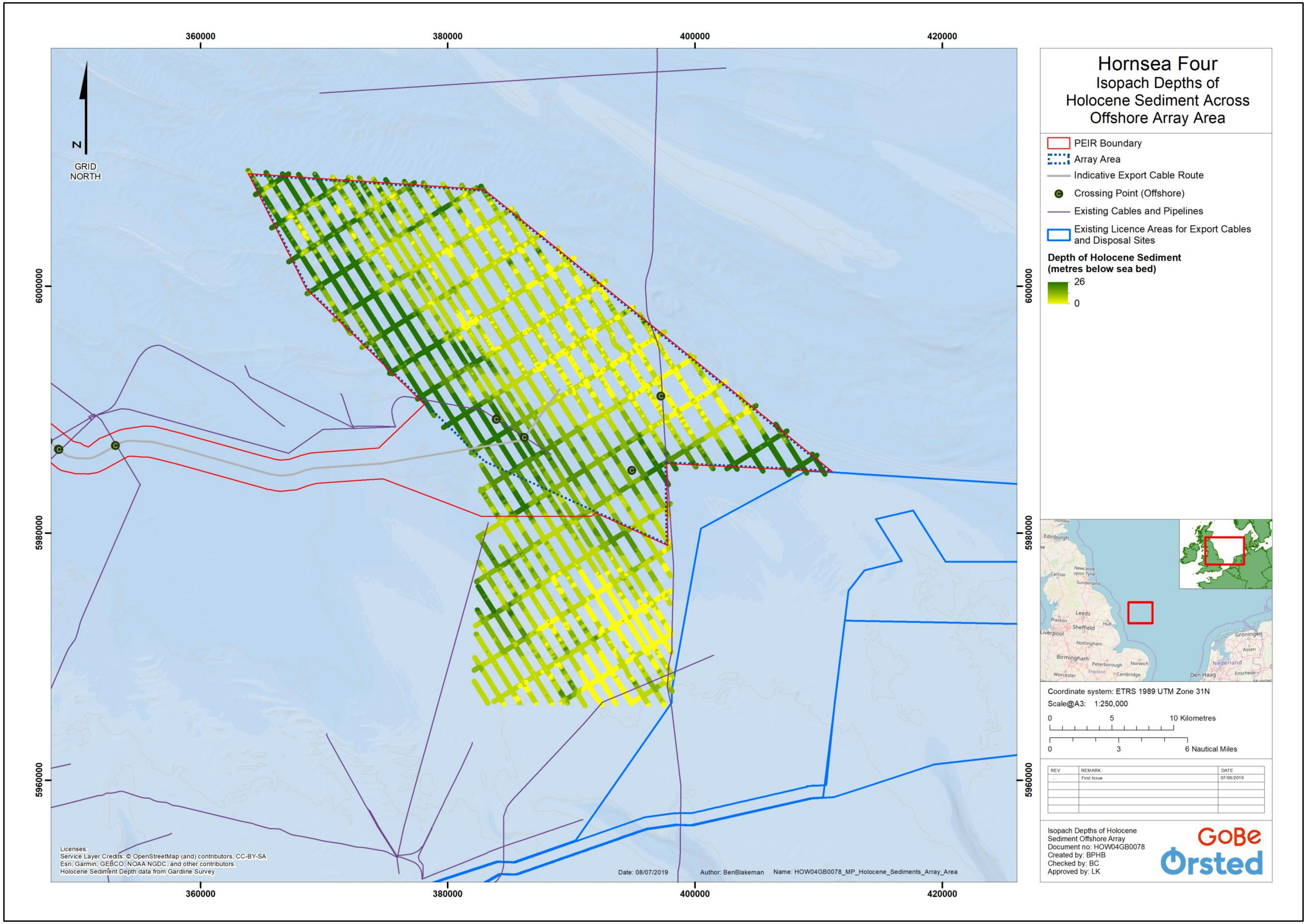

Figure 31: Isopach depths of Holocene sediment across offshore array (not to scale).

Sediment types

4.4.5.3 Surficial sediment types across the offshore array are presented in **Figure 17**. Sands (medium sized) are dominant with some areas including a small gravel content to create a gravelly sand substrate.

4.4.5.4 The composition of sub-surface sediments is unknown at present, although reference has been made to both firm to stiff clay (glacial till) of the Bolders Bank formation and a possible area of Cretaceous chalk in the north of the array (Gardline, 2019).

4.4.5.5 Given trenching depths are up to 2 m and given sandwave clearance may have removed some sand cover prior to trenching, then the expectation is that sub-soils will encountered by the trenching activities which would have a much higher content of fines than the mobile surface sands.

4.4.5.6 There are two grab samples within the array on the eastern boundary which coincide with limited sediment cover (recorded as 0 m Holocene sediment depth) and where the percentage of fines (muds) is 13.7 (ENV19) and 15.3% (ENV17), respectively. These sites are likely to be indicative of the wider area where trenching reaches the underlying soils.

Maximum sediment volumes

4.4.5.7 The maximum sediment volume expected to be displaced by MFE across the offshore array is approximately 4,140,000 m^3. The amount accounts for both array and interconnector cables covering a total distance of 690 km.

4.4.5.8 The rate of trenching will determine the release rate of sediments into the water column, with higher trenching rates releasing the most amount of sediment per unit time and developing the highest source concentrations. A trenching rate of 125 m/hour is assumed to account for likely stiffer soils to be trenched. In a one hour period the release would be 750 m^3 over a distance of 125 m. If this was through a soil type of "glacial till, very mixed grained" then the equivalent dry bulk density would be 2,160 kg/m^3 (Bray, Bates, & Land, Dredging: A Handbook for Engineers 2nd Edition, 1996) and the total sediment release rate would be 450 kg/s.

4.4.5.9 The composition of sub-soils is estimated by consideration of grab samples ENV17 and ENV19. Both samples are described as gravelly muddy sand. Their content of finer sized sediments (fine sand, very fine sand and silts) is generally around 26%. Based on this proportion the release rate of fines which could form a sediment plume would be 117 kg/s.

4.4.5.10 Some moderation on the release of fines is expected where the depth of Holocene sands remains greater than 2m. This is most likely to occur along the northern extent, western boundary and southern extent.

4.4.5.11 For the offshore array, the estimated sequential time required for a single vessel to complete all trenches is 230 days, assuming the dredging rate is not quicker when excavating areas with a thicker layer of sands.

Hydrodynamic conditions

4.4.5.12 Water depths across the offshore array are generally between 40 to 55 m below CD with predicted tidal flows reaching a peak of around 0.55 to 0.60 m/s on mean spring tides and 0.27 to 0.30 m/s on mean neaps. The equivalent tidal excursions are around 8 to 8.5 and 4 to 4.3 km, respectively.

4.4.5.13 Flow measurements at site L1, in the southern part of the offshore array, indicate speeds can exceed 0.8 m/s when the tidal range is greater than MSR. The equivalent maximum excursion at this time is determined as 8.8 km.

Sediment plumes

4.4.5.14 Sediment plumes are likely to form by the advection and dispersion of finer sediment from the point of release. Coarse sediments will fall to the bed relatively quickly.

4.4.5.15 The MFE process uses high volumes of seawater at relatively low pressure to displace sediments away from under the device. This hydraulic force is likely to mobilise the finer sediment fractions relatively high into the water column to form a suspension. In this case, the assumption is made for 10 m above the seabed.

4.4.5.16 Because the plume is made up of sediment particles which are denser than seawater then the plume will behave with negative buoyancy. The longevity of the plume is determined by the particle settling velocity and the potential for maintaining suspension or resuspension of the component sediment types. Table 20 offers theoretical settling velocities for fine sand, very fine sand and silts and muds along with time for material to fall out of suspension.

Table 20: Representative sediment types in overspill plumes formed by MFE trenching across array.

Sediment type	Size range (mm)	Representative size (mm)	Settling velocity (m/s)	Time to fall out of suspension (minutes)
Fine sand	0.125 to 0.250	0.18	0.017	10
Very fine sand	0.063 to 0.125	0.09	0.004	42
Silts / muds	< 0.063	< 0.05	0.001	Remains in suspension

4.4.5.17 The fine sands and very fine sands are able to disperse over a short period only before they fall out of suspension and during this time they are likely to still remain relatively close to the trench. If the release occurred during peak spring flows then the very fine sand could reach a maximum distance of around 1.5 km from the trench and at all other times they would fall closer to the trench.

4.4.5.18 The silts and muds are likely to remain in suspension and form a plume which may still be relatively close to the seabed. As the offshore array is not a deposition environment for silts then this plume is likely to only partially fall out of suspension during slack water / low flow periods and resuspend during peak flows. Over time the material would disperse more widely with this pattern of re-suspension, transport, temporary deposition and re-suspension.

4.4.5.19 The main axis of any plume trajectory will be governed by tidal advection at the point of release with reduced concentrations around this axis due to dispersion and diffusion mixing processes spreading the plume. Plume concentrations will reduce over distance due to increased mixing. During a neap tide the plume will be advected over a shorter distance than a spring tide, and since the rate of mixing will be less at these times due to weaker flows, then suspended sediment concentrations can be expected to be proportionally higher. On spring tides, the plume will spread further and have proportionally lower concentrations.

Hornsea Project Two

4.4.5.20 Plume modelling was performed for Hornsea Project Two assuming jetting into till (at a site along the export cable route in water depths between 25 to 30 m and typically slightly higher flows than Hornsea Four offshore array). Sediment with a 20% content of fines was assumed with a dry density of 1700 kg/m^3. The predictions suggest a typical plume width of 100 m for concentrations above 20 mg/l and 40 m for concentrations above 30 mg/l. These values represent depth-average concentrations in and area between 25 to 30 m depth, rather than near-bed equivalents which would expect to be much higher. Initial deposition occurred during periods of low flow and was around 2 mm thick for locations at around 60 m from the release, and based on a sediment with a settling velocity of 1 mm/s (comparable to the value used in **Table 20**). Permanent deposition was considered to be negligible (SMart Wind, 2015).

Summary

4.4.5.21 Sediment disturbance issues for sandwave clearance, seabed levelling and trenching occur as sequential activities. The range of sediments being disturbed is likely to include coarser and finer sized particles, with only the finer sediments (i.e. very fine sand and silts) able to form sediment plumes which advect away from the source. Coarser grained sands and gravels are the dominant sediment likely to be disturbed which further limits the amount of finer sediments able to form any plume. Some of the finer sediments may remain in suspension due to ambient turbulent mixing in the flows but wider dispersion over time will minimise their concentration. Coarser grained material will fall out of suspension relatively quickly and remain close to source.

4.4.5.22 The main smothering risks occur from spoil disposal from dredging when the coarser sediments are discharged from the hopper and fall quickly to the seabed. Most coarser sediments may be relatively immobile which limits the spoil mounds from quickly dispersing and reducing in profile.

4.4.6 Foundation installation: drilling

Overview

4.4.6.1 Drilling may be required for foundation options which install piles into the seabed and where these piles cannot be installed solely by the use of percussive piling through harder sub-soils or rock. The anticipation is that drilling will only be required for 10% of pile installations (this may be either 10% of sites or 10% of the depth of installation).

4.4.6.2 Drilling produces drill arisings that will be brought back to the drilling vessel prior to surface discharge into the sea. Up to two drilling rigs may be operating at the same time. If this occurred at adjacent sites along a tidal excursion, then there is the potential for sediment plumes to disperse together and lead to higher overall increases in SSC.

4.4.6.3 The composition and particle size of drill arisings is unknown at present and depends on many variables, not least; local rock type(s), size of drill, drill speed, drill pressure, etc. The typical conservative assumption is to treat 100% of material as fines, although existing evidence of drill cutting piles suggests this is unlikely and in some cases semi-permanent cuttings piles have formed of relatively large casts, for example at North Hoyle (DECC, 2008).

4.4.7 Drilling at HVAC booster area

4.4.7.1 One of the foundation options in the HVAC booster area is the six-legged Piled Jacket (Medium OSS) with 4 m pin piles with an embedment depth of up to 100 m. Provisions for drilling these piles assumes a potential for up to 4,618 m^3 of drill arisings, if required. This potential volume of sediment release is comparable to seabed levelling and the potential release of fines from the same location in overspill (see **Section 4.3.4**) which has a higher estimated total volume of up to 8,578 m^3.

4.4.7.2 The apportionment of the drill arising to a specific location or the time to complete drilling remains uncertain at this time. For comparison, Hornsea Three assumed a production rate of 88 m^3/hr and Hornsea Project One and Two a rate of 235 m^3/hr for 10 and 15 m diameter monopiles. Using these values as an indicative range would equate to a total drilling period of between approximately 20 to 53 hours for Hornsea Four to produce up to 4,618 m^3 of drill arisings. This estimate ignores repositioning of the drill rig.

4.4.7.3 Applying the presently available details, and assumed drilling rates, would suggest comparable sediment plumes and deposition effects to those previously discussed in **Section 4.3.4**, and potentially less in proportion due to the smaller release volume.

4.4.7.4 Given the MDS option for seabed preparation is based on a Suction Caisson Jacket (Medium OSS), this implies a lesser requirement of seabed preparation for all other foundation options in the HVAC Booster Area, including Piled Jacket (Medium OSS). Any lasting deposition effects from the seabed preparation phase would therefore be lesser in this case.

4.4.8 Drilling within offshore array area

4.4.8.1 The MDS considerations for drilling within the offshore array are based on the information presented in **Table 21**.

Table 21: Summary of drill arisings for foundations across the offshore array.

Unit	Foundation type	Number	Drill arising volume (m^3)	Equivalent volume per foundation (m^3)
WTG	Monopile	180	127,235	707 * 180 7070 * 18
OSS	Piled jacket (Medium OSS)	9	13,854	1,540
Offshore Accommodation	Piled jacket (Medium OSS	1	1,540	1,504
	Total	**190**	**142,629**	

4.4.8.2 For illustrative purposes 10% of monopile sites equates to a volume of approximately 7070 m^3 and 10% of all sites a volume of approximately 707 m^3. The allocation of the maximum volume of drill arisings for monopiles in the offshore array could also be somewhere in this range.

4.4.8.3 In comparative terms, these quantities of drill arisings are lower than the overall volume requirements for seabed levelling at the same locations (see **Section 4.3.5**). For reference, the assessment of seabed levelling assumed 5% of the total volume is represented as fines.

4.4.8.4 The apportionment of the drill arising to a specific foundation location or number of locations or the time to complete drilling means that neither the location nor rate of drill arisings can be certain at this time. For comparison, Hornsea Three assumed a production rate of 88 m^3/hr and Hornsea Project One and Hornsea Project Two a rate of 235 m^3/hr for 10 and 15 m diameter monopiles. Using these values as an indicative range would equate to a total drilling period of between approximately 607 to 1621 hours for Hornsea Four to produce up to 142,629 m^3 of drill arisings. This estimate ignores repositioning of the drill rig, etc.

4.4.8.5 Based on presently available details, and assumed drilling rates, would suggest comparable sediment plumes and deposition effects to those previously discussed in **Section 4.3.5**, and potentially less in proportion due to the smaller release volume.

Hornsea Project One

4.4.8.6 Provisions for drilling in the offshore array were part of the Hornsea Project One (and Hornsea Project Two) application; however, when the installation of foundations at Hornsea Project One was completed in 2018 and 2019 no drilling was required.

4.4.8.7 Hornsea Project Two modelled the drill arising for monopile foundations with a volume of drill arisings of 3,927 m^3 and a jacket with a volume of 8,482 m^3. The contribution of fines was based on borehole information with a fine content of between 28 to 43%.

4.4.8.8 Sediment plumes only formed from the release of fines with sand sized sediments falling rapidly out of suspension and local to the point of release. Fine sediment did not form any appreciable deposition but dispersed at the scale of the local tidal exclusion with depth-average concentrations of > 10 mg/l extending 2.5 to 8 km from the point of release. Any peaks in raised levels of sediment concentration were short-lived (SMart Wind, 2015).

4.5 Scouring around foundations

Overview

4.5.1.1 Scouring is a near-field process when flows are locally blocked and need to accelerate past an object. The intensified flow speeds passing around the object create vortices (turbulence) that increase bed shear stress acting on the seabed which can then lead to local scouring when the sediment is susceptible to these higher erosion forces. The scouring continues to an equilibrium condition which eventually accommodates and dissipates the faster flows and vortices. This situation is generally described as the equilibrium scour depth. Scouring can be mitigated by placing scour protection around an object to armour the seabed against the heightened erosion forces.

4.5.1.2 The rate of scouring to the equilibrium depth can be slow when there is limited active sediment transport because flows are too weak or no erosion because the soils are resistant. When an object with a surface profile is introduced and blocks incident flows, the accelerated flows have the potential to create "clear water scour".

4.5.1.3 The rate of scouring can be fast when the seabed is already highly mobile, this is generally referred to as a "live-bed" regime. Where wave forces act on the seabed then the rate of scouring can increase.

4.5.1.4 When exposed to tidal flows the extent and depth of scouring is mainly related to the scale of the structure as well as the shape of the structure and the soil properties (e.g. angle of repose). For slender monopiles (i.e. when the ratio of pile diameter, D to water depth, h is < 0.5), the scour depth is a function of the pile diameter, D, and a near-circular form of scour is created, although this can also be asymmetric in shape depending on the way ebb and flood flows affect the structure. This is referred to as "local scour".

4.5.1.5 When piles are closely spaced (e.g. multi-legged jacket structures) then the extents of local scouring around each pile can overlap and create larger "group scour".

4.5.1.6 Large (and non-cylindrical) foundations generally exceed the criteria of slender piles and flow separation occurs around the structure. As a consequence, the shape of scouring may be spread differently around the base of the structure, rather than uniformly. In addition, the alignment and shape of the structure with incident flows will also determine where scouring occurs. For example, a larger rectangular structure facing incident flows may have greatest scour at the corners of the base.

4.5.1.7 General changes in seabed levels, separate to any influence of structures, can also occur which also present a risk to foundations, this is commonly referred to as "global scour".

4.5.1.8 The existing design option may place scour protection on the seabed prior to foundation installation. In this case scouring is mitigated. The other option is to install the foundations first and then add scour protection. In this case, the period between foundation installation and placement of scour protection leaves the structure prone to scouring. The amount of scour that may take place in this period depends on many factors, including; the local sediment types, flow environment and structure shape.

4.5.1.9 The consequence of scouring is normally limited to the near-field of individual structures and is likely to be limited in extent to scales of tens of metres. The time to achieve the equilibrium scour depth is also relatively quick and likely to be within a few tidal cycles for a live bed condition. Any eroded material from the scouring process will also become quickly assimilated as part of the wider sediment transport regime. In some live-bed situations, scour "tails" have been observed over several hundred metres downstream of monopile foundations in shallow water which are considered to be a product of turbulent forces in the wake continuing to affect the seabed (DECC, 2008).

4.5.1.10 The environmental impact of scouring is minimal when the local scale of change is largely limited to each foundation. The separation distance between foundations is also typically sufficient to remove any risk of group scour which may lead to the risk of destabilising a large morphological feature, such as a sandbank.

4.5.1.11 The main environmental change is likely to be related to the introduction of rock armour as scour protection around the periphery of the structure, e.g. situations where rock armour changes a sandy substrate into a much coarser substrate. Apart from any ecological relevance, this change would also locally modify the roughness of the seabed.

4.5.2 Scour around cofferdams: Landfall Area

Structures

4.5.2.1 Up to eight cofferdams are proposed in the nearshore area to accommodate HDD exit pits. Although their precise location is unknown at this time, the likelihood is they will be dug into the glacial till.

4.5.2.2 Each cofferdam is assumed to be square in shape with a length and width of 10.6 m, although the effective width facing incident waves or flows depends on the orientation of the structure. If the structure were at 45° to incident flows or waves, then the effective width becomes 14.1 m.

4.5.2.3 The minimum spacing between cofferdams is potentially 10 m which means they have the potential to act together and create group scour, although the sub-surface sediments of stiffer glacial till may be more resistant than any surface layers of sand. Collectively, they cover an immediate area of approximately 900 m^2.

4.5.2.4 The precise location of the cofferdams is unknown at this time, but the likelihood is they will be dug into the glacial till, a sediment layer which is likely to be slow to scour.

Evidence base

4.5.2.5 The evidence for slow scour rates in glacial till is demonstrated from the local beach and also from an existing offshore windfarm installation.

4.5.2.6 A series of tank traps remain on Fraisthorpe Sands which show evidence of shallow scour pools in the sandy beach. The depth of scour is likely to be limited by the underlying glacial till. These structures have been in place since WWII, around 80 years.

Figure 32: Example of scour around rank traps (IECS, 2019).

4.5.2.7 Scour observations are also available from the Barrow Offshore Wind Farm which reported near zero scour depths on the glacial till bed material and with some marginal scouring which developed slower than foundations in sand (DECC, 2008).

4.5.3 Foundation scour: HVAC Booster Area

Structures

4.5.3.1 The MDS option for the HVAC booster area is based on three large 75 m wide GBS (Box-type) foundations in an area of 24 km^2 located around 34 to 42 km offshore and within the offshore ECC. The location of each foundation is yet to be determined and their orientation with respect to incident flows also remains unknown. If flows are at 45° to the structure, the effective width increases to 106 m.

4.5.3.2 The base of each foundation will occupy an area of approximately 5,625 m^2 with provisions for scour protection adding an additional 25,000 m^2.

4.5.3.3 The minimum separation between each foundation is 100 m.

Sediment types

4.5.3.4 The surficial sediment type is mainly gravelly sand becoming slightly gravelly sand to the eastern part of the area. The depth of surface sediments is unknown at this time. The Creyke Beck export cable route geophysical survey also crossed a similar area to the north and describes the seabed as a veneer of gravelly sand with occasional megaripples, cobbles and boulders and sub-cropping bedrock and till (ForeWind, 2013).

Local hydrodynamic conditions

4.5.3.5 The water depths at the HVAC booster area are generally around 50 m below CD with predicted tidal flows reaching a peak of around 0.84 m/s on mean spring tides and 0.42 m/s on mean neaps. These magnitudes of peak flows are only sufficient to mobilise medium sands during spring tides. There are also no reported bedforms in this area. The likelihood is the existing conditions are not typical of a live bed regime.

4.5.3.6 Water depths are sufficient to mitigate any wave affects acting on the seabed.

Scour assessment

4.5.3.7 Given the ratio between water depth and size of structure, the scour case exceeds slender pile assumptions. If the seabed is susceptible to increased flow speeds at these locations, then scouring will most likely be greatest at the edges of the structure and related to the orientation to incident flows.

4.5.3.8 If the foundations are close together, at the minimum separation of 100 m, then flow interactions between structures are likely and more complex scouring might occur between structures.

4.5.3.9 The amount of material that may be scoured from around the base is likely to be lower than the quantities considered for seabed levelling at the same location.

4.5.3.10 Material that is susceptible to being scoured is likely to be limited to the sand fraction with the gravel fraction more likely to remain *in situ* and helping to armour the seabed. The sand fraction only becomes mobile during peak flows on spring tides.

4.5.3.11 Deeper scour could be limited by the underlying sediment layers. The depth of these layers and type of sediments remain unknown at this time. The further campaign for geophysical survey data will provide additional information on sediment composition and thickness.

Evidence base

4.5.3.12 Scour observations around large gravity bases are relatively limited in comparison to more common monopiles and bridge piers.

4.5.3.13 The F3 Offshore GBS Platform is part of oil and gas infrastructure installed in the Dutch part of the southern North Sea in 1992. The dimensions of the GBS are 70 by 80 m width and 16 m high, with three caissons on the top. Around the base of the foundation is a 1 m gravel filter layer with gabion mattresses placed on top which extend the width by 6 m on all sides. The gabions are fixed to the GBS to prevent scouring undermining the structure.

4.5.3.14 The platform is in a water depth of 42.3 m on a seabed of mainly fine sands (D_{50} = 0.15 mm and D_{90} = 0.21 mm). The spring tide peak flow at this location is reported as 0.41 m/s which would suggest no sediment transport unless storm events and surge currents exceeded this magnitude. The one-year return period storm wave is estimated to be a wave height of 4.9 m and a period of 9.4 s, which theoretically will also initiate sediment transport.

4.5.3.15 Based on seabed inspections of the structure, a scour hole is noted in the south-west corner with a maximum depth of 3 m (**Figure 33**). Without gabions protecting this corner, the scour would have expected to undermine the GBS in this location. A further and smaller scour hole was also reported in the south-east corner.

4.5.3.16 The locations of the scour holes are considered to be due to the tidal currents which flow in the east west directions (Bos, Chan, Verheij, Onderwater, & Visser, 2002).

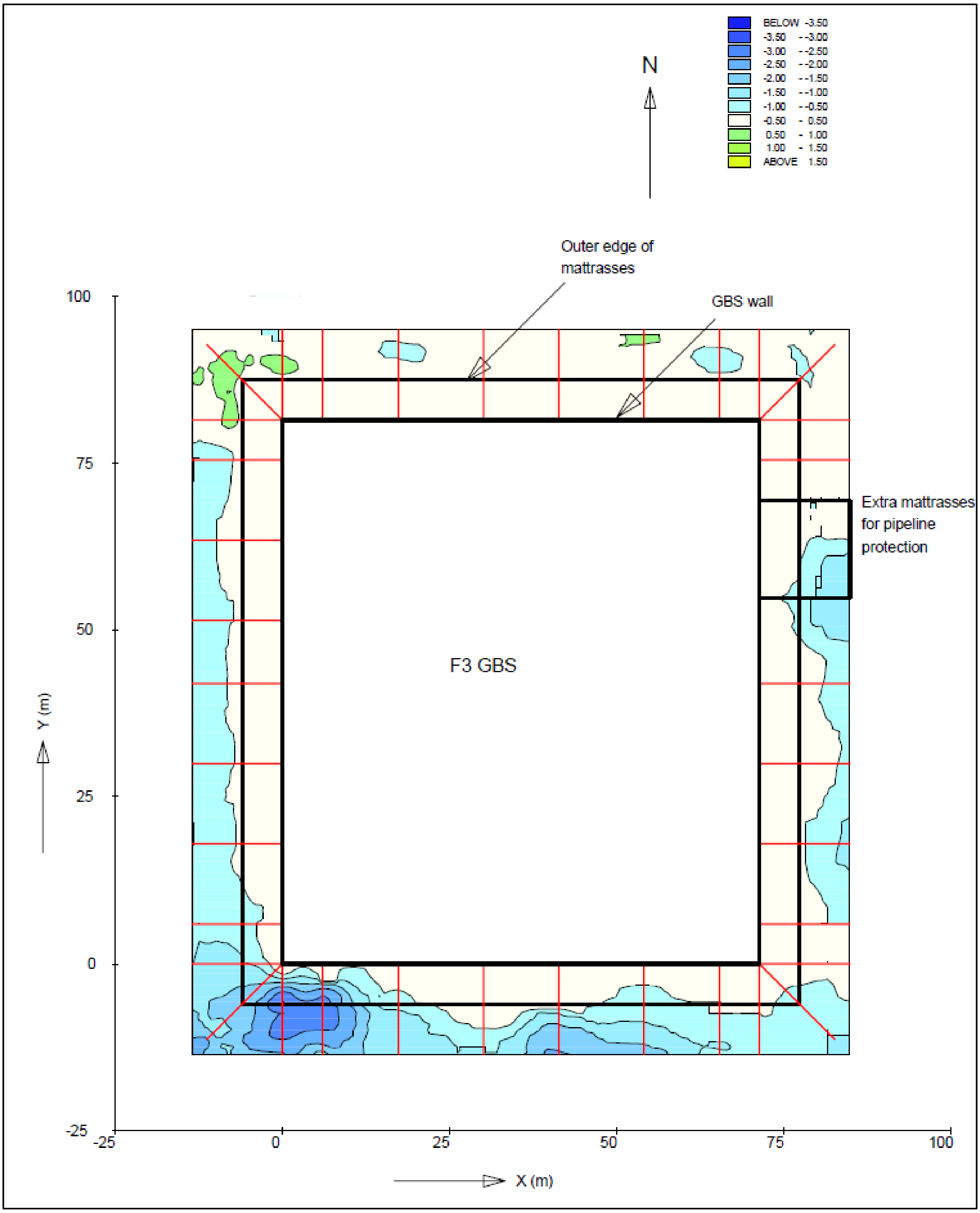

Figure 33: Observed scour around F3 Offshore GBS, after 6 years (Bos, Chan, Verheij, Onderwater, & Visser, 2002).

4.5.3.17 The observations around F3 GBS tally with expected scour behaviour and provide a suitable insight for the expected scour around an individual GBS box-type foundations at the HVAC booster area.

4.5.4 Foundation scour: Offshore Array

Structures

4.5.4.1 The MDS foundation options for the offshore array are based on the structures which are considered to exert the greatest amount of blockage to incident flows and therefore create the largest amounts of turbulence which has the potential to induce most scouring of the local seabed. Relative scales of blockage for each foundation type have been assessed using indicative solidity ratios applicable across the area of the foundation presented to incident flows. For example, a solid structure will have a solidity ratio of 1 whereas an open lattice jacket will have a solidity ratio of around 0.3.

4.5.4.2 **Table 22** summaries the MDS foundation options for blockage for the offshore array.

Table 22: Summary of MDS foundation options for blockage.

Unit	Foundation type	Number	Base Width (m)
WTG	Suction Bucket Jacket	180	3 * 20
OSS large	GBS (Large OSS)	3	150
OSS small	GBS (Medium OSS)	6	75
Offshore Accommodation	GBS (Medium OSS)	1	75

4.5.4.3 The WTG suction bucket jacket is a 3-legged lattice tower mounted on a set of 3 suction bucket caissons which are each 20 m in diameter with a profile above the seabed of up to 5 m. The effective width at the seabed is 65 m, narrowing to 45 m at the base of the lattice structure which further reduces to 25 m at the sea surface. There is a potential for local scouring around the base of each bucket as well group scouring around all buckets.

4.5.4.4 The effective base width for 75 and 150 m box-type BGS increases when the incident flow is at 45°, this leads to effective widths of 106 and 212 m, respectively. Scour protection is planned around the periphery of these foundations over a distance of 50 m.

4.5.4.5 An indicative layout for the 190 foundations is presented in **Figure 34**. There is no allocation of unit and foundation type at this stage. Spacings between foundations also remain indicative at this time, although a minimum distance of 810 m will be maintained between centres of all WTG.

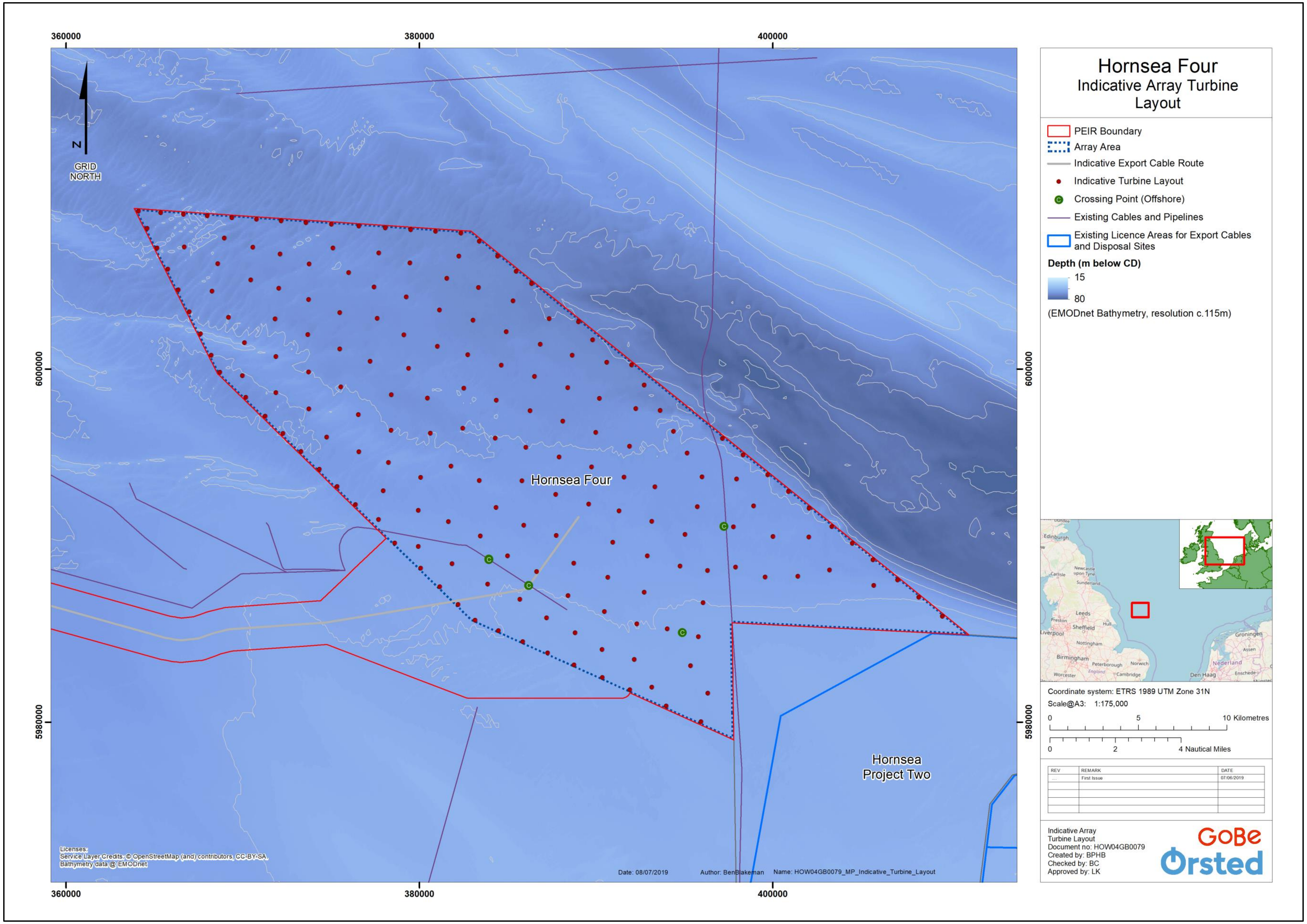

Figure 34: Indicative layout for 190 foundations across the offshore array (not to scale).

Sediment types

4.5.4.6 The surficial sediment types across the offshore array are mainly sands with small patches with some gravel or fines. This sediment cover is generally less than 2 m thick with a layer of firm to stiff clay of the Bolders Bank formation beneath (Gardline, 2019).

4.5.4.7 Sandwave crests are evident across much of the offshore array, apart from the southern extents. These crests are generally aligned perpendicular to the axis of tidal flows and indicate sediment mobility, suggesting a live-bed regime for sands.

Local hydrodynamic conditions

4.5.4.8 The most common sediment fraction is medium sands (particle size in the range 0.25 to 0.50 mm) (Gardline, 2019). This sediment size requires flows in excess of 0.5 to 0.6 m/s to become mobilised, based on standard theoretical expressions (Soulsby, 1997). Tidal mapping from the Atlas of UK Marine Renewable Energy suggests this magnitude is generally limited to peak flows during spring tides and is not attained during neap tides.

4.5.4.9 Water depths are sufficient to limit any wave affects acting on the seabed which could lead to sediment transport.

Scour assessment

4.5.4.10 All MDS foundation structures exceed the case of slender piles. For these situations, flow separation is likely to occur around the base of all foundations leading to edge scour.

4.5.4.11 The likely extent of scour is taken to be less than the planned extent for scour protection, which in the case of the box-type gravity bases is 50 m from the edge of the structure. All foundations are considered to be sufficiently separated to mitigate the chance of group scour.

4.5.4.12 The amount of material that may be scoured from any foundation base is likely to be lower than the quantities considered for seabed levelling at the same location. Once any scouring has removed the surface layer of sands, deeper scour is likely to be moderated by the underlying till which is expected to have a much slower rate of scouring.

4.5.4.13 The surface sands that become susceptible to being scoured will quickly assimilate into the wider transport regime.

Evidence base

4.5.4.14 For the larger GBS box-type foundations a similar scour pattern is likely to that described in **Section 4.5.3** for the HVAC booster area box-type GBS.

4.5.4.15 Hornsea Project Two considered scour development around a 40 m diameter GBS. The maximum predicted scour under currents was between 0.4 to 1.9 m. Based on the angle of response for stable sediment slopes, the assumed extent of the scour hole was estimated between 38 to 101 m (SMart Wind, 2015). These details provide a proxy for equivalent scour depths and extents around the WTG suction bucket jacket foundations being considered for Hornsea Four.

4.5.4.16 Hornsea Project Two also considered the potential resistance of stiff clay till using an Erodibility Index approach which indicated the underlying till would resist scour development.

Summary

4.5.4.17 In the case that scour protection is laid after installation of foundations then some scouring around all foundations is likely to occur.

4.5.4.18 The extent of scouring may be limited if scour protection was laid directly after installation.

4.5.4.19 The depth of scouring is likely to be limited by underlying clay till in many areas of the offshore array when the depth of the Holocene sands is relatively thin (< 1 m).

4.5.4.20 The maximum amount of scour protection is very small in comparison to the whole offshore array area and is estimated to be < 0.18%.

4.6 Cable Crossings

Overview

4.6.1.1 Rock armour is the MDS option for cable crossings as well any requirement for cable protection / reburial. Cable crossings are identified over existing assets as well as proposed assets in both the offshore ECC. Reburial requirements remain as a provision and the sites are unspecified although a CBRA would expect to identify vulnerable sites based on sediment mobility, although reburial requirements may also arise from other causes or events such as anchor drags.

4.6.1.2 The Project Description for Hornsea Four (**Volume 1 Chapter 4: Project Description**) provides a generic example for the creation of rock berms. This rock grading generally has mean rock size in the range of 90 to 125 mm and maximum rock size up to 250 mm, although larger rocks may be necessary if protection from larger anchors is required.

4.6.1.3 The existing cable or pipeline will first be covered with a pre-lay rock berm of a typical length of around 55 m in length and 12.4 m in width and to a depth of around 0.3 m. The cable will be laid at right-angles over this material and then covered with a post-lay rock berm which is notionally 500 m in length and 10.4 m in width. The final profile of the rock berm will be a trapezium shape, up to approximately 1.5 m above the seabed with a 3:1 gradient. The actual cross-section in any specific case may vary and be dependent on expected scour as well as cases where the existing pipeline is surface laid.

4.6.1.4 The potential environmental concerns for placing rock on the seabed are related to the change of substrate type as well as the effects the height, length and orientation may have on interrupting sediment pathways, notably bedload.

4.6.1.5 Hornsea Project Two considered that these relatively low, narrow profiles were not sufficient to influence wave transformation significantly in deeper water (> 12 m). In addition, the likely extent of the cable protection measures does not constitute a continuous blockage along the cable route corridor (SMart Wind, 2015).

4.6.2 Cable crossings: Offshore ECC

Locations

4.6.2.1 Paragraph 3.3.3.17 notes the nine existing pipelines along the offshore ECC which require cable crossings. These sites are all distant from the coastline and in relatively deep water (> 40 m depth) and would not expect to interfere with wave energy transformation. Table 23 summaries the expected environmental conditions at each location. Most pipelines are surface laid and are not trenched for burial so that they remain accessible for remedial repairs, etc.

Table 23: Summary of environmental conditions at identified pipelines requiring cable crossings.

Crossing	Type	Depth below CD (m)	Substrate	Orientation to flows
Langeled	Pipeline	51	gravelly sand	Traverse
Ravenspurn to Dimlington	Pipeline	44.5	sand	Parallel
Minerva to Cleeton Piggy (3 crossings within 40 m)	Pipeline	47	sand	Parallel
Neptune to Cleeton	Pipeline	47.5	sand	Traverse

4.6.2.2 The Minerva to Cleeton Piggy crossing is notable where there are three crossings identified within 40 m of each other. The expected alignment of the export cables at this location would imply that passing flows would run parallel to any cable crossings at this location.

Nearshore

4.6.2.3 In addition, paragraph 3.3.3.18 identifies the need for cable crossings with the export cable from the Creyke Beck offshore windfarm at a planned location seaward of Smithic Sands. The expected environmental conditions at the crossing are:

- Water depth < 20 m below CD;
- Berm alignment is assumed to be mainly flow parallel;
- Traverse to refracted waves from offshore; and

- Sandy gravel seabed.

4.6.2.4 Given that there are expected to be up to four export cables for the Creyke Beck offshore windfarm and six for Hornsea Four, this equates to 24 crossings, however, these are likely to be achieved in a matrix arrangement rather than as separate crossings. The final arrangement of these crossings depends on many issues, not least the separation between each of the export cables. The full crossing arrangement is expected to cover an area of around 4 km^2 within which there will be several rock berms up to 1.5 m high. In relative terms this could reduce local depths by around 7.5% with a potential to locally modify both waves and flows.

4.6.2.5 Existing sediment pathways suggest sands are moved to the south-west onto Smithic Sands from Flamborough Head (**Figure 15**) as bedload transport and a series of rock berms may partly interfere with this process, along with possible scouring where there are accelerated flows around the ends of the berms. The existing tidally driven sand pathways may be deflected by the berm and sediment supply to the bank for this section becomes interrupted. Storm waves may also dissipate some energy on the berm ahead of shoaling onto Smithic Sands. Over a period of time the rock berms may become buried by the build-up of sands enabling sediment pathways to Smithic Sands to re-establish.

4.6.2.6 Smithic Sands also represents a nearshore morphological feature which is in dynamic equilibrium with the existing baseline. This dynamism is a function of sediment supply and tidal circulations developing and sustaining the profile of the bank against higher energy storm events which may lead to temporary redistribution of sands. Consequently, burial depths across Smithic Sands (to be established as part of the CBRA) need to account the risk of seabed mobility which may lead to variations in bank levels. The nearshore section of the export cable across Smithic Sands is therefore considered to be a potential area where additional cable protection measures may be required during the operational period.

4.6.3 Cable crossings: Offshore Array

4.6.3.1 Provisions for cable crossings are also required within the offshore array, these provisions may need to account for two new pipelines which would increase the potential number of cable crossings to 40 separate locations. The likely area required to accommodate rock berms for all these crossings is estimated to be up to 255,000 m^2, or up to 6,375 m^2 per crossing. Whilst this is a relatively large area, in proportion to the total offshore array area this represents a little more than 0.04%.

4.6.3.2 The implication of 40 crossings in the offshore array to marine processes is likely to be relatively minor since the area already has mobile sandwaves of larger amplitude as well as existing surface laid pipelines.

4.6.3.3 If the vertical profile a berm was 1.5 m above bed level, then the relative reduction in water depth for the shallowest (37.5 m below CD at Site L1) locations across the southern part of the array would be 4%. For the deepest sites, around 55 m below CD, this reduction becomes 2.7%.

4.6.3.4 The main environmental issue is likely to be the introduction of a coarser substrate (rock armour) onto a mainly sandy environment.

Hornsea Project Two

4.6.3.5 Hornsea Project Two considered that the relatively low, narrow profiles of rock berms would not be sufficient to influence wave transformation significantly in deeper water (> 12 m).

4.7 Turbulent wakes

Overview

4.7.1.1 Turbulent wakes (rather than wakes that increase turbidity) are an extension of the near-field scour related blockage affects. Wakes occur on the leeward side of a foundation and are generally represented in models as a reduction in the time-averaged flow speed. At the same time, the intensity of turbulence within the wake increases which can also lead to faster rates of dispersion and mixing. The extent of a flow reduced wake can be considered as a proxy for the area which is also affected by more intense turbulence.

4.7.1.2 Turbulent wakes propagate away from a structure and have the potential to influence the far-field with higher levels of turbulence. The main consideration for turbulent wakes is in regard to potential disruption to the Flamborough Front.

4.7.2 Turbulent wakes at landfall area

4.7.2.1 Flow and wave related wakes will form locally around the eight cofferdams used to protect offshore HDD exit pits. Since these are relatively small temporary structures the effect is likely to be negligible over the period and reversible when removed.

4.7.3 Turbulent wakes at HVAC booster area

4.7.3.1 Flow and wave related wakes will form locally around the three 75 m wide box-type gravity bases. Wave related effects are discussed in **Section 4.8**.

4.7.3.2 Due to the scale of this foundations, incident flows will be decelerated onto the face of the structure and then become separated around the structure, most likely to create localised faster flows and separate vortices around edges. In the near-field, the flow related wakes will be responsible for scour development around the corners of the structure. The expectation is the turbulent flow wakes would quickly dissipate and decay in intensity thereafter along the axis of the tidal ellipse (north-east on the ebb and to south-west on the flood) with no further influences on the seabed. Ambient flows will also contain some turbulence, and this may help the rate of dissipation of foundation related turbulence.

4.7.3.3 The precise form of these wakes remains dependent on the relative orientation of each foundation to incident flows and their relative spacing, noting that a minimum spacing of

100 m is specified.

Hornsea Project Two

4.7.3.4 Hornsea Project Two considered the likely flow wakes from two GBF foundations required for HVAC reactive compensation substations along the export cable corridor. The foundations were up to 50 m wide at the base. Given the depth of water (23 m below MSL), the principal changes in tidal flows were considered to remain localised to the structures, although it was predicted that wake effects could extend several km to the north and south of the structure under peak tidal flows (1.3 m/s). The greatest changes were considered to be local to the foundation where the largest flow accelerations were expected to occur.

4.7.4 Turbulent wakes at offshore array

4.7.4.1 Flow and wave related wakes will form locally around the 190 foundations in the offshore array. Wave related effects are discussed in **Section 4.8**.

4.7.4.2 There are three types of foundations in the offshore array which will develop different scales of wakes in proportion to their size and shape (and orientation to incident flows with respect to box-type GBS):

- 180 WTG suction bucket jackets with an effective seabed width of 65 m, narrowing to 25 m at the sea surface (with an assumed solidity ratio of 0.3);
- 3 large GBS box-type with 150 m width base; and
- 7 medium GBS box-type with 75 m width base.

4.7.4.3 The distribution of these foundation types across the indicative layout is unknown at this time, neither is the orientation nor spacing between any of the box-type GBS foundations.

4.7.4.4 A layout comprising of only suction bucket jackets would expect to lead to individual wakes around each structure that would only interact if the ebb and flood wake alignments reached an adjacent foundation, however, separations between adjacent foundations are likely to be sufficient to limit this interaction, especially if their alignment avoids the tidal axis.

4.7.4.5 The inclusion in the offshore array of large box-type foundations with greater widths and non-cylindrical shapes increases the potential for wake to wake interactions across parts of the array which are in the leeward path of these larger foundations. Wakes from these structures are likely to form initially with flow separations broadening the overall wake widths.

4.7.4.6 Based on detailed temperature modelling, and times when there is development of thermal stratification in the northern North Sea from spring to summer, Hornsea Four has been assessed to be within the area of stratification and around 5 km to the north of the divide (at the closest point) with the area to the south remaining well-mixed (**Figure 26**). The (seasonal) divide is regarded as the location of the Flamborough Front, which is the area of main biological interest. Wakes from the very southern extent of Hornsea Four could reach the front on the flood tide and during periods of spring tides, but any affect is both very spatially limited and time limited.

4.7.4.7 Increased seasonal mixing from Autumn to Winter, due to stronger winds, increases wave stirring effects as well as surge related currents which act together to de-stabilises the stratification and the front dissipates at these times.

4.7.4.8 Wakes will add turbulent mixing into the water column which has the potential to locally inhibit thermal stratification from spring and summer and quicken the destabilisation process during the autumn and winter period. However, the wakes are relatively small-scale features that vary in position between ebb and flood tides and magnitude during the spring and neap periods so that their influence is not constant.

Wake monitoring

4.7.4.9 The outcome from a review of current and wake monitoring at Barrow, Burbo Bank and Lynn & Innner Dowsing (all based on slender pile monopile foundations) demonstrated that the turbulent wake around a single foundation is directly influenced by the width of the structure and the incident current speed. In addition, provided foundations are located at a sufficient distance from one another, cumulative array-scale effects of flow separation and wake changes will not be an issue (MMO, 2014).

Hornsea Project One

4.7.4.10 Blockage effects on tidal flows has been modelled for Hornsea Project One as modifications to the time average flows, turbulence is not represented in the model; however the scale of wakes represented as a reduction in mean flows provides a proxy for the area within which turbulence effects can be considered to be greatest. Hornsea Project Two and Hornsea Project Three both refer to the same evidence without modelling their respective layouts or alternative foundation sizes.

4.7.4.11 Hornsea Project One modelled the densest layout (Layout 1) comprising 332 WTG foundations, plus five HVAC collector substations, two offshore High Voltage Direct Current (HVDC) converter stations, two accommodation platforms and one offshore HVAC reactive compensation station (a total of 341 structures). All sites were represented with the same 50 m diameter GBF. The separation between foundations was 924 m.

4.7.4.12 Flows reduced slightly along a line of foundations which were also aligned with the tidal axis, indicating wake to wake interactions. Flows increased slightly between rows. All changes in flows were shown to be less than 0.05 m/s.

4.7.4.13 Wakes generally remained within the boundary of the wind farm but some effects were still evident just beyond the array. The ebbing tide showed wakes extending from Hornsea Project One into Hornsea Project Two. The single HVDC station to the south of the array provides an indication of the scale of a wake from a box-type foundation which appears to be around 4 km on the ebb tide to the detectable limit of 0.01 m/s flow reduction.

4.7.4.14 For reference, Hornsea Project One is now built. The alignment of foundations along rows is comparable to Layout 1 but with an increased spacing. There are less WTG (174) and using a slender monopile foundation of 8.1 m diameter. The actual scale of tidal wakes is likely to be substantially less than the conservative case presented from the modelling.

Hornsea Three

4.7.4.15 Hornsea Three discussed the implications of changes to water column stratification with associated potential impacts to the Flamborough Front, although different information was used to illustrate the location and development of the front. The considerations were offered in relation to water passing foundations across the array rather than a foundation developing a turbulent wake which passed out of the array with increased mixing potential.

4.7.4.16 The assessment suggested a possibility that when stratification occurred the foundations in the Hornsea Three array may cause some minor decrease in the strength of water column stratification. Only a small proportion of water passing through the array area would actually interact with individual foundations, causing only partial and localised mixing of any stratification. Numerous passes through the array area would also be needed for an initially stratified body of water to become mixed; however, this was considered unlikely due to net displacement of the water body out of the array area over shorter time periods by residual tidal currents. On this basis, stratified water entering the Hornsea Three array area was considered unlikely to become fully mixed. Regional scale patterns of stratification in the North Sea would be unaffected and would continue to be subject to natural processes and variability. The location and physical characteristics of the Flamborough Front were not considered to become measurably affected and would remain within the range of natural variability (Ørsted, 2018).

4.7.4.17 In addition, all other proposed wind farms were considered to be located more than one tidal excursion from the Hornsea Three array area, so there was considered to be no potential for cumulative impacts from turbulent wakes on stratification.

German Bight

4.7.4.18 An investigation into the potential impacts of offshore wind farms on North Sea stratification, in the region of the German Bight, suggested that extensive development, leading to turbulent wakes and increased turbulent mixing, could theoretically impact large-scale stratification, although this was unlikely with the present scale of development which was considered to have a very small impact (Carpenter, et al., 2016).

4.7.4.19 Although not included in their existing analysis, a further hypothesis was made that added drag forces from scour protection (and rock armour on cable crossings) may further exasperate the issue of turbulent mixing beyond the influence of foundations alone.

Summary

4.7.4.20 Hornsea Four will add up to 190 foundation structures across an area of 600 km^2. Each structure will form an individual turbulent wake which will oscillate in position between ebb and flood flows and increase and decrease in length between spring and neap tides. The measurable extent of wakes is likely to be less than the full tidal excursion.

4.7.4.21 Given the spacing and alignment of foundations in the indicative layout, most wakes are likely to remain independent of each other, however, for the ten box-type gravity base structures there remains a potential for larger scale wakes interacting with wakes from adjacent structures. There would appear minimal opportunity for individual turbulent wakes to form a larger array-scale wake affect.

4.7.4.22 The extent of wakes from the PEIR layout created on a spring flood tide could potentially reach the Flamborough Front for locations at the very southern limit of the offshore array, although this will be a spatially and temporally limited effect, comparable to a vessel passing through the same area creating a ship wake.

4.7.4.23 Apart from turbulent wakes, the addition of scour protection material could locally increase seabed roughness, however, the MDS provision for scour protection around foundations and at cable crossings represent less than 0.22% of the offshore array area. The present distribution of sandwaves across the same area has a far more extensive influence on the seabed boundary layer.

4.7.4.24 Collectively, the influence of structures forming turbulent wakes and scour protection on added seabed friction are limited to small-scale effects which are unlikely to be sufficient on their own to increase mixing to destabilise stratification across the array or interfere with the Flamborough Front.

4.8 Changes to waves affecting coastal morphology

Overview

4.8.1.1 Waves acting on the coastline are an important mechanism for eroding the base of the cliffs and transporting sandy material along the beach as longshore drift. The oblique direction of waves arriving at the coastline determines if the longshore transport is to the north or south. The sands that are transported in a northerly direction provide a supply of sediment to help develop and maintain the profile of Smithic Sands. In turn, the profile of this sandbank feature also acts to dissipate some of the wave energy (due to shoaling effects) associated with large storm waves moving towards Bridlington before reaching the coastline. Substantial modification to waves arriving at the coastline has the potential to affect the balance in these nearshore processes.

4.8.1.2 There will always be some intra-annual and inter-annual variability in wave conditions. In addition, climate change may also modify the frequency, magnitude and direction of storm tracks, although there is limited certainty at this time on the how these changes may be manifested.

4.8.1.3 Offshore structures can also interfere with the transmission of wave energy reaching the coastline through various forms of interaction, most notably through reflection and scattering off the vertical surface and through drag forces (skin friction) as waves pass around structures. The added effect of diffraction depends on the relative scale of the obstacle versus the wavelength of the passing wave. For slender monopiles, the diameter of the obstacle is generally too small for diffraction to occur. When the (effective) diameter (D) is large relative to the incident wavelength (L) then diffraction effects become important. The criterion for diffraction is generally accepted to be when the ratio of D/L > 0.2 (Isaacson, 1979). Collectively, the interactions between an incident wave and a structure are regarded as blocking type effects with a downwind change possible in wave height, period and direction. The downwind change is also referred to as a (wave) wake.

4.8.1.4 Array scale blocking can also form when a foundation develops a wake that extends to a down-wind structure which then adds to the wake. Wake recovery normally occurs beyond the array through dissipative effects with wave recovery also possible by further wind related stresses.

4.8.2 Changes to waves at HVAC booster area

4.8.2.1 The HVAC booster area is situated in the offshore ECC from around 34 to 42 km from the coast. Within this area there is an option for up to three 75 m wide box-type GBS foundations. When these structures are at 45° to incident waves their effective width becomes 106 m. Water depths at this location are generally 50 m below CD.

4.8.2.2 The precise location, spacing and orientation of the three foundations remains unknown at this time; however, there is a stated minimum separation of 100 m between foundations.

4.8.2.3 Waves moving towards the coastline from the HVAC booster area are likely to be similar to measurements further offshore since water depths are generally too deep to lead to any shoaling or refraction effects modifying wave energy transformation and there are no sheltering influences from the coastline. Indicative wavelengths for wave periods in the range 4 to 8 s are 26 to 100 m. The ratio of D/L indicates diffraction is important for the large structures.

4.8.2.4 The worst-case effect of the HVAC booster station foundations on waves is for the situation when their combined effective width and separations are aligned to become an effective barrier to waves over a total width of more than 300 m. Waves would reflect and scatter off the incident faces of structures and diffraction would occur around the structures redistributing wave energy into the shadow zone created by the structure.

Hornsea Project One

4.8.2.5 A single offshore HVAC reactive compensation substation was included in the assessment of waves for Hornsea Project One. A worst-case foundation option with a 50 m base diameter GBF structure was assessed for a location in a water depth of around 24 m below CD and around 53 km offshore of Spurn Head. The assessment for Hornsea Project One concluded the following (SMart Wind, 2013):

"Wave scattering around the structure will occur, and will be greatest for the GBF, but the effects will be spatially limited due to the single foundation. As the offshore HVAC reactive compensation substation is located in deep water offshore, it will not affect the wave climate at the shoreline."

<u>Hornsea Project Two</u>

4.8.2.6 Two offshore HVAC reactive compensation substations were included in the assessment of waves for Hornsea Project Two. The worst-case foundation option was also the 50 m GBF structure. These structures were to be located alongside the Hornsea Project One structure with all three notionally 500 m apart along an alignment across the shared offshore ECC. The assessment for Hornsea Project Two concluded that they would not affect the wave climate at the coast (SMart Wind, 2013).

<u>Hornsea Three</u>

4.8.2.7 Hornsea Three considered up to four offshore HVAC booster stations halfway along the offshore cable corridor. The base dimension of these structures was 75 m for a box-type GBS. The layout or separation of these structures was not specified. The assessment concluded that they would not affect the wave climate at the shoreline. (Ørsted, 2018).

<u>Summary</u>

4.8.2.8 The HVAC booster station foundation structures for Hornsea Four are comparable to those proposed for Hornsea Three but with only three structures. Whilst waves will undoubtably locally interact with these structures their distance offshore is considered to be sufficient for any wave modifications to be fully dissipated before a measureable effect reaches the coast.

4.8.3 Changes to waves at the offshore array

4.8.3.1 There are three types of foundations in the offshore array area which will interact with waves. The type of interaction will depend on their size and shape, as well as their incident wave characteristics:

- 180 WTG suction bucket jackets with an effective seabed width of 65 m, narrowing to 25 m at the sea surface (with an assumed solidity ratio of 0.3);
- 3 large GBS box-type with 150 m width base; and
- 7 medium GBS box-type with 75 m width base.

4.8.3.2 The size and shape of the suction bucket jacket is expected to have a much lesser interaction with waves than a GBS foundation type with an equivalent base diameter due to the open lattice arrangement of the jacket structure.

4.8.3.3 Additional interaction of waves may occur across the array between adjacent foundations. This type of interaction depends on the relative spacing and orientation to incident waves that also allows a wake effect to pass along and reach the downwind foundation. The array scale interaction represents the aggregate of all foundation interactions and becomes the more relevant consideration for effects on the far-field.

4.8.3.4 The distribution of foundation types across the indicative layout for Hornsea Four is unknown at this time, neither is the orientation nor spacing between any of the box-type GBS foundations which are expected to lead to the greatest modification to incident waves.

4.8.3.5 A comparison of the relative blockage at the scale of an array for all projects within the former Hornsea Zone is offered based on the scale occupied by all foundation per array area (Table 24). Although this first order metric of relative blockage for array ignores the shape of each array, the foundation layouts and scales of any specific foundation type, the comparison between projects remains useful to indicate likely scales of effect on waves for comparable sized arrays.

Table 24: Comparison in scale of relative blockage for projects within the former Hornsea Zone.

Project	Status	Array Area (km^2)	Number of foundations	Footprint of all foundations (km^2)	Relative blockage for array (%)
Hornsea Project One	Consented	407	335	0.65	0.162
Hornsea Project One	Final	407	174	0.01	0.002
Hornsea Project Two	Consented	462	258	0.68	0.148
Hornsea Project Two	Final	462	165	0.01	0.003
Hornsea Three	Application	696	319	0.77	0.111
Hornsea Four	PEIR	600	190	0.38	0.063

4.8.3.6 The consented Hornsea Project One has the highest array scale blockage (resulting from 335 50 m diameter gravity bases) which would be expected to produce the greatest effect on waves for a comparable area. In this case, Hornsea Project One has now been built with a fewer number of smaller 8.1 m diameter monopile foundations which dramatically scales down the array scale blockage effect by a factor of 73. In addition, these smaller diameter piles would no longer expect to exceed the criteria for diffraction.

4.8.3.7 Existing wave modelling assessment from Hornsea Project One and Two and Hornsea Three are reviewed to help establish likely scales of wave reduction from Hornsea Four.

Hornsea Project One

4.8.3.8 A review of array blockage effects on waves has been completed based on the comparison between pre- and post-construction observations to the north and south of the array. This review is presented in **Appendix C** and concluded no discernible changes in wave heights attributed to the presence of the monopile foundations now in place for Hornsea Project One.

4.8.3.9 This is an important moderation to what was presented in the assessment of waves for consent application.

4.8.3.10 The method for wave modelling was also assessed independently to be highly conservative in the way areas of foundations were represented in the far-field wave model (SWAN) by imposing wave transmission reductions across groups of 3 x 5 structures. This approach prevented any waves from passing through the array unimpeded and gave the array a blanket effect of reducing all waves passing through the area. The method of determining wave transmission reductions was also biased by the apparent limitation of the near-field wave model grid not being capable of resolving short period waves typical of the area (assessed by measurements to be typically in the range 4 to 6 s and up to 8 s in peak events) and instead was only applicable for wave periods between 8 to 16 s.

Hornsea Project Two

4.8.3.11 The construction of Hornsea Project Two is planned for 2020. The expectation is that 165 10 m diameter monopiles will be used instead of the maximum consented option for 258 58 m diameter GBS foundations. This change reduces the array scale blockage by a factor of 53. Similar to Hornsea Project One, this is a further important moderation to what was presented in the assessment of waves for consent application and for any cumulative impact concerns between these two projects as well as any adjacent projects.

4.8.3.12 For reference, the same wave modelling approach provided for Hornsea Project One was repeated for Hornsea Project Two, inferring the same conservative outcomes and bias in the results.

Hornsea Three

4.8.3.13 Waves assessments were considered using a variety of approaches, including an alternative wave model (MIKE21 SW). Various configurations of wind farms were modelled (Ørsted, 2018):

a. Hornsea Project One and Two alone in their worst-case configurations;
b. Hornsea Project One revised to the updated final configuration, and Hornsea Project Two and Hornsea Three in their worst-case configurations. This represents the cumulative case; and
c. Hornsea Three alone for the worst-case configuration.

4.8.3.14 **Figure 35** presents patterns of wave height reduction for these different configurations for the 50% non-exceedance wave height which represent typical conditions rather than less frequent stormy events. The equivalent offshore wave height was 1.1 m and period of 5 s for N, NE and NNE wave directions. These directions are selected as they were the only directions with modified waves potentially reaching the coast around Suffolk. No scenarios impacted the Holderness Coast.

4.8.3.15 Results for configuration **a.** are shown for completeness but are now largely superseded by the alternative configurations in place for Hornsea Project One and planned for Hornsea Project Two which would markedly reduce such effects.

4.8.3.16 Results for configuration **b.** introduce Hornsea Three but also modify Hornsea Project One to the final configuration. This lessens the amount of wave height reduction from this array and only the northerly scenario has wave height reduction reaching the coast. If Hornsea Project Two was also now updated, then no wave reduction effects would be expected to reach the coast when all three arrays are represented together.

4.8.3.17 Results for configuration **c.** are for Hornsea Three in isolation. No wave reductions reach the coast. The relative array blockage for Hornsea Three is also approximately double that expected for Hornsea Four, so a similar and lesser spread of wave reduction would be expected if these effects were transferred to Hornsea Four.

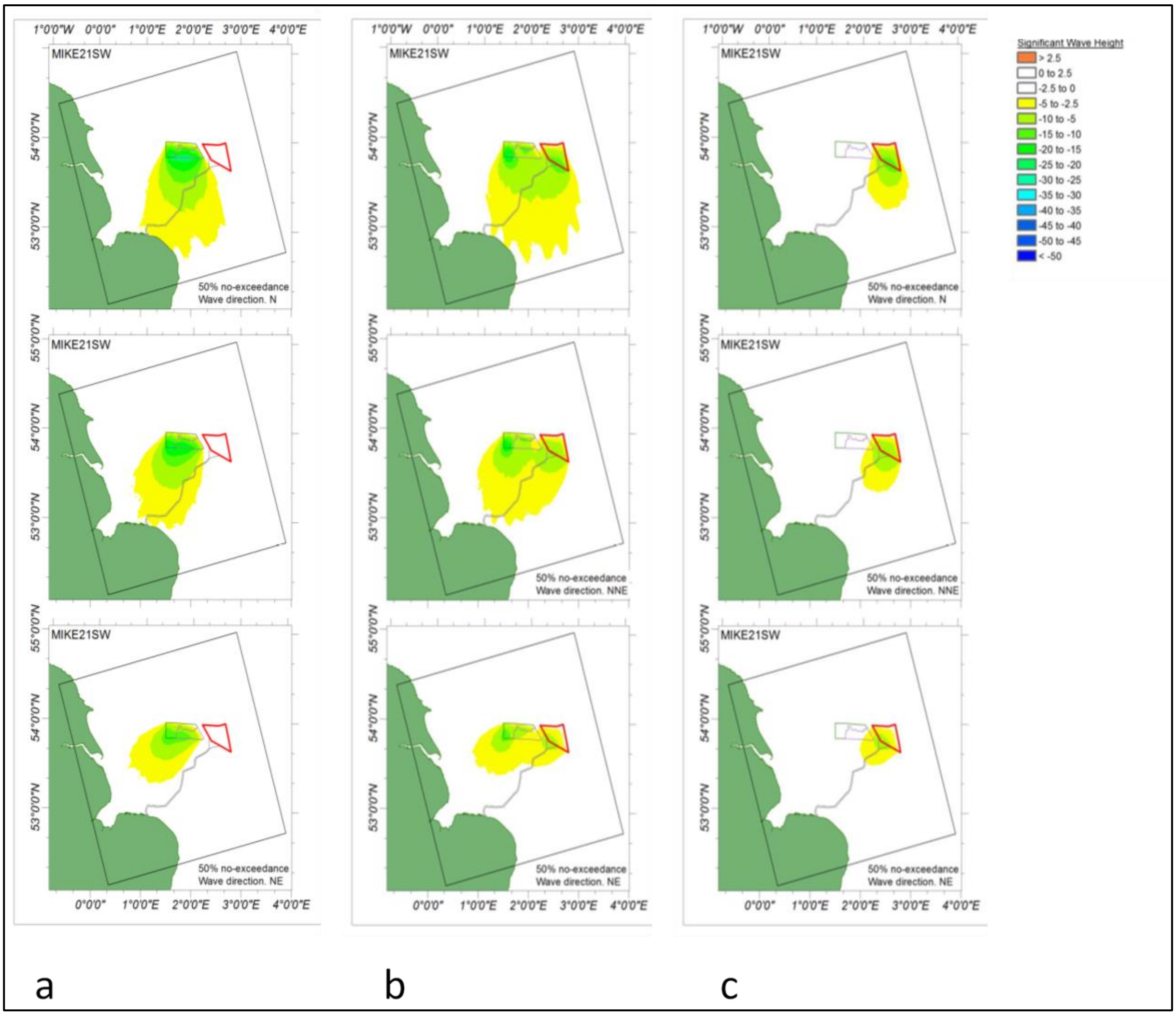

Figure 35: MIKE 21 SW wave modelling of the 50% non-exceedance waves from N, NNE and NE wave directions (from Ørsted (2018)).

Summary

4.8.3.18 Based on the various wind farm configurations represented in the existing wave modelling it is possible to deduce that wave height reductions from Hornsea Four in isolation, or in combination with Hornsea Project One and Hornsea Project Two (final configurations) and Hornsea Three, would not reach the adjacent coastlines and lead to any effects on coastal morphology.

4.9 Changes to nearshore sediment pathways

Overview

4.9.1.1 The nearshore is considered here to be the shallowing area within the shelter of Flamborough Head up to the coast, including Smithic Sands. The relevant activities that might lead to a change in nearshore sediment pathways are considered to include:

- Landfall works involving open-cut trenching;
- Cable installation activities along nearshore section of offshore ECC;
- Cable crossings with Creyke Beck – Dogger Bank offshore ECC;
- HVAC booster area foundations; and
- Requirements for remedial measures to rebury cables.

4.9.1.2 The important nearshore sediment pathways are summarised on **Figure 15**. Cliff erosion by storm waves provides an important source of beach material which is moved along the coast by wave driven longshore drift. Some of this material is transported offshore into an ebb dominant tidal channel where the pathway moves material towards Flamborough Head. Ebb flows, reinforced by wave driven current from north of the headland maintain a one-way drift to the south which then forms a pathway for sands onto Smithic Sands. Waves help to limit the profile of the bank with larger waves dissipating some of their energy onto the bank creating a southern section of the bank which is wider and smoother than the northern part of the bank where tidal flows accelerate around the headland and act to develop distinct sandwave bedforms.

Short-term construction activities

4.9.1.3 Landfall works and cable installation activities are considered short-term and will not leave permanent structures on the seabed which may modify nearshore sediment pathways.

Long-term installations

4.9.1.4 The cable crossing of Creyke Beck – offshore ECC has been discussed in **Section 4.6.2**. Depending on the length and profile of any rock berm in this location, then some disruption might be expected on existing offshore sediment pathways moving material onto Smithic Sands due to tidal flows sweeping sands from the one-way net southerly drift direction around Flamborough Head **(Figure 15)**. Any deflected flows along the rock berm may encourage the bedload pathway here to be locally redirected to the south-east for the length of the rock berm.

4.9.1.5 The three HVAC booster station box-type GBS have the potential to locally modify waves in an offshore area fronting the nearshore (**Section 4.8.2**). The amount of wave energy moving from the east, passing the HVAC booster stations and then onward to the coast is likely to be reduced at some level, although the further distance to the coast will both dissipate this effect and also allow wind stress to renew waves over the remaining fetch. The scale of this wave reduction effect on sediment transport is likely to be minor in the nearshore. Smithic Sands will still act more locally to absorb wave energy heading to the coast and be the final moderator to wave driven longshore drift.

4.9.1.6 The potential is for cables to become unburied at any location, including the nearshore. This may happen due to anchor dragging or changes in seabed levels across an area with a mobile seabed, for example. The profile of any rock armour will generally follow the alignment of the cable with a profile which may be locally higher than the adjacent seabed.

The rock armour may then act as a partial barrier to bedload sediment transport along the length of the rock berm. Material in suspension is not expected to be affected. Depending on the situation, coarser grained mobile sediments may build up against this barrier where flows are weakened, as well as bypass around the ends where flows accelerate.

4.10 Decommissioning effects

Sediment disturbance

4.10.1.1 Decommissioning issues include sediment disturbance events during removal of foundations as well as cables.

4.10.1.2 All decommissioning activities are likely to have a comparable type (but lesser magnitude) of sediment disturbance than any activity described during construction for seabed preparation for foundations and cables. Accordingly, the level of any impacts from decommissioning can be considered smaller than those described for construction.

Blockage

4.10.1.3 Once foundations are removed their associated blockage effects will also cease. This returns the wave and tidal conditions back to a condition that represents a future baseline. Most blockage effects are remote from any receptors, so a potential reinstatement of a higher energy situation is unlikely to lead to any concern.

4.11 Cumulative Effects

Overview

4.11.1.1 Cumulative impacts result from the combined effect of Hornsea Four in combination with the effects from a number of different projects or activities, on the same single receptor/resource. For marine processes, the following projects and activities have been identified for potential cumulative impacts:

- Spoil disposal at HU015;
- Creyke Beck export cable landfall works; and
- Hornsea Project One and Hornsea Project Two blockage effects.

4.11.2 Spoil disposal activities

4.11.2.1 The spoil site HU015 is used to dispose of maintenance dredgings from Bridlington Harbour which are typically silts. During these times, plumes will form at the disposal site as the silts are rapidly dispersed away. The use of the spoil site is expected to be relatively infrequent and on demand. **Paragraph 3.2.3.12** suggests the typical number of disposals expected each year. This varies year to year and month to month.

4.11.2.2 If Hornsea Four is discharging overspill of fine silts and sands in the nearshore from cable trenching by MFE on an ebb tide period at the same time as spoil disposal is occurring at HU015 then a larger and sediment plume may form, however, this will also quickly disperse given the location of the spoil site in an area of faster flows. The cumulative impact is considered to be negligible due to the low likelihood of occurrence and relatively short-term impacts.

4.11.3 Creyke Beck export cable landfall works

4.11.3.1 The assumption is that all landfall works for Creyke Beck will be completed and the area made good before similar landfall activities occur for Hornsea Four. On this basis there are not expected to be any larger cumulative effects on the integrity of the local beach.

4.11.4 Hornsea Project One and Hornsea Project Two

4.11.4.1 Hornsea Project One and Hornsea Project Two are immediately adjacent offshore wind farms to Hornsea Four. The consented layouts and foundation types for both Hornsea Project One and Hornsea Project Two assumed GBS foundations with wide bases that would have had a blockage effect on waves and flows which could have acted cumulatively with Hornsea Four. The moderation of this potential concern for a greater level of blockage now exists because both Hornsea Project One and Hornsea Project Two are being developed with an alternative layout with a fewer number of smaller diameter foundations which will dramatically reduce the effective scale of blockage both for an individual foundation and for all foundations at the arrays scale.

4.11.4.2 Hornsea Three is considered to be less relevant to possible cumulative interactions for blockage because of the further distance from Hornsea Four, the flow and sediment pathways not passing between these two projects and that waves are mainly from the northerly sector. On this basis Hornsea Three is excluded from the cumulative effects with Hornsea Four.

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Appendix A - Comparison of marine processes across the former Hornsea Zone

1. Introduction

The marine processes assessment for Hornsea Four is delivered with an evidence-based approach where existing assessments from adjacent offshore windfarm developments are considered for supporting information and to help determine scales of potential impacts. The basis of this approach is that comparable projects in comparable environmental settings can be expected to lead to comparable effects.

The justification of comparable environmental conditions is made by reviewing the similarities and differences of the baseline environmental setting of Hornsea Four with the adjacent offshore windfarms of Hornsea Project One and Hornsea Project Two, in particular, given their close proximity. The comparative review is offered for array areas in the main since the export cable corridor and landfall location of Hornsea Four is geographically separate.

The following marine processes topics are considered:

- General setting;
- Bathymetry;
- Tidal levels;
- Flows and excursions;
- Waves;
- Surficial sediments;
- Bedforms; and
- Suspended sediment.

2. Primary evidence

In the main, comparisons between project areas are offered with reference to regional scale mapping, this includes EMODnet for bathymetry and surficial sediments, the UK Atlas of Marine Renewable Energy for flows, tidal excursions and waves, and synoptic maps of SPM derived from satellites.

Where helpful, consideration is also made to the zonal metocean survey to help validate aspects of the regional scale information.

3. General setting

The former Hornsea Zone is being developed as four separate projects. The offshore wind turbine array areas are located as follows, from west to east;

- Hornsea Four is (relatively) closest to the coast, with Flamborough Head around 70 km to the west. The offshore array of Hornsea Four covers an area of 600 km^2.
- Hornsea Project Two abuts to the south-eastern corner of Hornsea Four and extends further eastward. This project covers around 462 km^2 of seabed.
- The northern and western borders of Hornsea Project One join up with Hornsea Project Two. This project covers around 407 km^2 of seabed.

- Hornsea Three is the most easterly project and is also separated from Hornsea Project One by a shipping lane around 7.2 km wide. Hornsea Three is over 140 km from the coast, at the closest point. This project covers around 697 km^2 of seabed.

The four projects cover a distance of around 125 km west to east. The similarities and differences between these areas is considered over this distance.

In their respective settings, all windfarm areas can be considered to be remote from the immediate influence of the adjacent coast and are subject to "offshore" type conditions. A further common association between projects is that the northern boundary of each area is defined by Outer Silver Pit.

4. Water depths

Water depth is a consideration with regard to the potential for waves to stir the seabed and influence sediment transport. In addition, the fate of dredgings falling to the seabed, and their capacity to be advected in this period, varies between shallow and deeper water for the same flow conditions.

The general water depths, neglecting any tidal contributions or presence of large bedforms, for each project area are summarised with reference to EMODnet bathymetry, as follows:

- Hornsea Project One is typically 30 to 35 m deep across the array;
- Hornsea Project Two is typically 30 to 40 m;
- Hornsea Three is typically 30 to 45 m with local deviations up to 70 m (e.g. Markham's Hole); and
- Hornsea Four is typically 40 to 55 m, sloping into deeper water to the north where there also larger bedforms.

All depth values reference metres below CD.

4.1 Tidal influence

There is a slight variability in tidal levels across the zone with a small increase in amplitude with distance (to the west) from two tidal amphidromes in the southern North Sea. Hornsea Project One and Two are considered together in this comparison since they occupy a similar east-west position.

For mean spring tides, east to west;

- Hornsea Three: 2 to 2.5 m (Site L3 = 2.21 m);
- Hornsea Project One and Hornsea Project Two: 2.5 m to around 3 m (Site L2 = 2.67 m); and
- Hornsea Four: 3 to 3.5 m (Site L1 = 3.28 m).

For mean neap tides, the corresponding tidal range values are:

- Hornsea Three: 1 to 1.25 m (Site L3 = 1.09 m);

- Hornsea Project One and Hornsea Project Two: 1.25 m to around 1.5 m (Site L2 = 1.31 m); and
- Hornsea Four: 1.5 to 1.75 m (Site L1 = 1.61 m).

These tidal influences would all be additive to the general water depths referred to previously, maintaining Hornsea Four as the deepest area overall and Hornsea Project One as the shallowest, in comparison.

5. Metocean Conditions

5.1 Overview

Metocean conditions are considered here for tidal flows, tidal excursion and waves.

Tidal flows and excursion are relevant to the advection and dispersion of materials discharged into the marine environment such as dredged overspill and spoil disposal. The excursion provides an indication of both range and direction of advection over a tidal cycle.

Areas with higher flow speeds may be able to mobilise more sediment types more often than areas with weaker flows, a consideration for deposition and remobilisation of sediments in spoil.

Waves can influence sediment transport where their stirring effect reaches the seabed.

Waves and flows are also associated with the potential scale of blockage related effects. For tidal related blockage the axis of any wake effects is likely to remain with the primary axis of the tidal ellipse.

5.1 Tidal flows

There is only a slight variability in tidal flows across the zone with a small increase to the west from Hornsea Three.

For mean spring tides, east to west, peak speeds of:

- Hornsea Three: 0.30 m/s;
- Hornsea Project One and Hornsea Project Two: 0.55 to 0.70 m/s; and
- Hornsea Four: 0.55 to 0.70 m/s.

For mean neap tides, the corresponding peak flow speeds are:

- Hornsea Three: 0.25 to 0.30 m/s;
- Hornsea Project One and Hornsea Project Two: 0.30 to 0.35 m/s; and
- Hornsea Four: 0.30 to 0.35 m/s.

5.2 Tidal excursion

The shape of the tidal ellipse changes from a rectilinear form across most of Hornsea Three to a more rotary form across Hornsea Four (east to west) with some transition between these two forms across Hornsea Project One and Hornsea Project Two.

The excursion distance for sites central to each project area are described in **Table A 1**.

Table A 1: Excursion distances for sites central to each project area.

Project Area	Hornsea Three	Hornsea Project One	Hornsea Project Two	Hornsea Four
Mean spring excursion (km)	6.5	8.1	8.5	8.3
Mean neap excursion (km)	3.5	4.2	4.4	4.1

The axis of the ellipse also slightly varies from Hornsea Three (in the east) to Hornsea Four (in the west) from an east-south-east to a north-east flood direction and a west-north-west to a north-west ebb direction, respectively.

5.3 Waves

Average winter wave heights may be slightly larger in Hornsea Three than the other project areas although this variability is unlikely to be important to any local sediment transport processes since water depths are the limiting condition on wave energy attenuation onto the seabed.

6. Suspended Sediments

In general, suspended sediment concentrations (using a proxy of surface SPM), are relatively low for all project areas. There is a slight increase from west to east with winter levels in Hornsea Four up to 2 mg/l increasing to 5 mg/l in Hornsea Three.

7. Surficial sediments

The main relevance of surficial sediments is in relation to sediment disturbance events and the relative content of finer sediment (fine sands, silts and muds). In general, all sites can be considered as having coarser sediments with limited mud content. The relative content of gravels and sands shows more variability but this size of material will generally all fall out of suspension when disturbed.

- Hornsea Four is mainly sandy with some patches of slightly gravelly sand;
- Hornsea Project Two is sandy in the north but slightly gravelly sand in the south;
- Hornsea Project One shows more gravel content, with some patches of sand, slightly gravelly sand, gravelly sand and sandy gravel; and
- Hornsea Three has similar variability in surficial sediment as Hornsea One but with muddy sand in Markham's Hole.

8. Bedforms

Although sandwaves are found across all project areas, their prominence is greatest in the northern part of Hornsea Three where the features merge into a wider set of more dominant sandcrests and sandbanks known as The Hills.

9. Geology

The underlying solid geology of the region is complex and is overlain by varying thicknesses of Quaternary sediments. These generally increase in thickness in an easterly direction and may be 200+ m thick in the east of the former Hornsea Zone. The geophysical survey from the former Hornsea Zone suggests the Bolders Bank Formation is extensive across the area surveyed, and it is generally mantled by varying thicknesses of recent seabed sediment.

10. Summary of environmental conditions

A regional scale comparison has been provided to demonstrate similarities and differences in environmental conditions between Hornsea Four and Hornsea Project One, Hornsea Project Two and Hornsea Three.

Similarities and difference occur between all four sites with some general trends in the parameters under consideration and mainly as spatial variance east to west.

Hornsea Project Two is most comparable, as might be expected due to the closest proximity to Hornsea Four. The environmental conditions across Hornsea Project One are similar to Hornsea Project Two.

In relative terms, Hornsea Three is the least comparable, due to the furthest distance from Hornsea Four. Main differences are in the water depth, flow speeds, and sediment types.

Accordingly, the application of the evidence base to support Hornsea Four focuses mainly on the information available from Hornsea Project One and Hornsea Project Two.

Appendix B - Data and Information

Table B 1 summary of the key data and information which has informed the marine processes baseline understanding.

Table B 1: Key sources of marine processes data and information.

Source	Summary	Coverage
Zonal characterisation (ZoC) including metocean, geophysical and benthic surveys (SMart Wind, 2012)	Initial broad-scale evaluation of the Hornsea zone to help establish areas for development. This work was supported by extensive baseline metocean, geophysical and benthic surveys	**Offshore array**: metocean site L1 within the southern part of the AfL. Site L6 offers characterisation of the northern area, but is slightly west of the AfL. Waves, tides, currents, OBS, ABS, temperature.
Existing wave and tidal models (SMart Wind, 2015a)(SMart Wind, 2015a)(SMart Wind, 2015a), (SMart Wind, 2015b)(SMart Wind, 2015b)(SMart Wind, 2015b) and (Ørsted, 2018)(Ørsted, 2018)(Ørsted, 2018)	Wave and tidal models previously calibrated against ZoC metocean survey data provide existing outputs offering an expanded view of baseline conditions as well as a quanitifed assessment of potential impacts. Existing model outputs support simplified assessment approaches for Hornsea Four.	**Landfall:** full coverage **Offshore array:** full coverage **Offshore ECC:** full coverage
Atlas of UK Marine Renewable Energy (DECC, 2008)	Synoptic regional-scale description of waves, tidal levels and currents to complement other baseline information.	**Offshore array:** full coverage **Offshore ECC:** full coverage
The European Marine Observation and Data Network (EMODnet) for thematic mapping of bathymetry, seabed substrate and geology	Baseline mapping of bathymetry, seabed substrate and sub-surface geology to provide a regional overview of seabed conditions, complementing site specific surveys.	**Offshore array:** full coverage **Offshore ECC:** full coverage
GeoIndex	Database of analysed surficial sediment samples providing quantification of sand, gravels and mud content, directly complements EMODnet seabed substrates	**Offshore array:** full coverage with multiple samples **Offshore ECC:** full coverage with multiple samples
Southern North Sea Sediment Transport Study (HR Wallingford, CEFAS/UEA, Posford Haskoning, and Brian D'Olier, 2002)	An in-depth review of the sediment transport regime across the Southern North Sea	**Offshore ECC:** nearshore description of net sediment transport direction and indicators
Sand banks, sand transport and offshore wind farms (Kenyon & Cooper, 2005)	Complements the Southern North Sediment Transport Study, offering a UK-wide and regional perspective	**Offshore array:** check **Offshore ECC:** description of net sediment transport pathways

Source	Summary	Coverage
	of sediment pathways. Highlights relevance of sand transport issues to offshore wind farms	
Suspended sediment mapping (CEFAS, 2016)	Synoptic description of baseline seasonal (monthly) variation in surface SPM across the study area derived from long-term satellite observations	**Landfall:** sub-tidal only **Offshore ECC:** full coverage **Offshore Array:** full coverage
Temperature forecast modelling from Copernicus Marine Environmental Monitoring Service	Regional scale 3D forecasts of temperature structure to help describe development and location of Flamborough Front. Daily values obtained for 2018.	**Offshore ECC:** full coverage **Offshore Array:** full coverage
Nearshore seabed survey: Flamborough Head to Spurn Point (Channel Coastal Observatory, 2014)	Detailed mapping of coastline, including LiDAR, multibeam and EUNIS habitat mapping	**Landfall:** majority of area included **Offshore ECC:** partial coverage of nearshore, inclduing parts of Smithic Sands
UKHO	Digital soundings across study area to augment other sources of bathymetry data	**Offshore ECC:** partial coverage **Offshore Array:** partial coverage
Shoreline monitoring (ERYC)	Long-term monitoring of cliff recession and beach profiles to low water	**Landfall:** intertidal areas
Creyke Beck EIA	Particle size analysis of grab samples in the nearshore Nearhsore geophysical survey	**Offshore ECC:** partial nearshore coverage
BODC	Current meter records across the study area to augment other sources of flow information	**Offshore ECC:** limited coverage **Offshore Array:** limited coverage

Sediment classification

Where possible, the synthesis of evidence to describe the composition of surficial sediment across the study area adopts a consistent classification scheme based on Folk (1954). This approach enables individual sediment samples with particle size data to be coded with the same descriptions used in broad-scale sediment maps from EMODnet. This procedure also offers local scale validation of these sediment maps with site specific data by comparing regional interpretation of sands with quantified sands using grab samples and particle size analysis. Where validated, this also provides the means to infer other sediment properties to the regional maps, for example medium sands from the grab sample matching an area interpreted as sands on regional scale interpretations infers the mapped area of sands is likely to comprise of medium sized particles.

Sediment descriptions offered in the PEIR, including any mapping, are generally based on this descriptive classification scheme. Where necessary, information from particle size analysis of grab samples is also considered to provide additional quantification of sediment sizes.

Appendix C - Assessment of Waves at HOW01

Preface

A technical note providing a review of operational wave monitoring at HOW01 has been produced in a response to a suggestion made by Natural England at the first Marine Processes Evidence Plan discussion on 12 September 2018.

The technical note was distrubuted to members of the Marine Processes Evidence Plan forum (Natural England, MMO and Cefas) in February 2019 for their consideration. Subsequently, MMO (and Cefas) responded with comments in March 2019.

A discussion was held with MMO, and their techncial advisors; CEFAS, on 3 June 2019 to review their comments and agree an approach whereby all parties could reach a consensus on the findings of the technical note.

The version of the techncial note included in the PEIR remains as version 1a with the intention of completing the agreed actions to update this note for submission as part of the EIA report.

For completeness, a copy of the agreed actions is included below, for reference.

DCO/2018/00014 – Hornsea 4

Teleconference 03/06/19 – final actions

Attendees

- Bill Cooper (Cooper Marine Advisors) - **BC**
- Jon Rees (Cefas) - **JR**
- Melissa Gaskell-Burnup (MMO) - **MGB**

The HOW04 Marine Processes Evidence Plan Technical Note 1 Version A (January 2019) was discussed during this call. The technical note focuses on the information currently available from the operational wave monitoring from pre and during construction of Hornsea 1. A number of actions were identified:

- **BC** will re-explore the validity of the null hypothesis for north westerly waves.
- The *wave heights (in metres) based on statistical best-fit trendline relationships* (for example, table 5) tables will be updated to include a fifth column to quantify the variance of the scatter around the best-fit. JR will advise on a preferred statistical property, noting a comment from BC that this should ideally come from a statistical relationship already built into Excel. - **JR**
- A staged approach to assessing the potential geographical extent of impact on wave heights was agreed. The case to extend the analysis to further sites needs to be justified on the basis that a clear signal of change is detected from the existing comparison across the array. – **BC.**
- **BC** to pick one of the directional sectors to explore what happens to the statistics (as a sensitivity check) if we compare pre and post construction on a more like to like basis using a common range in wave heights. This would include a directional sector where one of the pre-construction periods had statistics based on waves up to 4-5m but the post construction only had up to 4m. BC has suggested to remove the 4-5m statistics.
- **BC** will plot the September storm event in Figure 11 in more detail so a single peak event comparison between WB1 and WB2 can be made.
- The updates to the technical note may not be complete in time for PEIR submission, as any further revisions also need to be agreed with Orsted. Should the existing note be included in the PEIR document then it will be made explicitly clear that the agreed scope of further review work is in progress and the updated version will be submitted with the formal application. **All** to agree to this approach.

PUBLIC

Hornsea 4

Ørsted

Hornsea Project Four Offshore Wind Farm – Evidence Plan

Marine Ecology & Processes Technical Panel

Technical Note - Marine Processes: Operational Wave Monitoring Assessment

February 2019

Prepared	Bill Cooper (Cooper Marine Advisors), 17 January 2019
Checked	Steve Bellew (GoBe), 04 February 2019
Accepted	Lauren Kirkland (GoBe), 22 February 2019
Approved	David King (Ørsted), 25 February 2019

Doc. no. 01493919 Ver. no. A

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Tables

Figures

Acronyms

AWAC	Acoustic Wave and Current
Cefas	Centre for Environment, Fisheries and Aquaculture Science
D	Diameter (m)
E	East
ENE	East-north-east
ESE	East-south-east
EIA	Environmental Impact Assessment
ES	Environmental Statement
GBS	Gravity-based structure
Hornsea Project One	Hornsea Project One Offshore Wind Farm
HOW04	Hornsea Four Offshore Wind Farm
H_s **or** H_{m0}	Significant wave height (m)
km	kilometre
L	Wavelength (m)
m	metre
MSL	Mean sea level
MMO	Marine Management Organisation
N	North
NE	North-east
NNE	North-north-east
NNW	North-north-west
NW	North-west
°N	Degrees north
RP	Return period (in years)
s	second
S	South
SE	South-east
SSE	South-south-east
SSW	South-south-west
SW	South-west
T_{m02}	Mean wave period (s)
T_p	Peak wave period (s)
R	Correlation coefficient
R^2	Coefficient of determination
W	West
WNW	West-north-west
WSW	West-south-west
WB	Wave buoy

1. Aim of this Technical Note

Following receipt of the Planning Inspectorate's Scoping Opinion for the Hornsea Project Four Offshore Wind Farm (Hornsea Four) dated November 2018 and the second Hornsea Four Marine Ecology and Processes Technical Panel meeting on 12th December 2018, consultee feedback from Natural England, the Marine Management Organisation (MMO), and Cefas noted that testing the marine processes modelling from previous projects with post-construction data (from operational wave monitoring) was important if no further modelling is proposed for Hornsea Four. An initial examination of the Hornsea Project One Offshore Wind Farm (Hornsea Project One) wave data was presented at the Evidence Plan meeting on 12th December. All attendees agreed that circulation of a more in-depth technical note would be beneficial for consultation prior to production of the Preliminary Environmental Information Report (PEIR), with a specific request from Cefas for consideration of sectoral wave direction and peak events as part of the review.

This technical note has been prepared by Cooper Marine Advisors Limited for GoBe Consultants Ltd and on behalf of Ørsted Hornsea Project Four Limited (hereafter referred to as Ørsted).

2. Scoping

Hornsea Four submitted a Scoping Request and Scoping Report to the Planning Inspectorate (PINS) on the 15th October 2018 under Regulation 10 of the Infrastructure Planning (Environmental Impact Assessment) Regulations 2017 (the EIA Regulations).

Natural England, in relation to marine processes, provided the following scoping response to PINS:

"Several assessments on these chapters (and as well as other chapters) draw on data from previous models developed for the former Hornsea zone. Although Natural England recognises the potential applicability of these models for Hornsea Project Four and agrees with maximizing the use of these data sets and previously developed models for the Hornsea zone, we would like to further justification of the applicability of these models to the Hornsea Project 4 area and to see these models testing using actual data that is increasingly becoming available from the Hornsea zone and other projects where relevant. Testing previous models has already been proposed at the evidence plan technical panel meetings and Natural England would like to reiterate the relevance of testing the models used. If it is established that the models are reflecting reality correctly then there is greater confidence in extrapolating those models to Hornsea Project Four".

3. Hornsea Project One

Hornsea Project One is the first offshore wind farm within the former Hornsea Zone to be constructed. The wind farm is around 14 km east of Hornsea Four and covers an area of 407 km^2. The seabed profile is relatively flat with depths around 30 m below mean sea level (MSL). According to The Crown Estate lease areas, the project is sub-divided into west, centre and east areas (Figure 1).

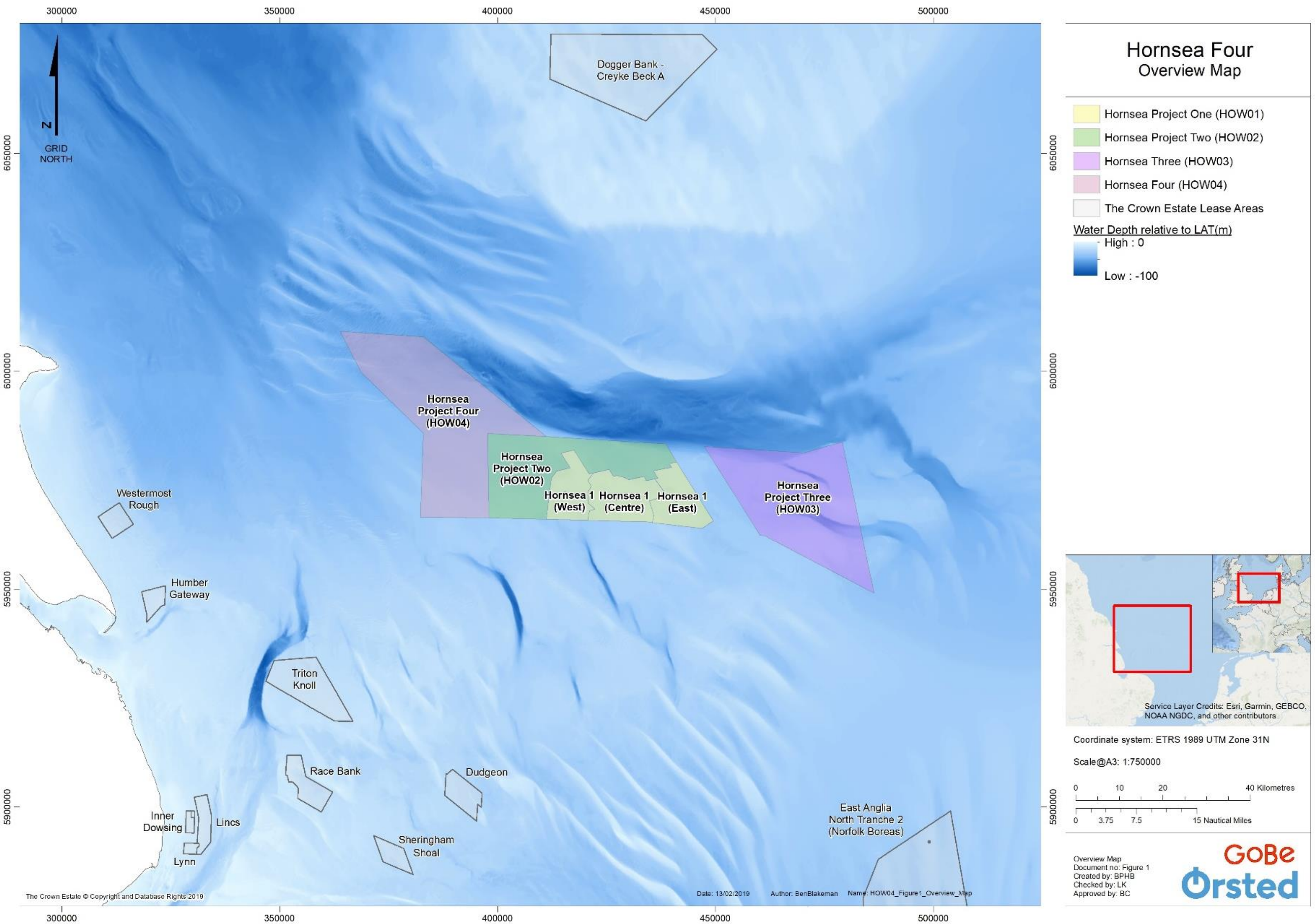


Figure 1: Overview map

4. Scope of the review

This review focuses on the presently available information from the operational wave monitoring for Hornsea Project One construction. The review also includes a consideration of the previous wave modelling undertaken as part of the Environmental Impact Assessment (EIA) for Hornsea Project One.

The aim of the review is to examine the operational wave monitoring for possible wind farm effects and relate this to the assumptions made in the EIA wave modelling to provide a means to justify the further applicability of these models to support the EIA of Hornsea Four.

Developing this technical note also serves to add to the evidence base by extending the baseline understanding for waves, substantiating the EIA methodology and assumptions, and establishing a dataset which helps quantify the effects of an offshore wind farm on waves, based on a monopile option.

5. Available information

This review considers both the EIA wave modelling supporting the Environmental Statement (ES) for Hornsea Project One and the operational wave monitoring from the initial phases of construction. The primary information includes:

- Annex 5.1.2 of Hornsea Project One EIA describing the wave modelling (SMart Wind, 2013)
- Wave monitoring from north and south operational wave buoys for the period 16 September 2017 to 19 November 2018
- Installation programme of foundations across Hornsea Project One up to 19 November 2018

6. EIA assumptions

The EIA for Hornsea Project One included an assessment of the potential impact on waves from the array of foundations (SMart Wind, 2013). The assessment was based on a set of conservative assumptions taken from the associated Design Envelope, in line with the Rochdale Envelope approach. Consequently, the EIA assessment represented the realistic worst case and assumed all other cases would provide a lesser level of impact. A brief review of the EIA wave modelling is provided so that the level of conservative assumptions can now be contrasted against the scheme being built, what this may imply to possible wave effects and whether this is discernible from the operational wave monitoring.

For the EIA wave modelling, the maximum effect on waves was related to the physical blockage of the array which was attributed to the greatest number of largest sized foundations with the smallest separation (i.e. the densest layout).

Accordingly, the wave modelling assumed 335 conical shaped gravity foundations with a base diameter of 50 m and a minimum spacing of 924 m (centre to centre) along rows. This configuration was referred to as (indicative) Layout 1 in the ES (Figure 2), representing a potential seabed footprint of 0.66 km^2 for all foundations across an array of 407 km^2, equivalent to a 0.162% take of the total seabed area.

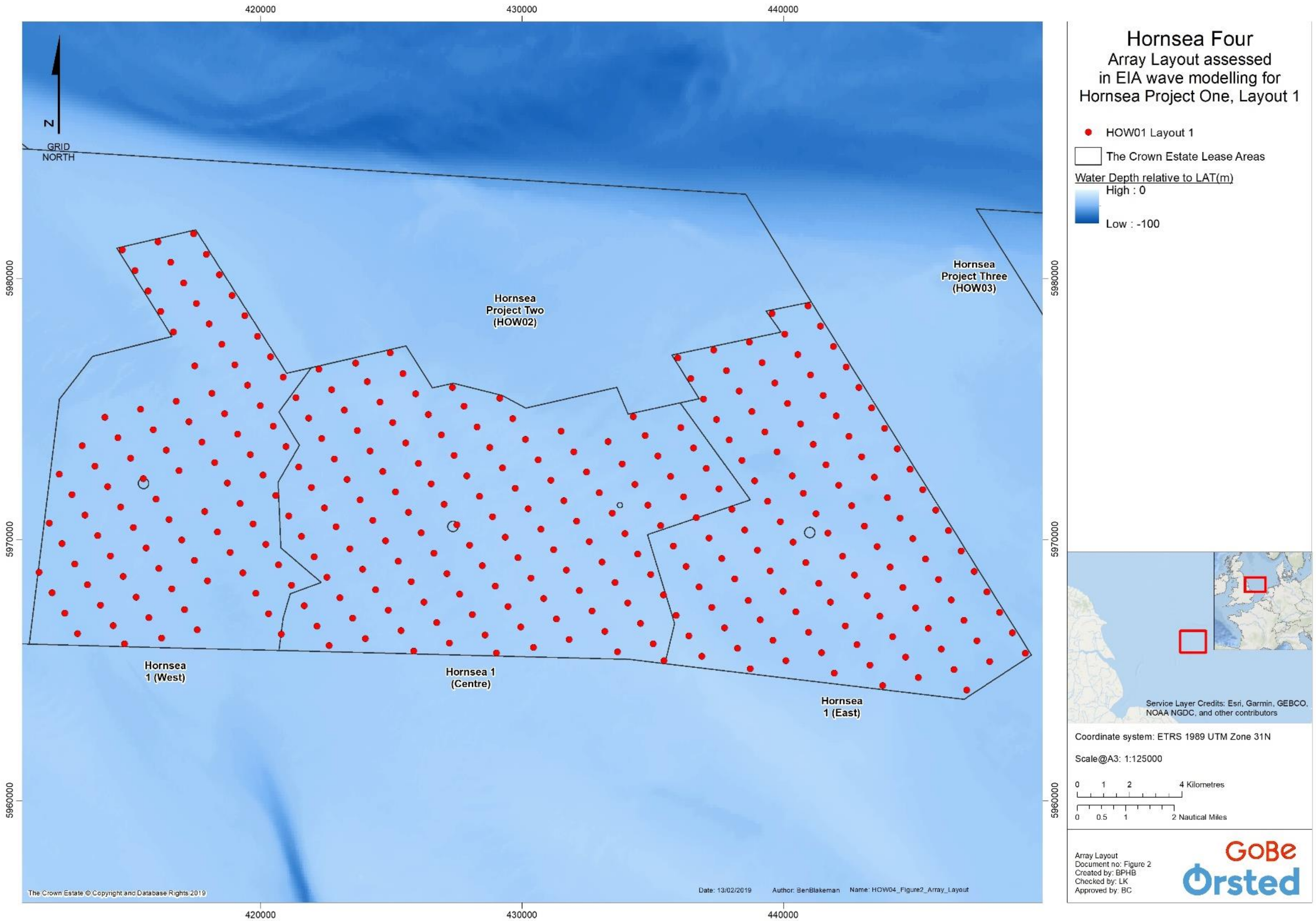


Figure 2: Array layout assessed in EIA wave modelling for Hornsea Project One, Layout 1

For simplicity, the conical shaped gravity base was considered as a cylinder with an effective diameter of 32.90 m to demonstrate the relationship between diameter (D) and wavelength (L), with the ratio D/L described as the diffraction parameter. The diffraction parameter is known as an important dimensionless variable related to the intensity of scattered waves (including reflected waves). For cases with D/L > 0.2, the structure is considered to be large (relative to the wavelength) and diffraction and wave scattering become important (Isaacson, 1979). For cases with D/L < 0.2, the structure is considered to be small and only drag forces (friction related) remain around the structure. Collectively, these are regarded as blocking type effects.

Wave modelling was undertaken for five wave directions (N, NNE, NE, ENE and E) with six return periods (50% non-exceedance, 0.1, 1, 10, 50 and 100 years). For all wave cases, the associated values of D/L for an individual foundation unit exceeded 0.2 (0.25 to 1.28, averaging 0.50 and derived from T_p), indicating the relevance of diffraction and wave scattering for the gravity base case.

The results of the wave modelling were quantified as a reduction in wave height (Hs). Site AWAC 2, central to the array, indicated reductions in wave height of between 0.12 to 0.31 m for the cases assessed (equivalent to between 2 to 24% of the baseline condition). The wave reductions generally dissipated within 50 to 60 km from the array without reaching the coastline, however, for the N 50% non-exceedance case a small reduction in wave height appeared to reach the coastline of North Norfolk (contour band 0.06 to 0.2 m wave reduction) (Figure 3).

By adopting the Rochdale Envelope approach, all alternative configurations proposed for Hornsea Project One, in terms of fewer and smaller foundation units and in a more widely spaced layout, could be considered as having a smaller scale of wave impact than those presented in the ES. This includes the proposed monopile option which assumed a maximum diameter of 8.5 m.

7. Scheme under construction

7.1 Final layout

The scheme now being constructed at Hornsea Project One comprises of 174 monopile foundations, each with a diameter of 8.1 m and a minimum spacing of around 1,500 m (centre to centre) along rows (Figure 4), although a closer spacing down to 930 m is evident along sections of the southern perimeter (sites A04 to A32).

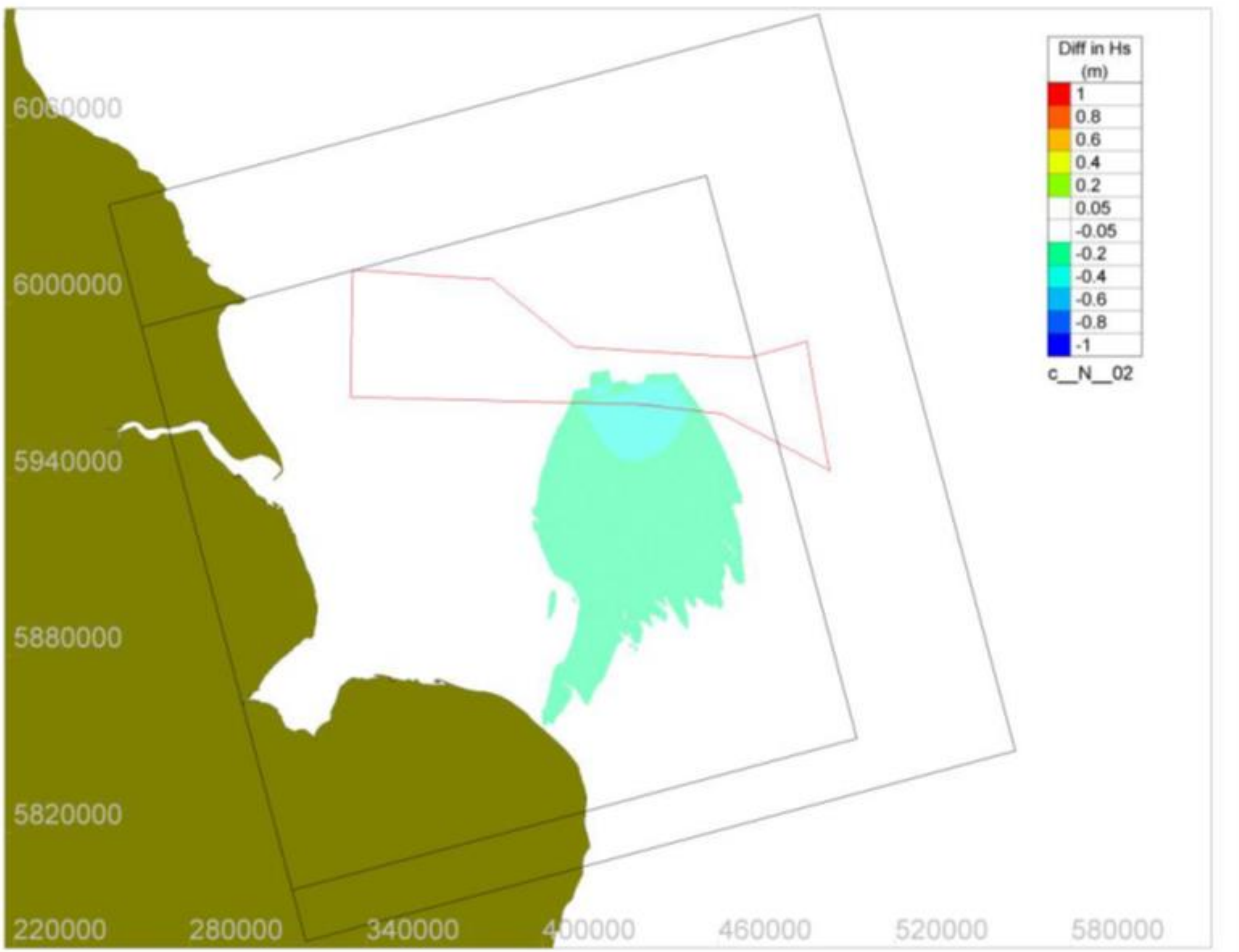


Figure 3: 50% non-exceedance case for N waves, showing extents of relative reduction in wave heights (from Smart Wind (2013))

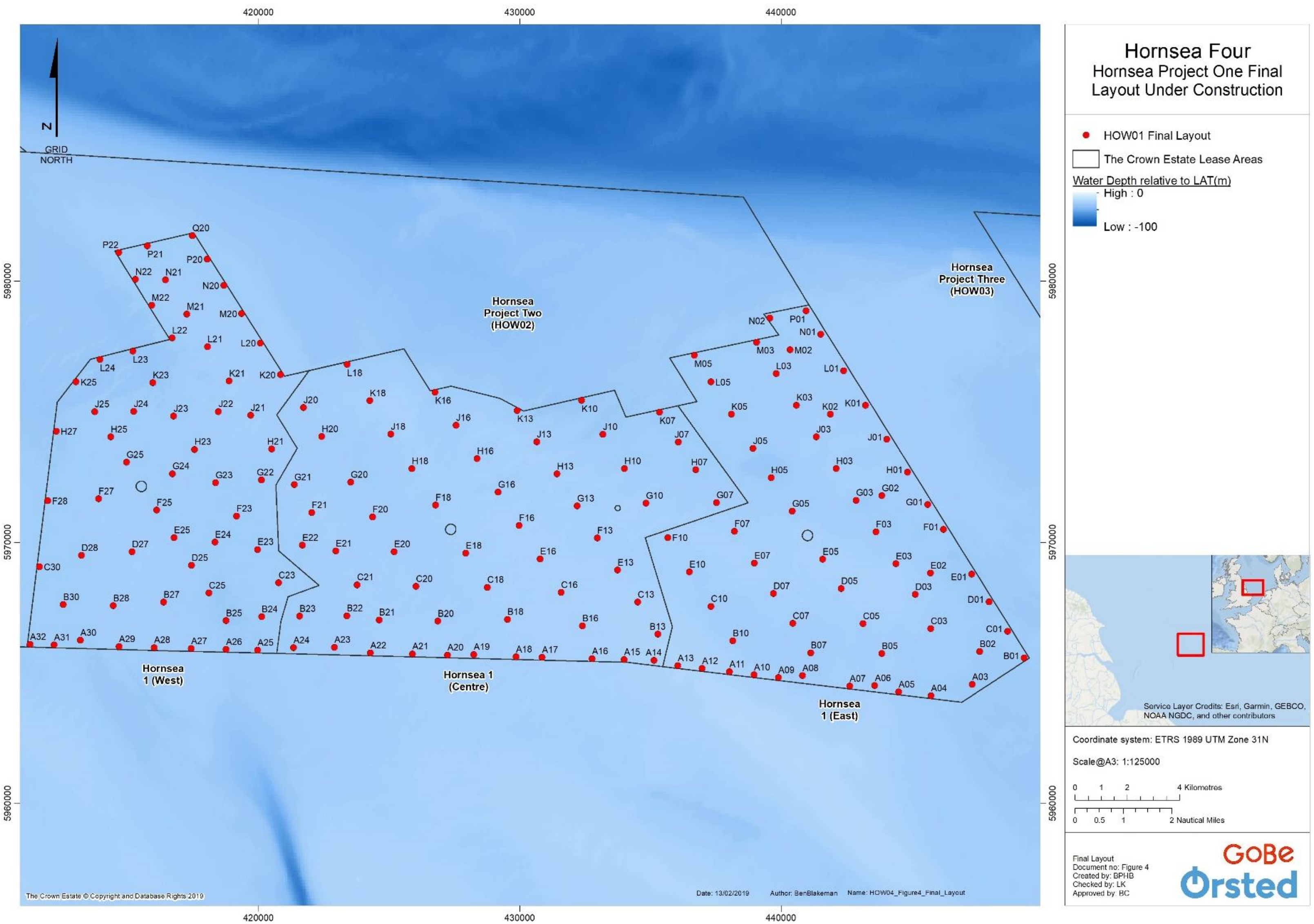


Figure 4: Final layout under construction for Hornsea Project One

This configuration represents a seabed footprint of 0.01 km^2 for all foundations across an array of 407 km^2, equivalent to a 0.002% take of the total area. Compared to the EIA case, Layout 1, the final layout under construction represents a scheme which is around 73 times less dense by area.

For this scale of foundation, the values of D/L for the equivalent EIA wave cases would be 0.04 to 0.21, averaging 0.08. This indicates that diffraction and wave scattering are far less relevant to the monopile case (i.e. the 8.1m diameter monopile can be considered small in relation to incident waves, i.e. slender pile), along with far fewer point sources (174 v 335) for any wave disturbance effects. For reference, only 50% non-exceedance waves from N, NNE and NE directions provide values of D/L > 0.2 and based on the assumption of wave periods (T_p) of 5 s.

7.2 Progress to date

The installation programme is advancing in phases. From 25 January to 14 April 2018 (Phase 1) a total of 60 foundations were installed across the mid-section of Hornsea Project One, Hornsea 1 (Centre), (Figure 5). In the period 15 April to 13 August 2018 there was an interval when no additional foundations were installed. From 14 August to 18 November (Phase 2) an additional 68 foundations were installed, mainly in the eastern block, Hornsea 1 (East), providing a total of 128 of the 174 planned foundations (Figure 6), representing 74% completion.

The remaining 46 foundation sites yet to be installed (as of 19 November 2018) are all associated with the westerly block, Hornsea 1 (West). This has relevance when considering directional wave effects as waves approaching from N through to E (comparable wave directions to the EIA assessment) would pass through areas already fully installed and any additional sites further to the west would make no contribution.

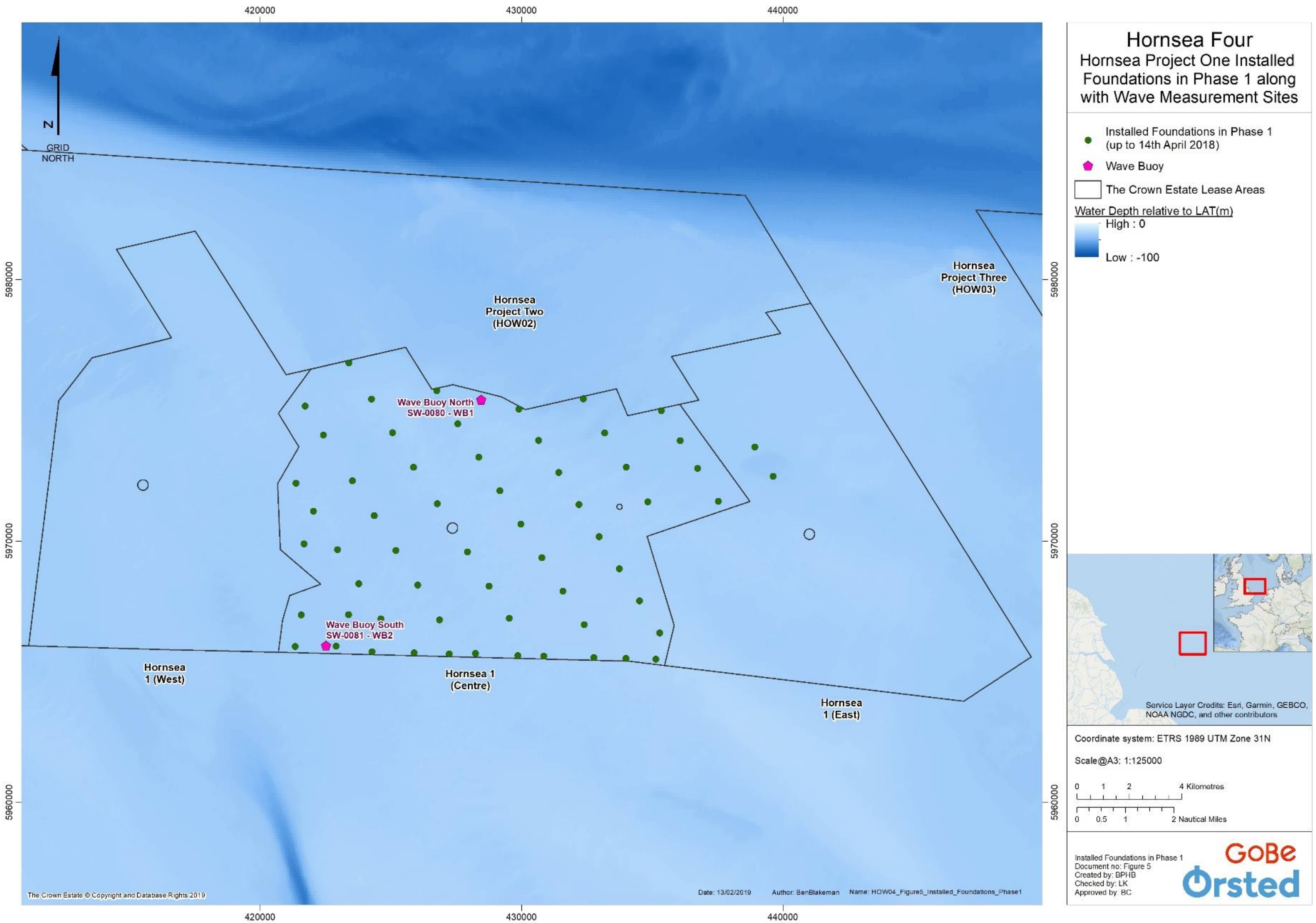


Figure 5: Installed foundations in Phase 1 (up to 14 April 2018), along with wave measurement sites; WB1 North and WB2 South

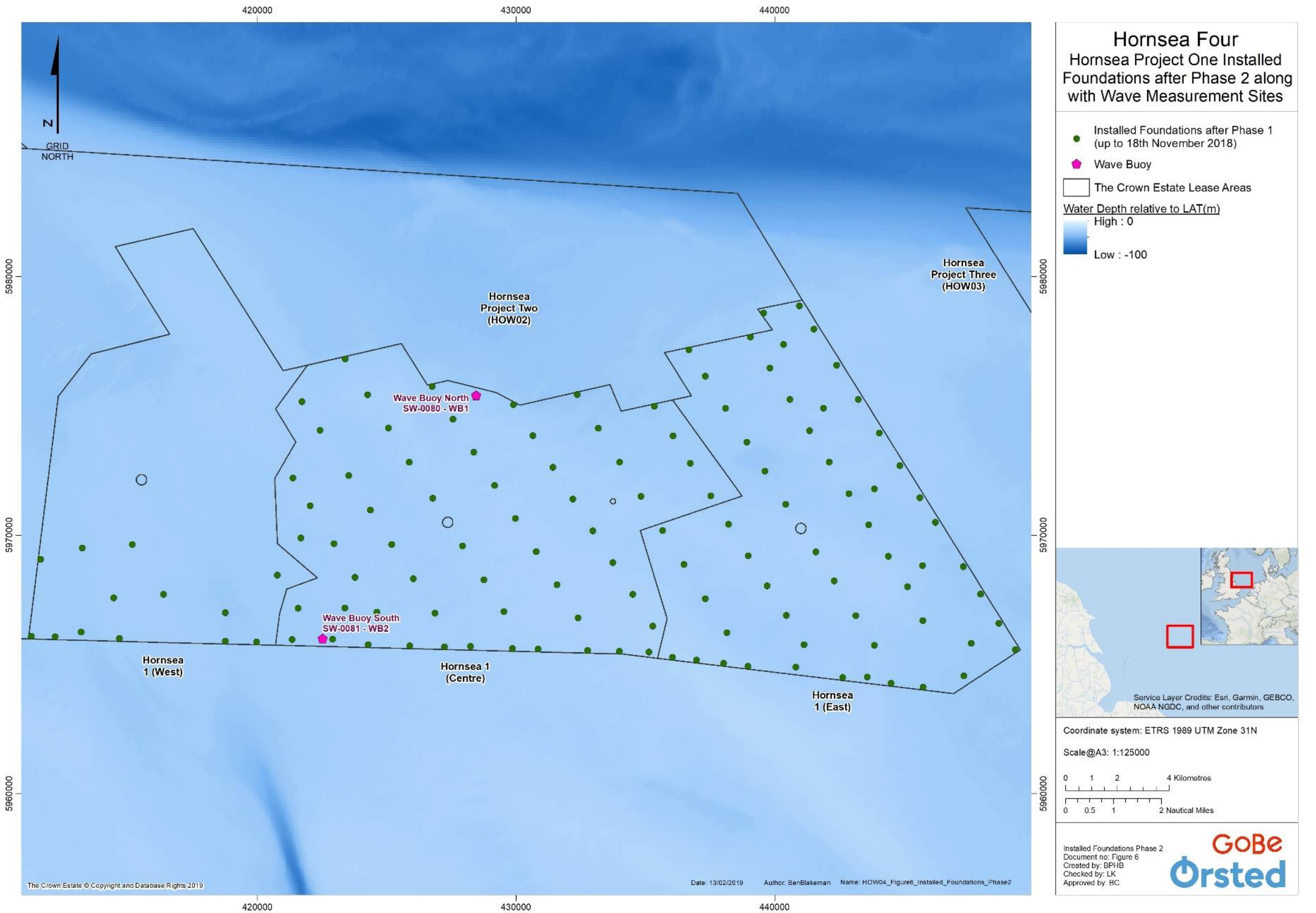


Figure 6: Installed foundations after Phase 2 (up to 18 November 2018)

8. Operational wave data

8.1 Measurement locations

The installation of foundations at Hornsea Project One has been supported by real-time wave data to assist operational planning. Wave data is provided from two sites (ongoing); WB1 is central to the northern perimeter at latitude 53.9220°N, longitude 1.9098°E, and WB2 on the southern perimeter at latitude 53.8368°N, longitude 1.8201°E, as shown in Figure 5 and Figure 6. The sites are approximately 11 km apart on a NNE to SSW orientation. WB2 is closest to foundation A23 which is due east by around 500 m, whereas WB1 is closest to J16 which is around 1200m to the south-west.

The relative position of these wave measurements presents the opportunity of considering possible changes in wave behaviour as (unaffected) waves move from northerly sectors to the south (or south to north) across the array and where each foundation along the trajectory of the wave may contribute a point source of scattering (i.e. reflection off the structures) and diffraction type interference. Arguably, some level of structure-type effects could be involved at both measurement sites for waves from easterly sectors (or westerly sectors), although the level of any such effects would be expected to be more prominent at WB2 because of the greater number and more closely spaced foundations (e.g. seven foundations for WB1 and 19 for WB2 at the end of Phase 2).

8.2 Measurement period

The operational wave data commenced on 26 September 2017 and provides standard measures for wave height (Hm0), period (T_{m02}) and direction, at an interval of 30 minutes.

The available wave data can be associated with different periods of site activity, as summarised in Table 1.

Table 1: Wave measurement periods relative to site activity

Period	From	To	Duration (days)	Site activity
Pre-construction / baseline	26 September 2017	24 January 2018	121	No foundation structures
Construction / Phase 1	25 January 2018	14 April 2018	80	First 60 foundations
Interval 1	15 April 2018	13 August 2018	120	No installations in period
Construction / Phase 2	14 August 2018	18 November 2018	96	68 foundations installed

For a fair basis of comparison, the pre-construction period can be considered against data from, and including, Interval 1, i.e. from 15 April to 19 November 2018. This omits Phase 1 of construction since this is potentially an evolving period of environmental change (rather than a stationary condition) between WB1 and WB2 when foundations were continuously being added. Phase 2 of construction involved foundation installations to the east which do not fall between WB1 and WB2, so are considered less relevant for any comparisons of northerly (or southerly) wave sectors.

8.3 Wave conditions: Pre-construction

Figure 7 presents a timeseries variation of wave height (H_{m0}), period (T_{m02}) and direction for the pre-construction period for both WB1 (north) and WB2 (south). In general, there is a very close similarity between both sites for all wave parameters. The data covers the period from 26 September 2017 to 24 January 2018, providing an elapsed period of 121 days.

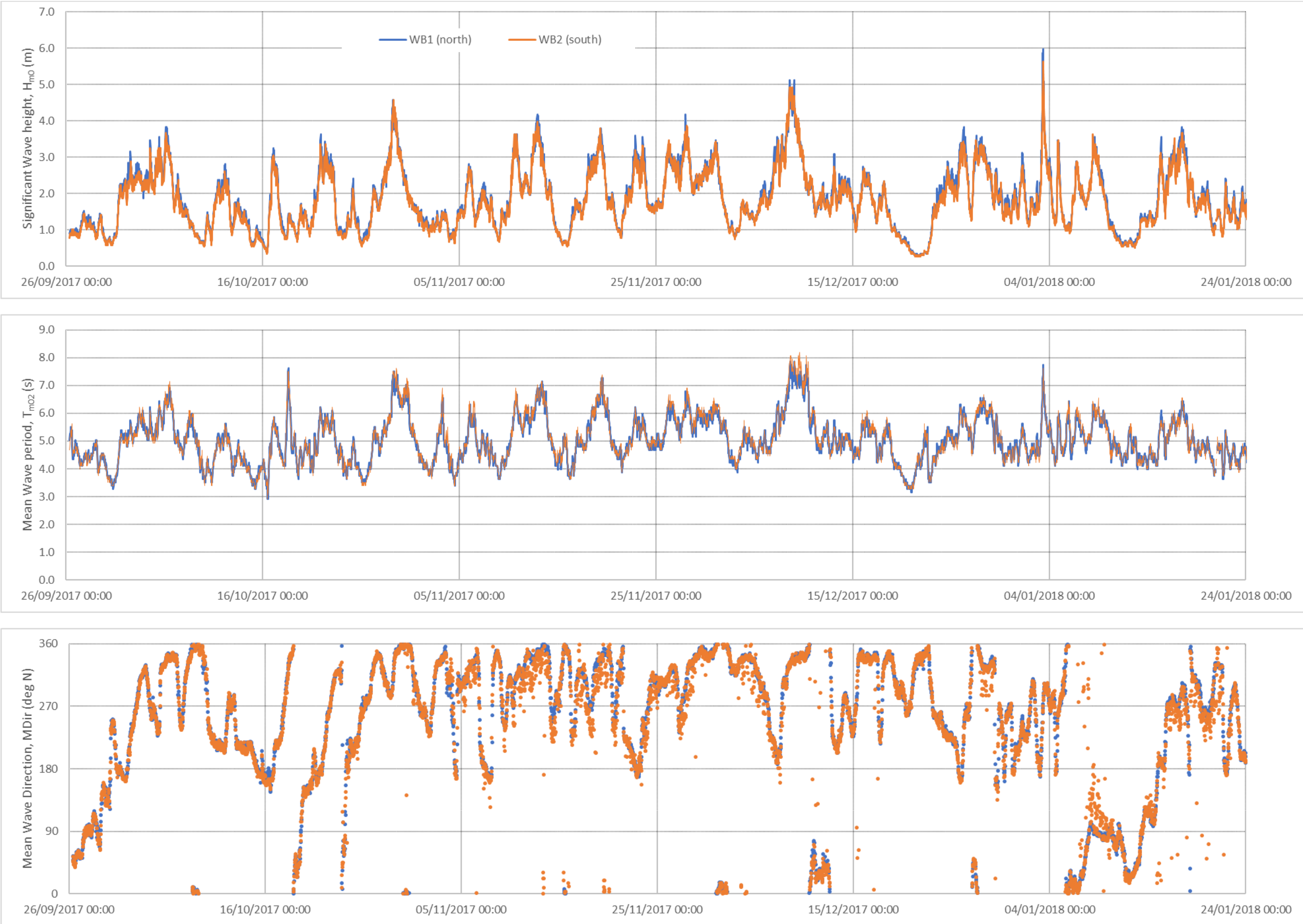


Figure 7: Timeseries of wave conditions for the pre-construction period at Hornsea Project One

The largest wave event recorded in this period occurred on 3 January 2018 at WB1, with a wave height of 5.98 m, wave period of 7.73 s (T_p of 9.14 s) and from 289.69°N (W – westerly sector).

Figure 8 presents the same information using a wave rose with 16 directional sectors, this choice of sub-division aligns with the wave directions applied in the EIA wave modelling. The wave rose identifies the most common wave directions in the pre-construction period were from NNW (326.25 to 348.75°N), representing nearly 17.5 % of all waves. This sector also contains most of the large waves with 1.2 % of all waves > 4.00 m.

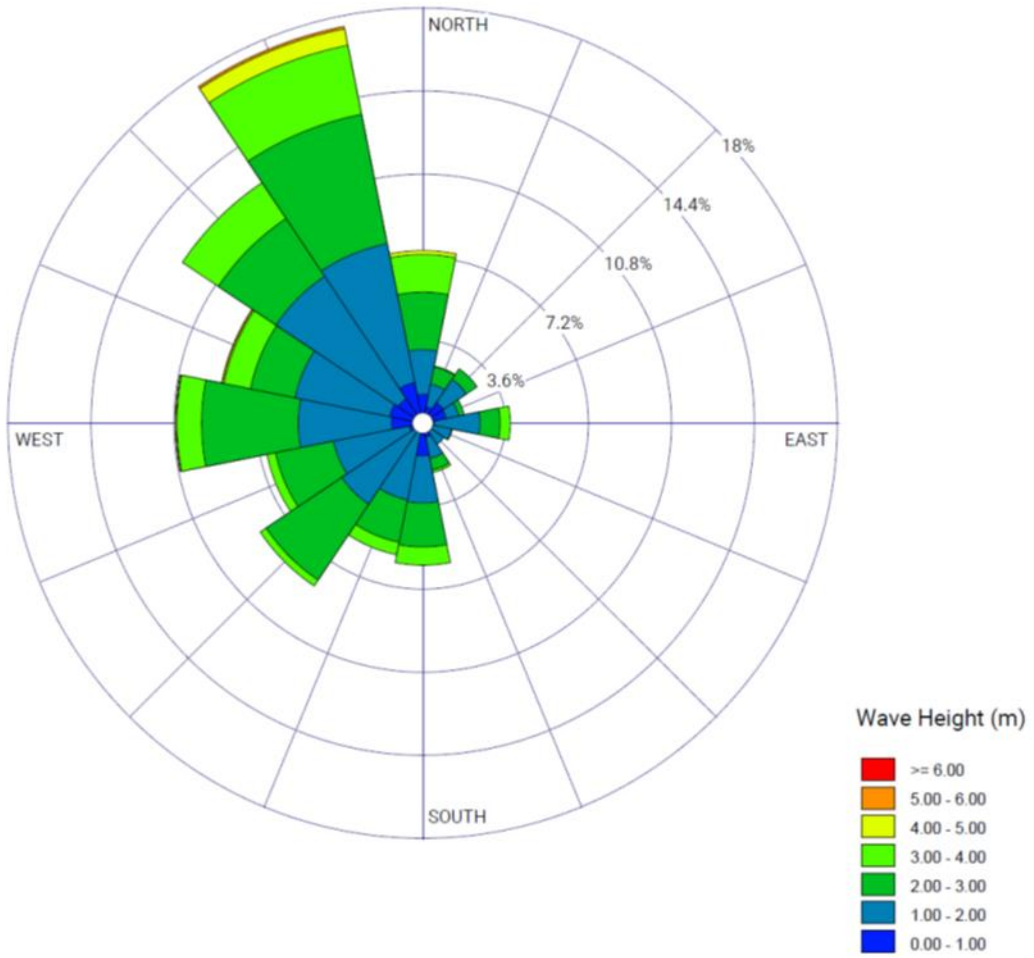


Figure 8: Wave rose for the pre-construction period at Hornsea Project One, based on WB1 measurements

8.4 Wave conditions: Phase 1 construction

Wave conditions occurring during the Phase 1 construction period are shown in Figure 9. The data covers the period from 25 January to 14 April 2018, providing an elapsed period of 80 days. The first monopile foundation (C21) was installed on 25 January 2018.

This period of observations appears to contain two large storm events, the first occurring on 1 to 2 March 2018 and the second on 17 to 18 March 2018, both events demonstrating a prolonged period of large waves (wave heights > 4 m and wave periods > 7 s) and both from the ENE (east-north-east) sector. The largest wave event recorded in this period occurred on 1 March 2018 (first storm period) at WB2, with a wave height of 6.33 m, wave period of 8.32 s (T_p of 11.02 s) and from 118.13 °N (ESE – east-south-east sector). There is some peculiar behaviour in (mean) wave directions at WB2 during these storms with frequent switching between directions similar to WB1 (around 75 to 90°N) and directions slightly further to the south (around 115 to 130°N). Wave heights and periods at WB2 are also slightly larger than conditions measured at WB1. These differences are not considered to be attributable to any wind farm related effects since there were very few foundations installed at these times (19) and none close to the wave measurement sites at locations in the 'upwind' direction. Furthermore, the equivalent wavelength of the storm waves was around 71 m, producing a ratio D/L of 0.11 for T_{m02} or 0.08 for T_p (i.e. values are < 0.2) meaning any possible diffraction related effects were most likely unimportant.

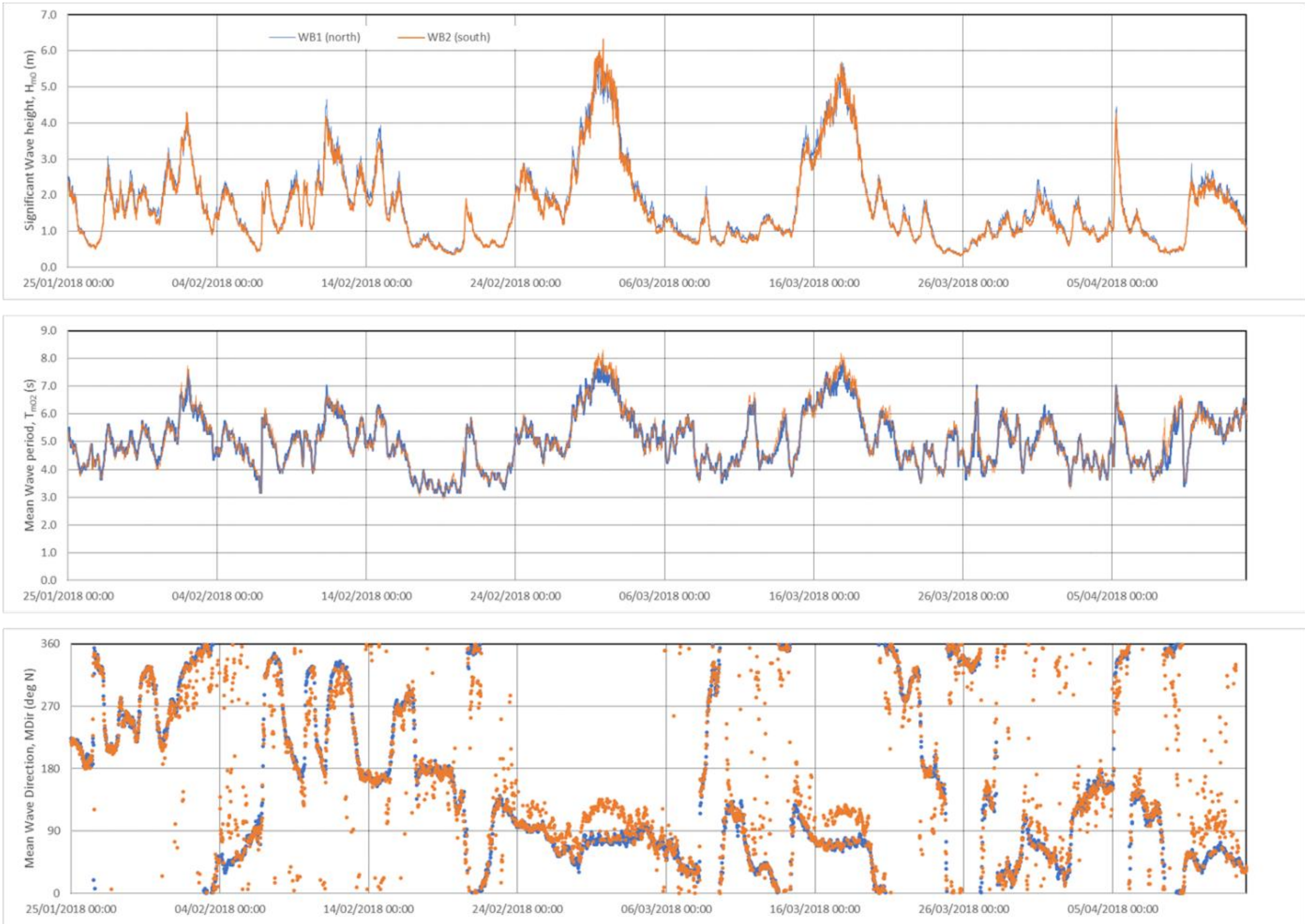


Figure 9: Timeseries of wave conditions for the Phase 1 construction period at Hornsea Project One

Figure 10 presents the same information using a wave rose with 16 directional sectors. The wave rose identifies the most common wave directions during Phase 1 which is from ENE (56.25 to 78.75°N), representing around 15% of all waves. The sector which contains most of the large waves with 4.4% of all waves > 4.00 m is also ENE, east-north-east, a feature which is largely attributable to the two storm events.

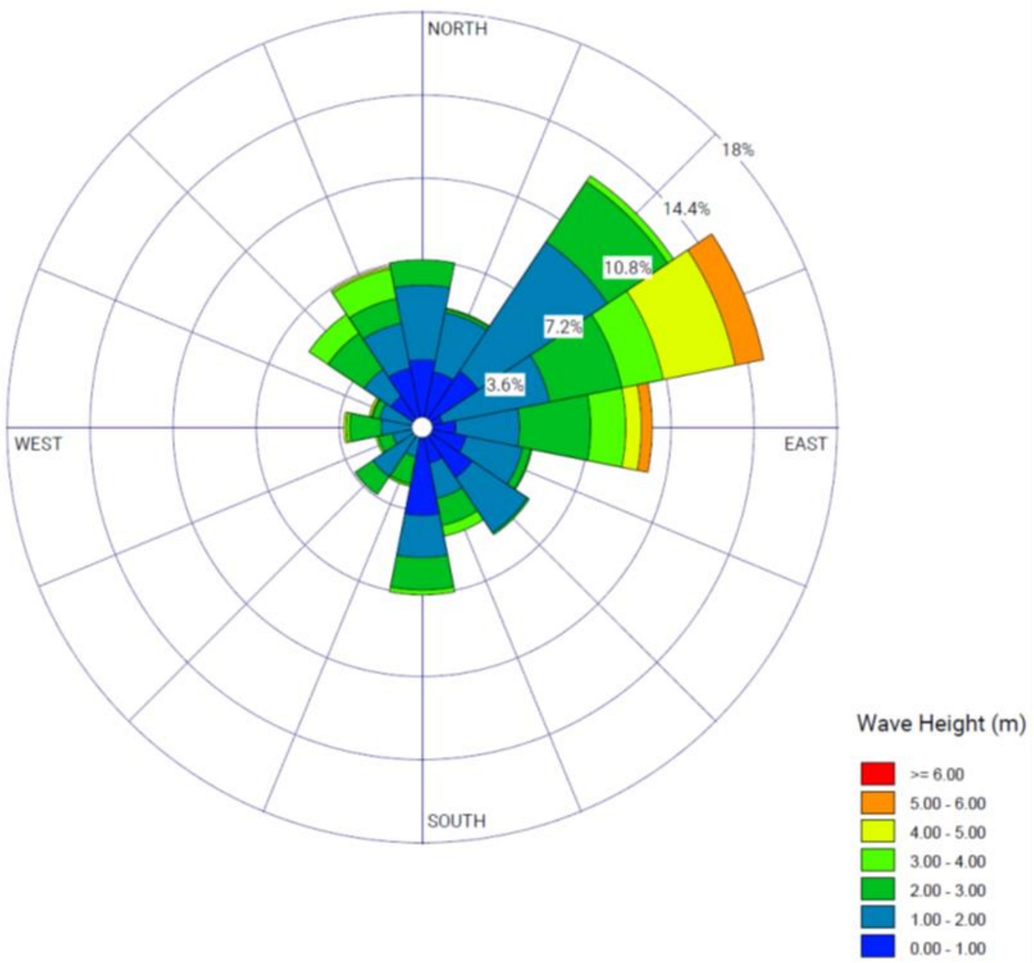


Figure 10: Wave rose for the Phase 1 construction period, based on WB1 measurements

8.5 Wave conditions: Post-Phase 1 construction

The wave events post-Phase 1 comprise an interval period when there were no further installations (15 April to 13 August 2018) and the second construction period, Phase 2, which installed a further 68 foundations, mainly in the easterly block (14 August to 18 November 2018). The final period of wave date extends to 20 November 2018, providing an elapsed period of 219 days from 15 April 2018. Wave conditions occurring in this period are shown in Figure 11.

The largest wave event recorded in this period occurred on 21 September 2018 at WB1, with a wave height of 4.41 m, wave period of 6.56 s (T_p of 7.62 s) and from 255.94°N (WSW – west-south-west sector).

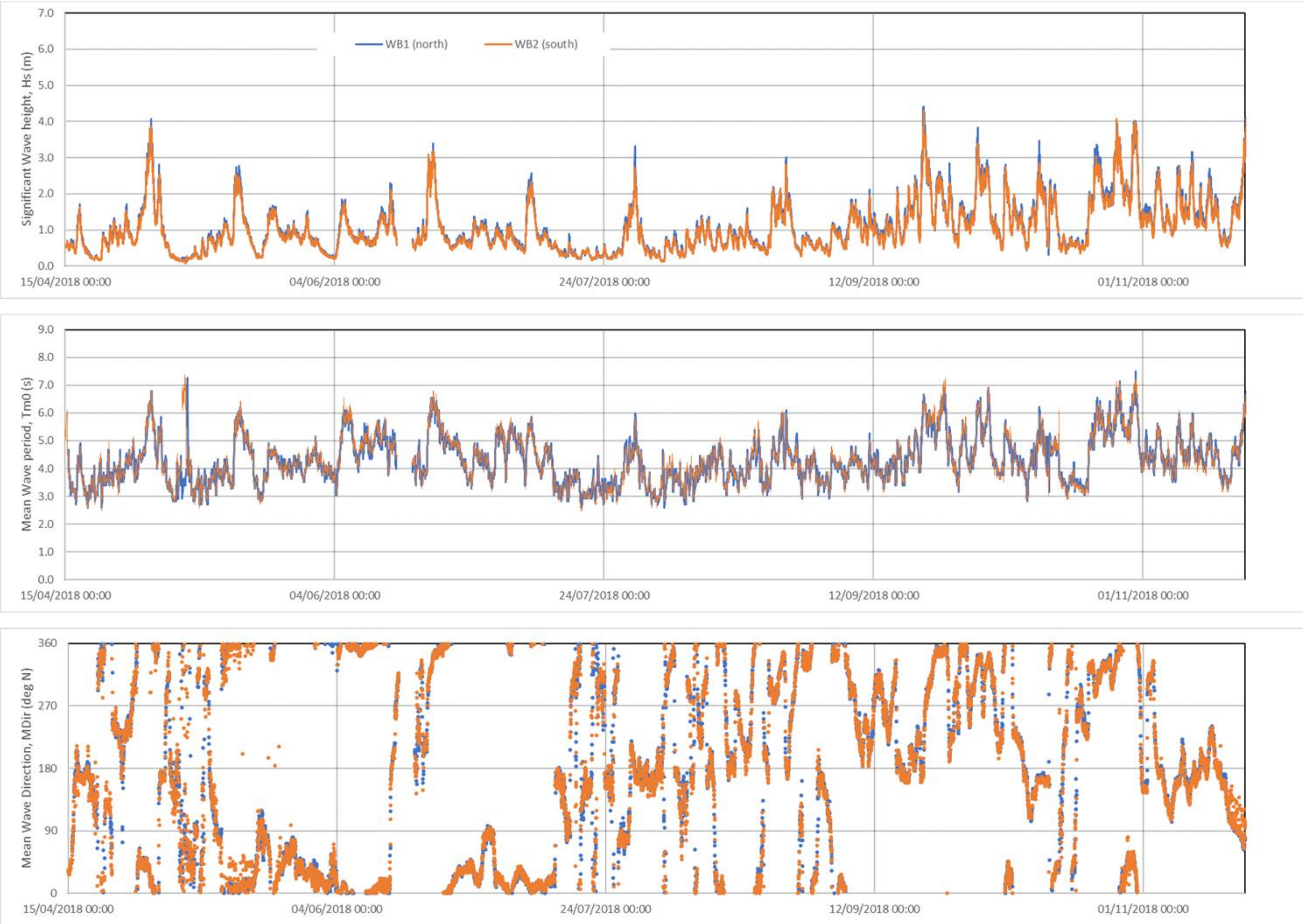


Figure 11: Timeseries of wave conditions for post-Phase 1 construction period at Hornsea Project One

Figure 12 presents the same information using a wave rose with 16 directional sectors. The wave rose identifies the most common wave directions during the post-Phase 1 period which is from N (348.75 to 11.25°N), representing nearly 14% of all waves. The sectors which contain most of the large waves with 0.4% of all waves > 4.00 m are ENE, east-north-east and W, west.

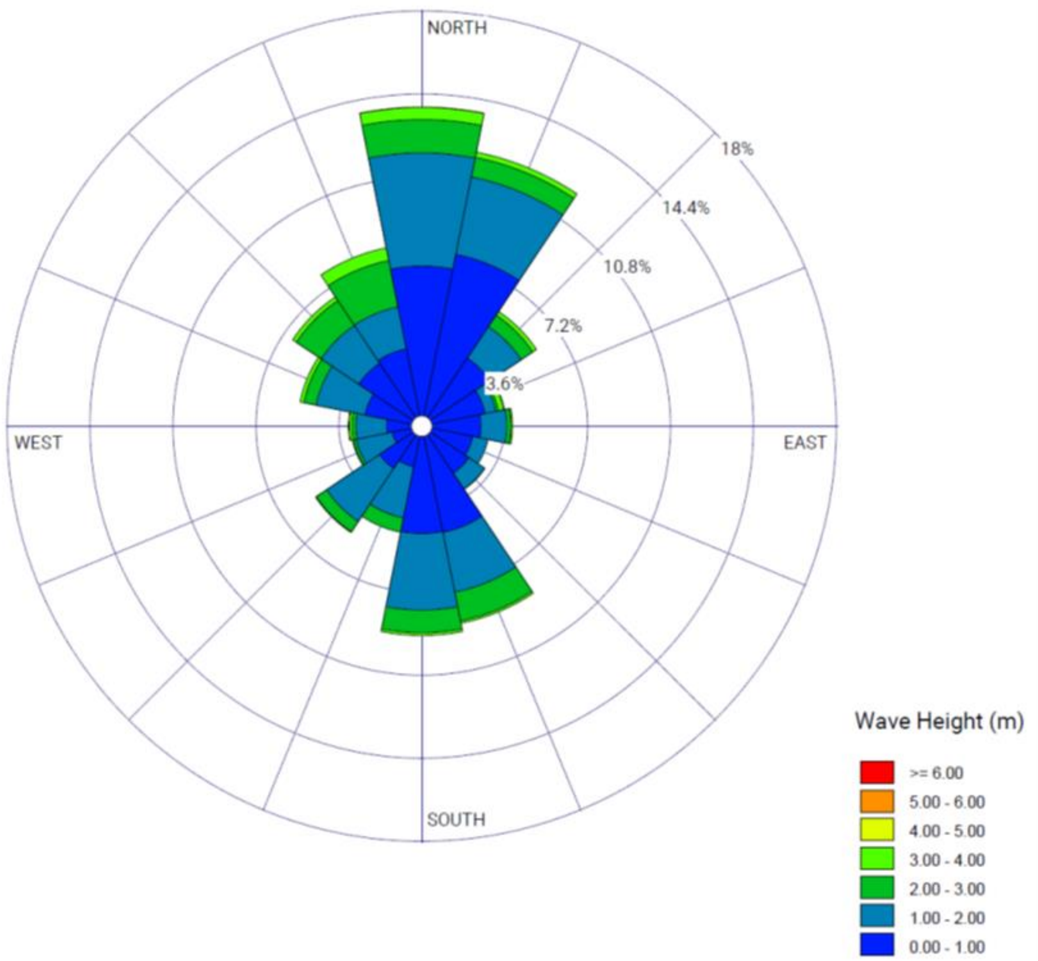


Figure 12: Wave rose for the post-Phase 1 construction period, based on WB1 measurements

8.6 Comparison of wave conditions

The available wave data has been associated with different periods of site activity across Hornsea Project One and reviewed for peak events and directional distribution.

Each period of site activity has a different duration, within and between these periods there are also marked differences in the types of wave conditions that have occurred.

Despite these differences, there remains an overall close similarity between wave conditions at WB1 and WB2, noting that these sites are only 11 km apart, in open water of similar depth and with no major notable influences from shallow water bathymetry features likely to cause refraction, either between or around the immediate area. A reasonable premise can therefore be made that the course of wave energy moving across the site is not diminished or modified in the pre-construction period and during the post-construction period the addition of foundations is the only extra influence which has the potential to modify the transmission of wave energy, either by reflection / scattering or diffraction related effects. A further necessary assumption in the comparison of waves is that all conditions can be considered as a uni-modal sea-state, i.e. all wave energy can be assigned to a mean wave direction and bi-modal sea-states with a split of wind-wave and swell waves from different directions are irrelevant. Finally, no account is made of the time taken for any individual wave to pass between WB1 and WB2, rather the burst period is assumed to provide sufficient overlap that means each site is sampling similar conditions.

Quantifying similarities between the various periods of site activity is based on a statistical approach using a range of similar directional conditions since "like-for-like" events have not occurred in each period. Where the level of similarity has changed between the pre-construction and post-construction period this may be due to various reasons, one being effects due to the installed foundations, others may be different sample sizes and statistical distributions. Attributing very small differences in levels of similarity (i.e. similar magnitude as statistical "noise") to a potential cause remains a practical limitation and only larger scale effect, such as those determined by the EIA modelling, is expected to be detectable and attributable.

A linear correlation between wave heights at WB1 and WB2 is used to determine similarity and any net bias. This correlation is performed for 16 directional sectors of 22.5° width, consistent with the wave rose presentation. These sectors also match with the wave directions considered in the EIA wave modelling. Table 2 summaries the detail of these wave sectors.

Table 2: Wave sectors (values in °N)

Directional sector	Centre value	Lower	Upper
N	0.00	348.75	11.25
NNE	22.50	11.25	33.75
NE	45.00	33.75	56.25
ENE	67.50	56.25	78.75
E	90.00	78.75	101.25
ESE	112.50	101.25	123.75
SE	135.00	123.75	146.25
SSE	157.50	146.25	168.75
S	180.00	168.75	191.25
SSW	202.50	191.25	213.75
SW	225.00	213.75	236.25
WSW	247.50	236.25	258.75
W	270.00	258.75	281.25
WNW	292.50	281.25	303.75
NW	315.00	303.75	326.25
NNW	337.50	326.25	348.75

Waves from northerly sectors (NNW through N to ENE) are expected to represent 'undisturbed' conditions at WB1 for both pre- and post-construction conditions. For post-construction, waves would pass through the wind farm before arriving at the southern boundary with any accumulated foundation-related wave effects. WB2 is used to represent conditions on the southern boundary, although only NNE and SSW cases are directly aligned with WB1.

Similarly, waves from southerly sectors (ESE through S to WSW) are expected to represent 'undisturbed' conditions at WB2 heading into the wind farm before arriving at the northern boundary.

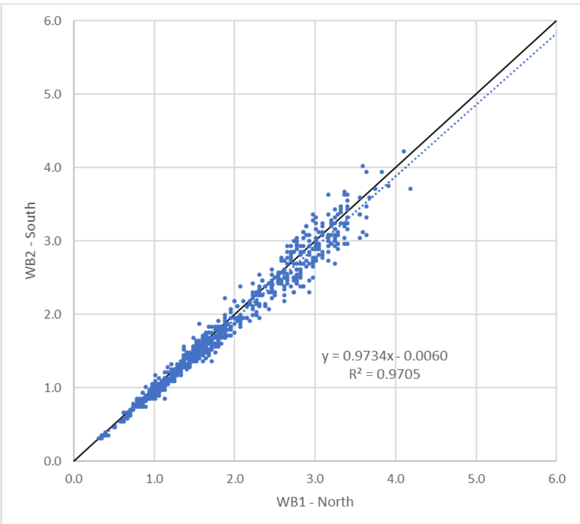
Westerly and easterly waves are expected to pass multiple foundations to reach both WB1 and WB2 in the post-construction case.

Waves approaching WB2 from the NW are considered as a unique case and are expected to be unaffected by foundations in the post-construction period since the western block where waves would have to pass through to reach WB2 is essentially incomplete at the time of developing this note. This case provides the opportunity of establishing a measure of statistical variance between the pre- and post-construction periods.

a. Comparison of NW waves

The comparison of NW waves based on the linear correlation between WB1 and WB2 is presented first to determine the statistical variance between pre- and post-construction periods which is not attributable to any wind farm related effects (Figure 13).

Pre-construction **Post-construction**



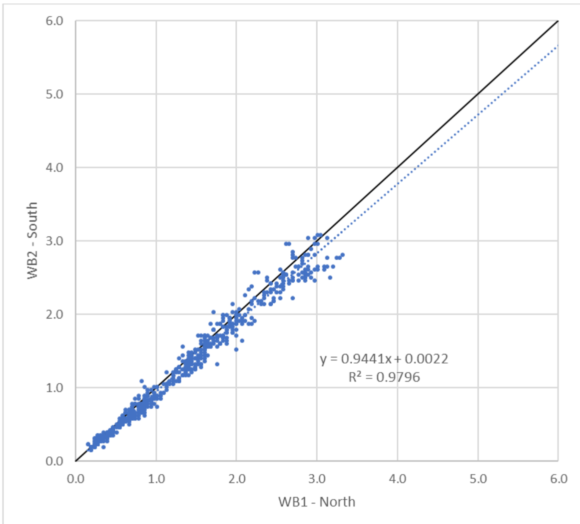


Figure 13: NW correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 97 to 98% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 683 events and spread of wave heights up to 4 m. The post-construction period consists of 638 events and wave heights up to around 3 m.

Wave heights at WB1 are marginally larger than equivalent events at WB2 (dotted trendline is towards WB1 from the 1:1 ratio – black line).

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB2 (Table 3).

Table 3: NW wave heights (in metres) based on statistical trendline relationships

WB1	WB2 pre-construction	WB2 post-construction	Difference at WB2
1.00	0.97	0.95	-0.02
2.00	1.94	1.89	-0.05
3.00	2.91	2.83	-0.08
4.00	3.89	3.78	-0.11

The difference in wave heights at WB2 between the two periods is considered as the level of statistical variance inherent in wave conditions. Importantly, this variance cannot be attributable to any wind farm influence, since NW waves would arrive at WB2 without passing any installed foundations. The statistical variance is related to the number and spread of wave heights in each sample and may only be partially resolved in this case, hence this scale of variance should be considered as an indicator only.

Although this non-wind farm related variance can only be resolved from NW waves, the same level of variance is likely to remain for all other wave directions between pre- and post-construction periods. Any wind farm related effects from other wave directions, where waves pass through the wind farm, should only be considered at scales above this variance, else they should be considered too small to be measurable.

b. Comparison of NNW waves

NNW waves have the potential to pass through an area occupied by a small number of installed foundations in the post-construction period before reaching WB2. The associated linear correlations between WB1 and WB2 are presented in Figure 14.

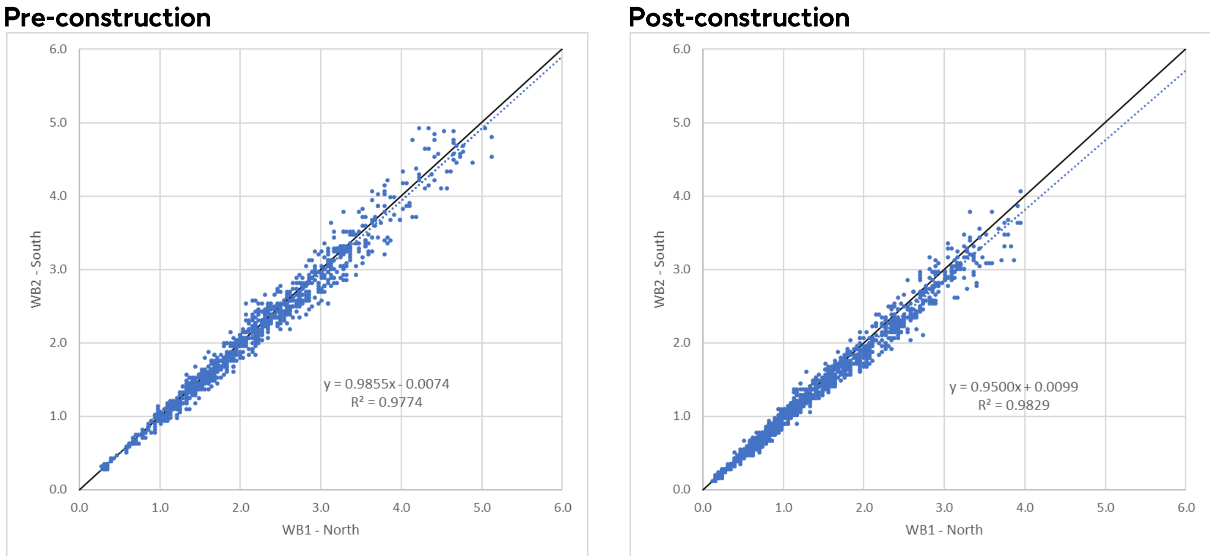


Figure 14: NNW correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) again show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 98% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 1020 events and spread of wave heights up to 5 m. The post-construction period consists of 1430 events and wave heights up to around 4 m. This is the most-common wave direction from the entire measurement period.

Wave heights at WB1 are marginally larger than equivalent events at WB2 (dotted trendline is towards WB1 from the 1:1 ratio – black line).

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB2 (Table 4).

Table 4: NNW wave heights (in metres) based on statistical trendline relationships

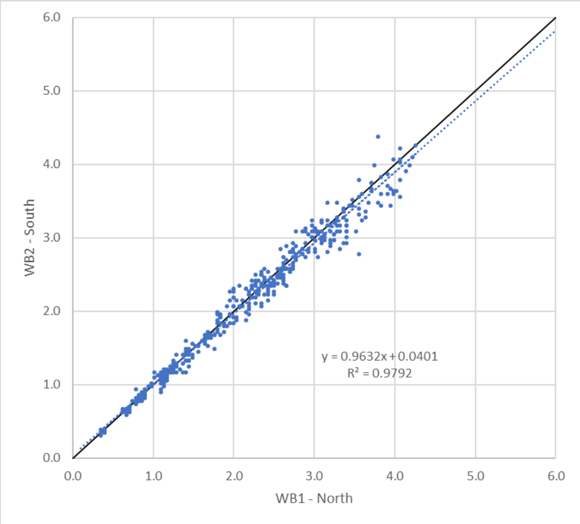
WB1	WB2 pre-construction	WB2 post-construction	Difference at WB2
1.00	0.98	0.96	-0.02
2.00	1.96	1.91	-0.05
3.00	2.95	2.86	-0.09
4.00	3.93	3.81	-0.12

The difference in wave heights at WB2 between the two periods is considered very similar to NW. There is no discernible scale of effect which can be attributed to the wind farm.

c. Comparison of N waves

N waves have the potential to pass through an area occupied by a small number of installed foundations in the post-construction period before reaching WB2. The associated linear correlations between WB1 and WB2 are presented in Figure 15.

Pre-construction



Post-construction

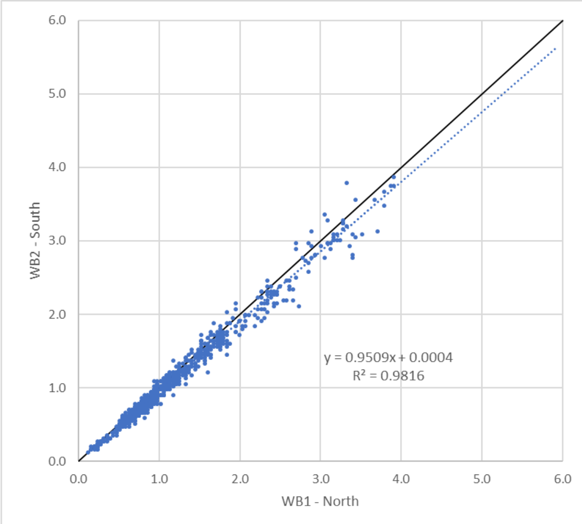


Figure 15: N correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) again show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 98% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 417 events and spread of wave heights up to 4 m. The post-construction period consists of 804 events and wave heights also up to around 4 m.

Wave heights at WB1 are marginally larger than equivalent events at WB2 (dotted trendline is towards WB1 from the 1:1 ratio – black line).

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB2 (Table 5).

Table 5: N wave heights (in metres) based on statistical trendline relationships

WB1	WB2 pre-construction	WB2 post-construction	Difference at WB2
1.00	1.00	0.95	-0.05
2.00	1.97	1.90	-0.06
3.00	2.93	2.85	-0.08
4.00	3.89	3.80	-0.09

The difference in wave heights at WB2 between the two periods is considered very similar to NW. There is no discernible scale of effect which can be attributed to the wind farm.

Northerly waves were also a consideration in the EIA wave modelling. Changes in wave height were quantified at a site central to the array at AWAC 2. Table 6 presents the predicted waves heights for a baseline case, for Layout 1 and for the difference attributed to the wind farm. Negative values signify a reduction in wave heights.

Table 6: Predicted wave heights at AWAC 2 for northerly waves (N), wave heights (in metres)

Scenario	Baseline	Layout 1	Difference
50% no exc	0.82	0.62	-0.20
0.1 yr RP	2.47	2.20	-0.27
1 yr RP	4.24	3.93	-0.31
10 yr RP	6.29	5.99	-0.30
50 yr RP	7.57	7.30	-0.27
100 yr RP	8.08	7.83	-0.25

Whilst AWAC 2 is a site central to the array, the level of change once waves have passed through the full width of the array would be expected to be even greater. The full level of impact from the EIA modelling at the down-wind side of the array is therefore expected to be around an order of magnitude greater than that shown from the wave measurements based on statistical variance. This effect can be seen on Figure 3 where the wave reductions on the southern boundary reach a level of around -0.4 m. Notably, the biggest reduction is associated with the 1-year return period event rather than for larger wave events. This is explained by larger waves having longer wave-lengths which reduce the ratio of D/L.

d. Comparison of NNE waves

NNE waves have the potential to pass through an area occupied by a small number of installed foundations in the post-construction period before reaching WB2. This is also the direction with closest alignment between WB1 and WB2. The associated linear correlations between WB1 and WB2 are presented in Figure 16.

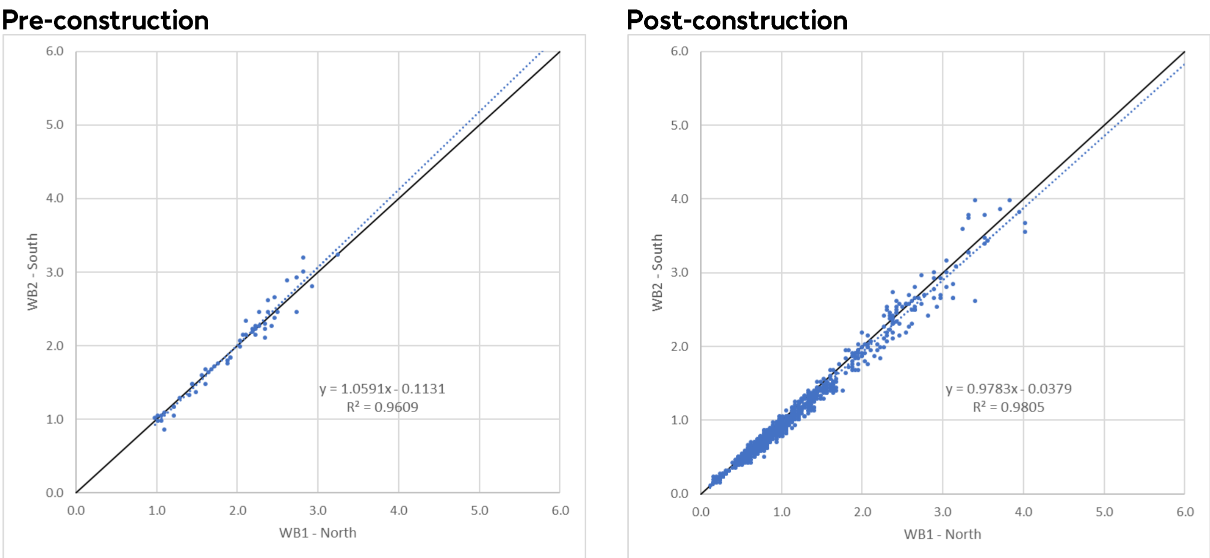


Figure 16: NNE correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) again show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 96 to 98% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 54 events and spread of wave heights up to around 3 m. The post-construction period consists of 1268 events and wave heights up to around 4 m.

Wave heights at WB1 are marginally larger than equivalent events at WB2 for the post-construction period (dotted trendline is towards WB1 from the 1:1 ratio – black line), however, the pre-construction period suggests waves heights at WB2 are marginally larger. This is a potential statistical anomaly due to the relatively small sample of pre-construction measurements which fall in this sector, suggesting the statistical relationship may not be fully resolved.

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB2 (Table 7).

Table 7: NNE wave heights (in metres) based on statistical trendline relationships

WB1	WB2 pre-construction	WB2 post-construction	Difference at WB2
1.00	0.95	0.94	-0.01
2.00	2.01	1.92	-0.09
3.00	3.06	2.90	-0.17
4.00	4.12 extrapolated	3.88	-0.25

The difference in wave heights at WB2 between the two periods is considered to be influenced mainly by the small pre-construction sample limiting a fully resolved statistical relationship. Furthermore, the wave conditions at WB1 and WB2 did not reach 4 m in the pre-construction period so the derived value at WB2 is extrapolated beyond the sample range. This is a limitation in the adopted method. There is no discernible scale of effect which can be attributed to the wind farm.

North-north-easterly waves were also a consideration in the EIA wave modelling. Changes in wave height were quantified at a site central to the array at AWAC 2. Table 8 presents the predicted waves heights for a baseline case, for Layout 1 and for the difference attributed to the wind farm. Negative values signify a reduction in wave heights.

Table 8: Predicted wave heights at AWAC 2 for north-north-easterly waves (NNE), wave heights (in metres)

Scenario	Baseline	Layout 1	Difference
50% no exc	0.83	0.68	-0.15
0.1 yr RP	2.08	1.90	-0.18
1 yr RP	4.50	4.27	-0.23
10 yr RP	6.97	6.73	-0.24

Scenario	Baseline	Layout 1	Difference
50 yr RP	8.35	8.14	-0.21
100 yr RP	8.84	8.66	-0.18

Whilst AWAC 2 is a site central to the array, the level of change once waves have passed through the full width of the array would be expected to be even greater.

e. Comparison of NE waves

NE waves have the potential to pass through an area occupied by a small number of installed foundations in the post-construction period before reaching WB2. The associated linear correlations between WB1 and WB2 are presented in Figure 17.

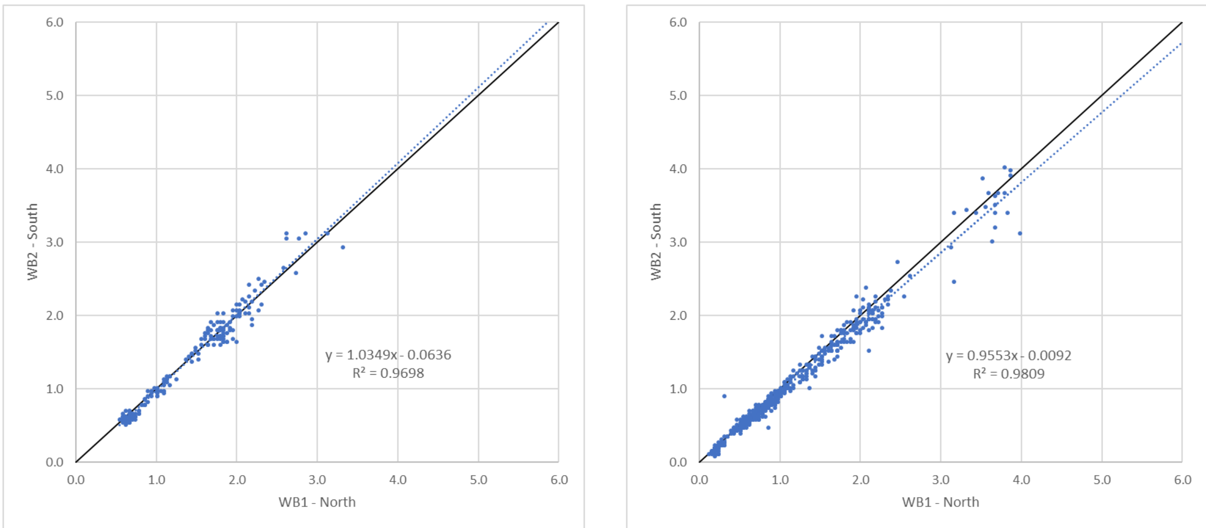


Figure 17: NE correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) again show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 97 to 98% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 194 events and spread of wave heights up to around 3 m. The post-construction period consists of 581 events and wave heights up to around 4 m.

Wave heights at WB1 are marginally larger than equivalent events at WB2 for the post-construction period (dotted trendline is towards WB1 from the 1:1 ratio – black line), however, the pre-construction period suggests waves heights at WB2 are be marginally larger. This is a potential statistical anomaly due to the relatively small sample of pre-construction measurements which fall in this sector, suggesting the statistical relationship may not be fully resolved.

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB2 (Table 9).

Table 9: NE wave heights (in metres) based on statistical trendline relationships

WB1	WB2 pre-construction	WB2 post-construction	Difference at WB2
1.00	0.97	0.95	-0.03
2.00	2.01	1.90	-0.10
3.00	3.04	2.86	-0.18
4.00	4.08 extrapolated	3.81	-0.26

The difference in wave heights at WB2 between the two periods is considered to be influenced mainly by the relatively small pre-construction sample limiting a fully resolved statistical relationship. Furthermore, the wave conditions at WB1 and WB2 did not reach 4 m in the pre-construction period so the derived value at WB2 is extrapolated beyond the sample range. This is a limitation in the adopted method. There is no discernible scale of effect which can be attributed to the wind farm.

North-easterly waves were also a consideration in the EIA wave modelling. Changes in wave height were quantified at a site central to the array at AWAC 2. Table 10 presents the predicted waves heights for a baseline case, for Layout 1 and for the difference attributed to the wind farm. Negative values signify a reduction in wave heights.

Table 10: Predicted wave heights at AWAC 2 for north-easterly waves (NE), wave heights (in metres)

Scenario	Baseline	Layout 1	Difference
50% no exc	0.82	0.70	-0.12
0.1 yr RP	2.50	2.33	-0.17
1 yr RP	4.70	4.49	-0.21
10 yr RP	6.52	6.31	-0.21
50 yr RP	7.53	7.34	-0.19
100 yr RP	7.88	7.69	-0.19

Whilst AWAC 2 is a site central to the array, the level of change once waves have passed through the full width of the array would be expected to be even greater.

f. Comparison of ENE waves

ENE waves have the potential to pass through an area occupied by a moderate number of installed foundations in the post-construction period before reaching WB2. The associated linear correlations between WB1 and WB2 are presented in Figure 18.

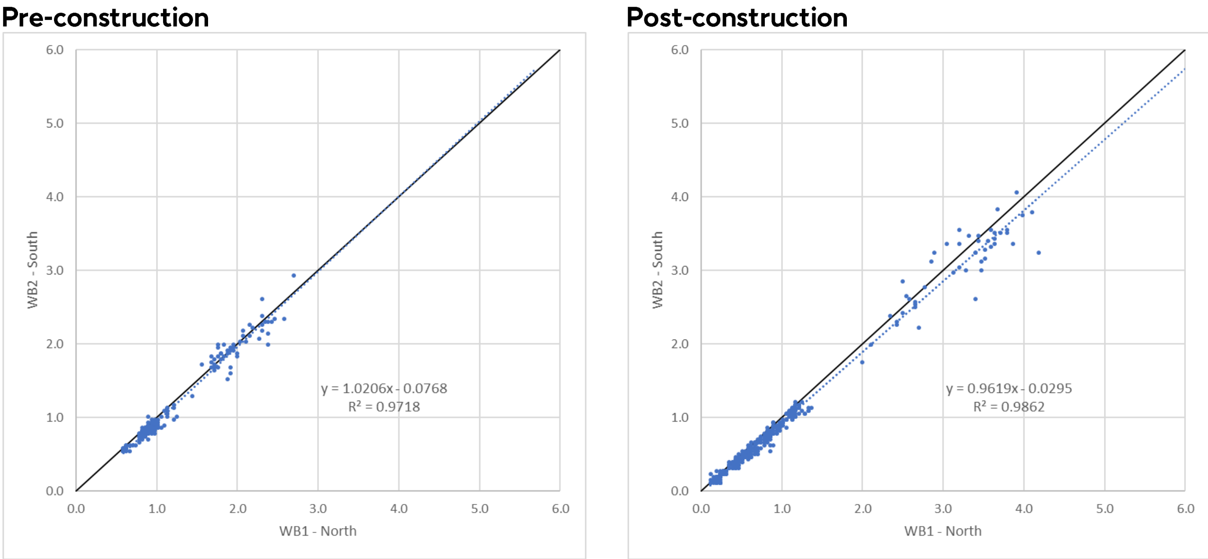


Figure 18: ENE correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) again show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 97 to 99% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 189 events and spread of wave heights up to around 3 m. The post-construction period consists of 384 events and wave heights up to around 4 m.

Wave heights at WB1 are marginally larger than equivalent events at WB2 for the post-construction period (dotted trendline is towards WB1 from the 1:1 ratio – black line), however, the pre-construction period suggests waves heights at WB2 are marginally larger. This is a potential statistical anomaly due to the relatively small sample of pre-construction measurements which fall in this sector, suggesting the statistical relationship may not be fully resolved.

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB2 (Table 11).

Table 11: NE wave heights (in metres) based on statistical trendline relationships

WB1	WB2 pre-construction	WB2 post-construction	Difference at WB2
1.00	0.94	0.93	-0.01
2.00	1.96	1.89	-0.07
3.00	2.98 extrapolated	2.86	-0.13
4.00	4.01 extrapolated	3.82	-0.19

The difference in wave heights at WB2 between the two periods is considered to be influenced mainly by the relatively small pre-construction sample limiting a fully resolved statistical relationship. Furthermore, the wave conditions at WB1 and WB2 did not reach 3 or 4 m in the pre-construction period so the derived values at WB2 are extrapolated beyond the sample range. This is a limitation in the adopted method. There is no discernible scale of effect which can be attributed to the wind farm.

East-north-easterly waves were also a consideration in the EIA wave modelling. Changes in wave height were quantified at a site central to the array at AWAC 2. Table 12 presents the predicted waves heights for a baseline case, for Layout 1 and for the difference attributed to the wind farm. Negative values signify a reduction in wave heights.

Table 12: Predicted wave heights at AWAC 2 for east-north-easterly waves (ENE), wave heights (in metres)

Scenario	Baseline	Layout 1	Difference
50% no exc	0.94	0.78	-0.16
0.1 yr RP	2.45	2.25	-0.20
1 yr RP	4.33	4.11	-0.22
10 yr RP	5.81	5.61	-0.20
50 yr RP	6.62	6.44	-0.18
100 yr RP	6.91	6.74	-0.17

Whilst AWAC 2 is a site central to the array, the level of change once waves have passed through the full width of the array would be expected to be even greater.

g. Comparison of E waves

E waves have the potential to pass through an area occupied by the largest number of most closely spaced foundations prior to reaching WB2 (estimated at around 20). Waves will also be passing through the top of the array to reach WB1 although there are less foundations (estimated at around 6) here which are also more widely separated. Arguably, waves from this direction present the most severe test for determining measurable wave effects.

The associated linear correlations between WB1 and WB2 for pre- and post-construction periods are presented in Figure 19.

Pre-construction **Post-construction**

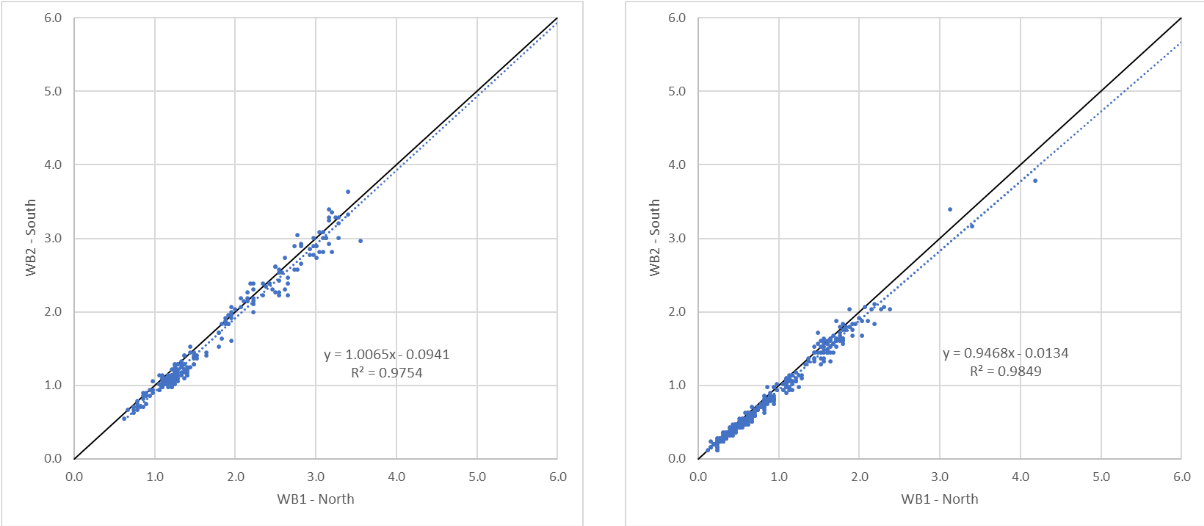


Figure 19: E correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) again show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 98% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 209 events and a spread of wave heights up to around 3.5 m. The post-construction period consists of 384 events and wave heights generally up to 2 m, noting there are also three events above 2 m.

Wave heights at WB1 are marginally larger than equivalent events at WB2 (dotted trendline is towards WB1 from the 1:1 ratio – black line).

Based on the linear trends ($y = a.x + b$) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB2 (Table 13).

Table 13: E wave heights (in metres) based on statistical trendline relationships

WB1	WB2 pre-construction	WB2 post-construction	Difference at WB2
1.00	0.91	0.93	0.02
2.00	1.92	1.89	-0.04
3.00	2.93	2.83 extrapolated	-0.10
4.00	3.93 extrapolated	3.77 extrapolated	-0.16

The difference in wave heights at WB2 between the two periods is considered to be influenced mainly by the relatively small pre and post-construction sample limiting a fully resolved statistical relationship. Furthermore, the wave conditions at WB1 and WB2 did not reach 4 m in the pre-construction period, or 3 m in the post-construction period so the derived values at WB2 are extrapolated beyond the sample range. This is a limitation in the adopted method.

The difference in wave heights at WB2 between the two periods is still considered very similar to NW. There is no discernible scale of effect which can be attributed to the wind farm.

Easterly waves were also a consideration in the EIA wave modelling. Changes in wave height were quantified at a site central to the array at AWAC 2. Table 14 presents the predicted waves heights for a baseline case, for Layout 1 and for the difference attributed to the wind farm. Negative values signify a reduction in wave heights.

Table 14: Predicted wave heights at AWAC 2 for easterly waves (E), wave heights (in metres)

Scenario	Baseline	Layout 1	Difference
50% no exc	1.03	0.86	-0.17
0.1 yr RP	2.92	2.69	-0.23
1 yr RP	5.09	4.82	-0.27
10 yr RP	6.90	6.64	-0.26
50 yr RP	7.92	7.67	-0.25
100 yr RP	8.30	8.07	-0.23

Whilst AWAC 2 is a site central to the array, the level of change once waves have passed through the full width of the array would be expected to be even greater.

h. Comparison of ESE waves

ESE waves have the potential to pass through an area occupied by a moderate number of installed foundations in the post-construction period before reaching WB1. The associated linear correlations between WB1 and WB2 are presented in Figure 20.

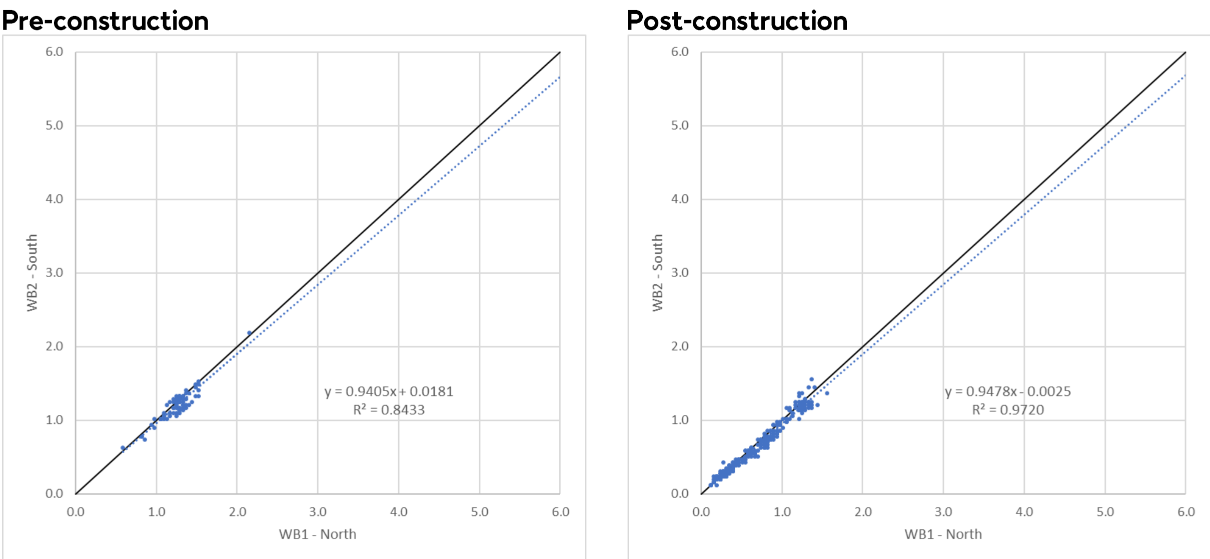


Figure 20: ESE correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) are limited by an incomplete sample distribution which impedes the proper determination of a linear trend, especially for the pre-construction period. The pre-construction period consists of 78 events and a spread of wave heights up to around 1.5 m, with a single event up to around 2 m. The post-construction period consists of 309 events and wave heights up to 1.5 m. Despite these limitations, the coefficient of determination, R^2 shows that WB1 and WB2 are still strongly related to each other (i.e. 84 and 97% of the variation in WB2 can be explained by the variation in WB1).

Wave heights at WB1 are marginally larger than equivalent events at WB2 (dotted trendline is towards WB1 from the 1:1 ratio – black line).

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB1 (Table 15).

Table 15: ESE wave heights (in metres) based on statistical trendline relationships

WB2	WB1 pre-construction	WB1 post-construction	Difference at WB1
1.00	1.04	1.06	-0.02
2.00	2.11	2.11 extrapolated	0.00
3.00	3.17 extrapolated	3.17 extrapolated	0.00
4.00	4.23 extrapolated	4.22 extrapolated	0.01

The difference in wave heights at WB1 between the two periods is considered to be influenced mainly by the relatively small pre- and post-construction sample limiting a fully resolved statistical relationship.

i. Comparison of SE waves

SE waves have the potential to pass through an area occupied by a small number of installed foundations in the post-construction period before reaching WB1. The associated linear correlations between WB1 and WB2 are presented in Figure 21.

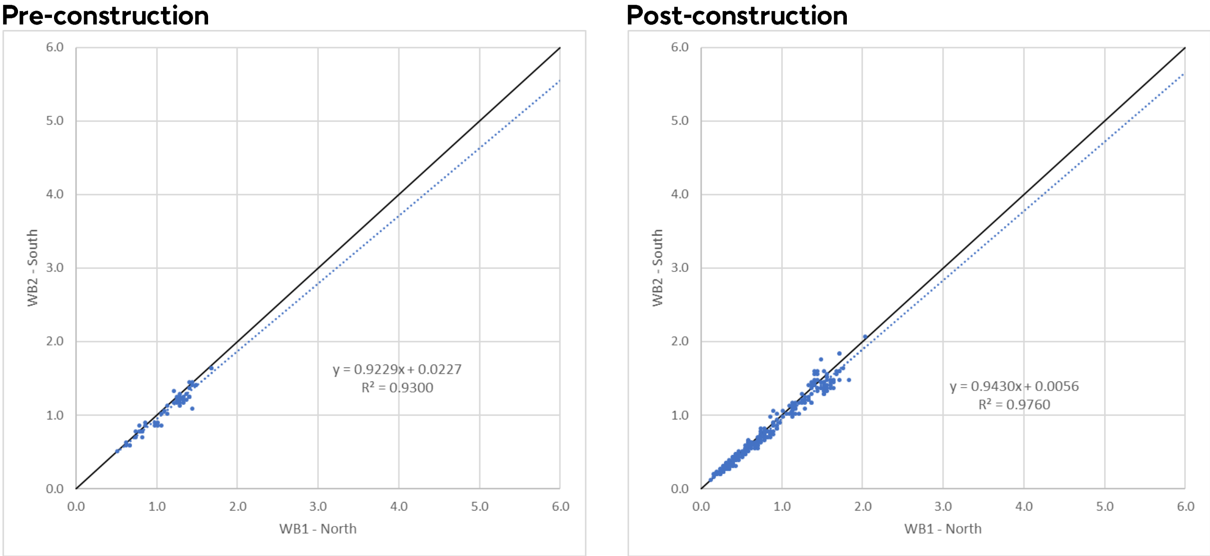


Figure 21: SE correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) are limited by an incomplete sample distribution which impedes the proper determination of a linear trend, especially for the pre-construction period. The pre-construction period consists of 52 events and a spread of wave heights up to around 1.5 m. The post-construction period consists of 339 events and wave heights up to 2 m. Despite these limitations, the coefficient of determination, R^2 shows that WB1 and WB2 are still strongly related to each other (i.e. 93 and 98% of the variation in WB2 can be explained by the variation in WB1).

Wave heights at WB1 are marginally larger than equivalent events at WB2 (dotted trendline is towards WB1 from the 1:1 ratio – black line).

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB1 (Table 16).

Table 16: SE wave heights (in metres) based on statistical trendline relationships

WB2	WB1 pre-construction	WB1 post-construction	Difference at WB1
1.00	1.06	1.05	-0.01
2.00	2.14 extrapolated	2.11 extrapolated	-0.03
3.00	3.23 extrapolated	3.18 extrapolated	-0.05
4.00	4.31 extrapolated	4.24 extrapolated	-0.07

The difference in wave heights at WB1 between the two periods is considered to be influenced mainly by the relatively small pre- and post-construction sample limiting a fully resolved statistical relationship.

j. Comparison of SSE waves

SSE waves have the potential to pass through an area occupied by a small number of installed foundations in the post-construction period before reaching WB1. The associated linear correlations between WB1 and WB2 are presented in Figure 22.

Pre-construction **Post-construction**

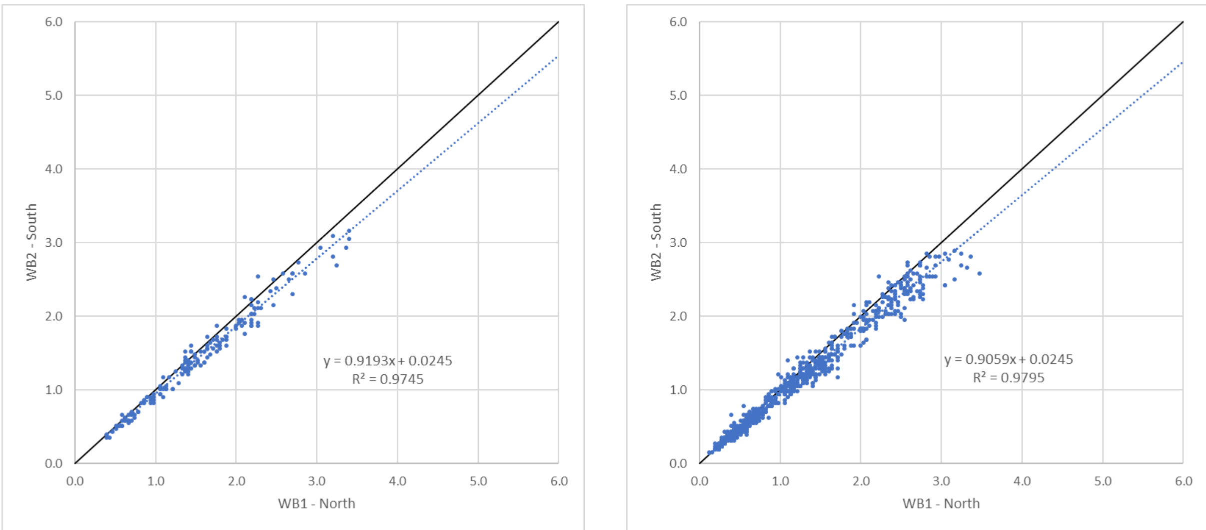


Figure 22: SSE correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 97 and 98% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 160 events and a spread of wave heights up to around 3 m. The post-construction period consists of 969 events and wave heights generally up to 3 m.

Wave heights at WB1 are marginally larger than equivalent events at WB2 (dotted trendline is towards WB1 from the 1:1 ratio – black line).

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB1 (Table 17).

Table 17: SSE wave heights (in metres) based on statistical trendline relationships

WB2	WB1 pre-construction	WB1 post-construction	Difference at WB1
1.00	1.06	1.08	-0.02
2.00	2.15	2.18	-0.03
3.00	3.24	3.28	-0.04

WB2	WB1 pre-construction	WB1 post-construction	Difference at WB1
4.00	4.32 extrapolated	4.39 extrapolated	-0.05

The difference in wave heights at WB1 between the two periods is small and in line with the statistical variance deduced for NW waves. There is no discernible scale of effect which can be attributed to the wind farm.

k. Comparison of S waves

S waves have the potential to pass through an area occupied by a small number of installed foundations in the post-construction period before reaching WB1. The associated linear correlations between WB1 and WB2 are presented in Figure 23.

Pre-construction **Post-construction**

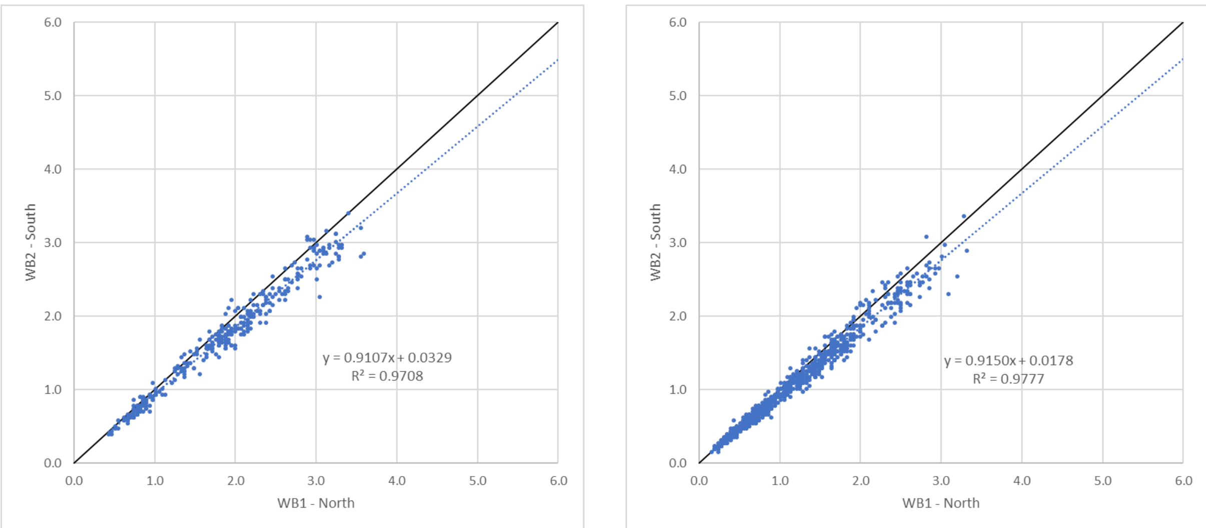


Figure 23: S correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 97 and 98% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 331 events and a spread of wave heights up to around 3.5 m. The post-construction period consists of 874 events and wave heights generally up to 3.5 m. Both periods appear well-resolved over this range of wave heights.

Wave heights at WB1 are marginally larger than equivalent events at WB2 (dotted trendline is towards WB1 from the 1:1 ratio – black line).

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB1 (Table 18).

Table 18: S wave heights (in metres) based on statistical trendline relationships

WB2	WB1 pre-construction	WB1 post-construction	Difference at WB1
1.00	1.06	1.07	0.01
2.00	2.16	2.17	0.01
3.00	3.26	3.26	0.00
4.00	4.36 extrapolated	4.35 extrapolated	-0.01

The difference in wave heights at WB1 between the two periods is small and in line with the statistical variance deduced for NW waves. There is no discernible scale of effect which can be attributed to the wind farm.

l. Comparison of SSW waves

SSW waves have the potential to pass through an area occupied by a small number of installed foundations in the post-construction period before reaching WB1. The associated linear correlations between WB1 and WB2 are presented in Figure 24.

Pre-construction **Post-construction**

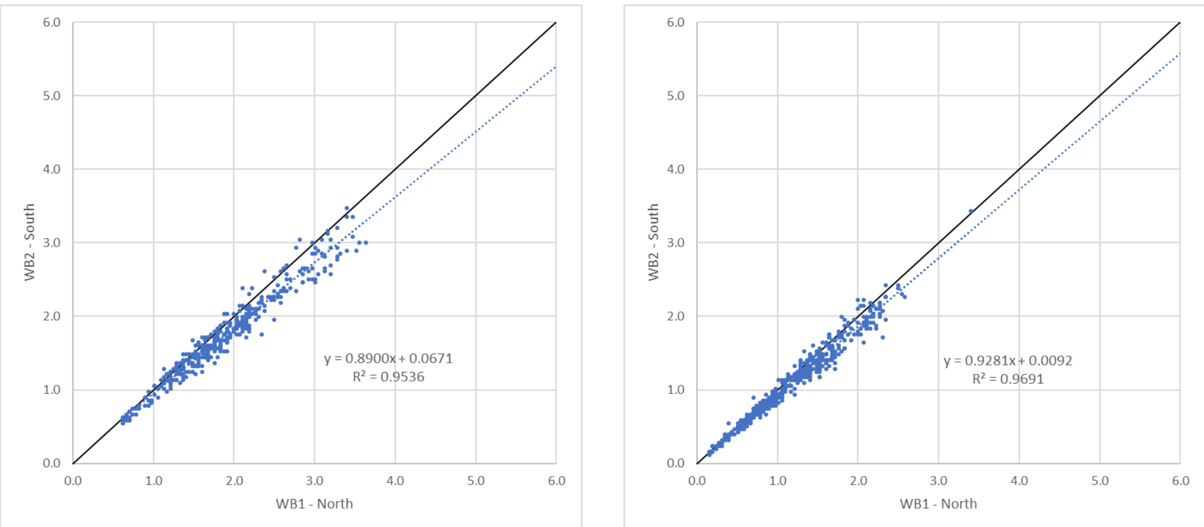


Figure 24: SSW correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 95 and 97% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 376 events and a spread of wave heights up to around 3.5 m. The post-construction period consists of 506 events and wave heights generally up to 2.5 m, although there is a single event up to 3.5 m.

Wave heights at WB1 are marginally larger than equivalent events at WB2 (dotted trendline is towards WB1 from the 1:1 ratio – black line).

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB1 (Table 19).

Table 19: SSW wave heights (in metres) based on statistical trendline relationships

WB2	WB1 pre-construction	WB1 post-construction	Difference at WB1
1.00	1.05	1.07	0.02
2.00	2.17	2.15	-0.02
3.00	3.30	3.22	-0.08
4.00	4.42 extrapolated	4.30 extrapolated	-0.12

The difference in wave heights at WB1 between the two periods is small and in line with the statistical variance deduced for NW waves. There is no discernible scale of effect which can be attributed to the wind farm.

m. Comparison of SW waves

SW waves have the potential to pass through an area occupied by a small number of installed foundations in the post-construction period before reaching WB1. The associated linear correlations between WB1 and WB2 are presented in Figure 25.

Pre-construction **Post-construction**

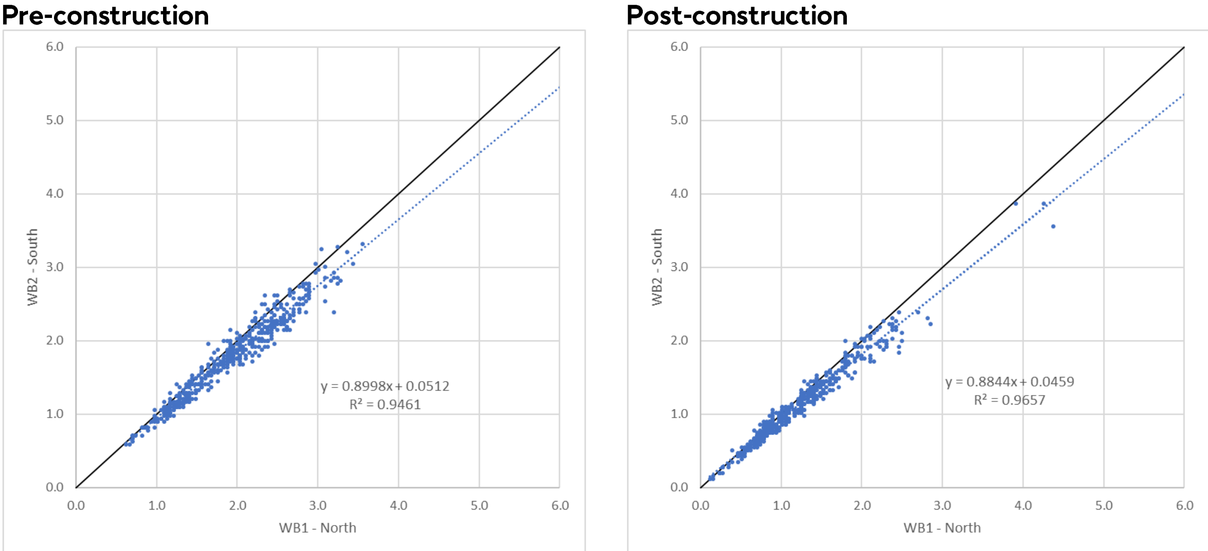


Figure 25: SW correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 95 and 97% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 450 events and a spread of wave heights up to around 3.5 m. The post-construction period consists of 540 events and wave heights generally up to 2.5 m, although there are a few events up to around 4 m.

Wave heights at WB1 are marginally larger than equivalent events at WB2 (dotted trendline is towards WB1 from the 1:1 ratio – black line).

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB1 (Table 20).

Table 20: SW wave heights (in metres) based on statistical trendline relationships

WB2	WB1 pre-construction	WB1 post-construction	Difference at WB1
1.00	1.05	1.08	0.03
2.00	2.17	2.21	0.04
3.00	3.28	3.34	0.06
4.00	4.39 extrapolated	4.47	0.08

The difference in wave heights at WB1 between the two periods is small and in line with the statistical variance deduced for NW waves. There is no discernible scale of effect which can be attributed to the wind farm.

n. Comparison of WSW waves

WSW waves have the potential to pass through an area occupied by a small number of installed foundations in the post-construction period before reaching WB1. The associated linear correlations between WB1 and WB2 are presented in Figure 26.

Pre-construction **Post-construction**

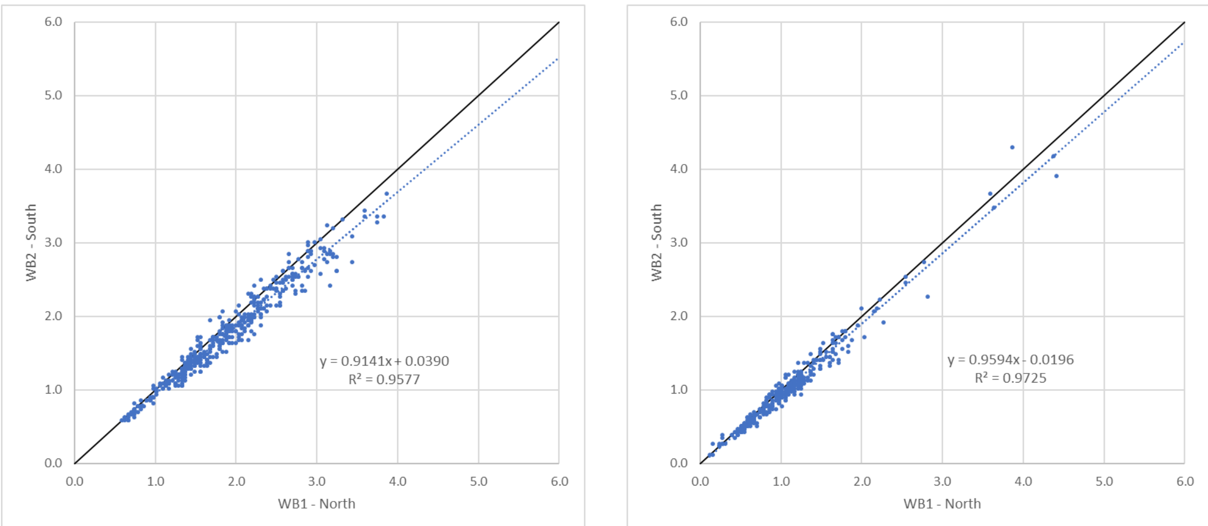


Figure 26: WSW correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 96 and 97% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 423 events and a spread of wave heights up to around 4 m. The post-construction period consists of 340 events and wave heights mainly up to 2 m, although there are a few events up to around 4 m.

Wave heights at WB1 are marginally larger than equivalent events at WB2 (dotted trendline is towards WB1 from the 1:1 ratio – black line).

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB1 (Table 21).

Table 21 - WSW wave heights (in metres) based on statistical trendline relationships

WB2	WB1 pre-construction	WB1 post-construction	Difference at WB1
1.00	1.05	1.06	0.03
2.00	2.15	2.11	-0.04
3.00	3.24	3.15	-0.09
4.00	4.33	4.19	-0.14

The difference in wave heights at WB1 between the two periods is small and in line with the statistical variance deduced for NW waves. There is no discernible scale of effect which can be attributed to the wind farm.

o. Comparison of W waves

E waves have the potential to pass through an area occupied by a moderate number of more closely spaced foundations prior to reaching WB2 (estimated at around 7). Waves will also be passing through the top of the array to reach WB1 although there are less foundations (estimated at around 2) here which are also more widely separated.

The associated linear correlations between WB1 and WB2 for pre- and post-construction periods are presented in Figure 27.

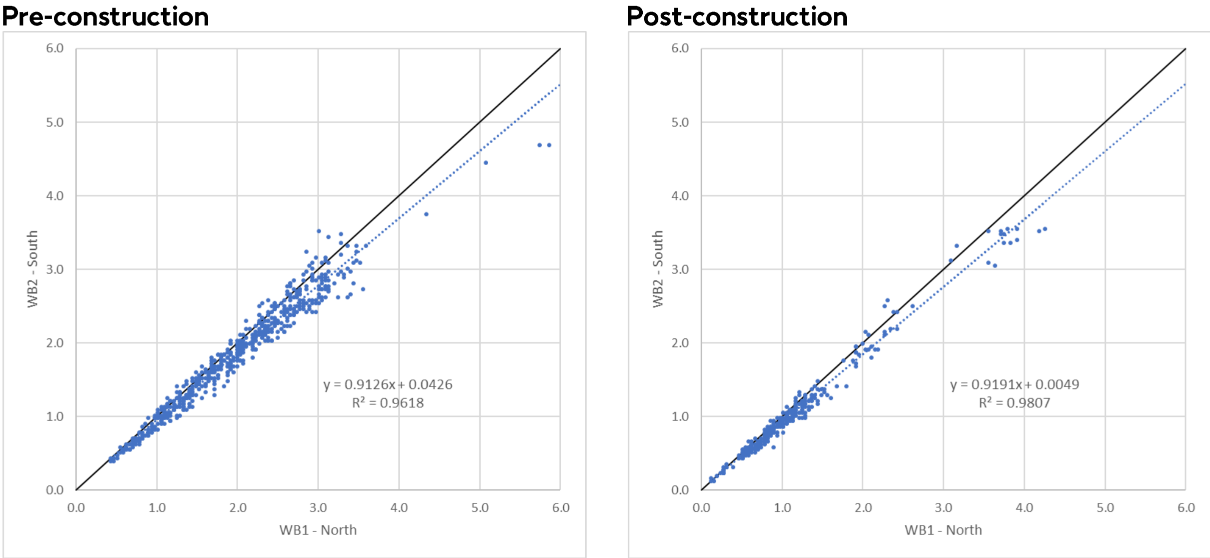


Figure 27: W correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 96 and 98% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 589 events and a spread of wave heights up to around 3.5 m, although there are a few larger wave heights close to 6 m (at WB1). The post-construction period consists of 310 events and wave heights mainly up to 4 m.

Wave heights at WB1 are marginally larger than equivalent events at WB2 (dotted trendline is towards WB1 from the 1:1 ratio – black line).

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB1 (Table 22).

Table 22: W wave heights (in metres) based on statistical trendline relationships

WB1	WB2 pre-construction	WB2 post-construction	Difference at WB2
1.00	0.96	0.92	-0.04
2.00	1.87	1.84	-0.03
3.00	2.78	2.76	-0.02
4.00	3.69	3.68	-0.01

The difference in wave heights at WB1 between the two periods is small and in line with the statistical variance deduced for NW waves. There is no discernible scale of effect which can be attributed to the wind farm.

p. Comparison of WNW waves

WNW waves have the potential to pass through an area occupied by a moderate number of installed foundations in the post-construction period before reaching WB2. The associated linear correlations between WB1 and WB2 are presented in Figure 28.

Pre-construction **Post-construction**

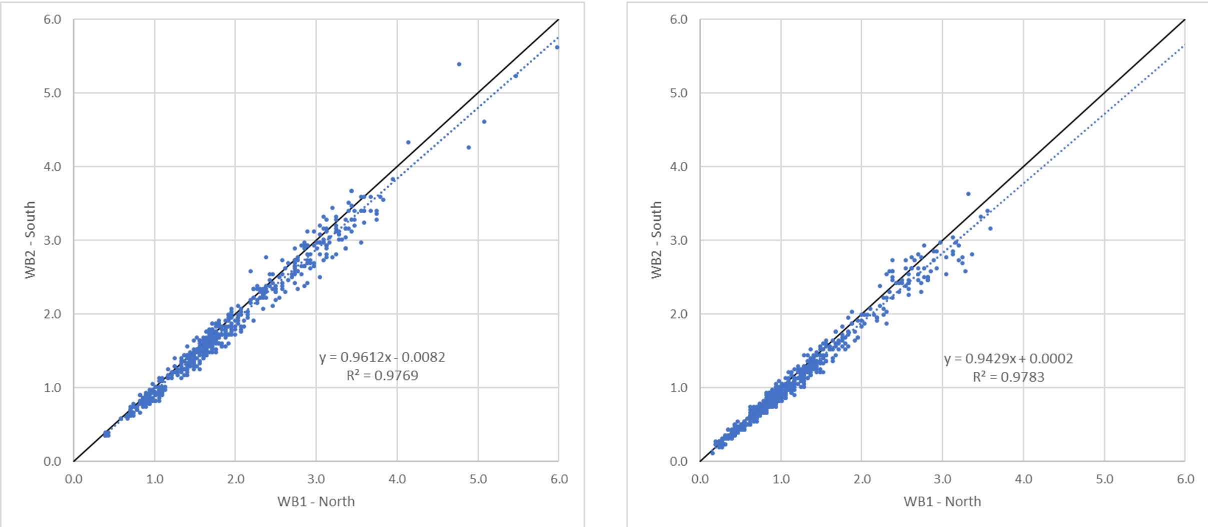


Figure 28: WNW correlation between WB1 and WB2 for pre-construction (left) and post-construction (right) periods

Both periods of wave activity (pre- and post-construction) show a linear correlation with the coefficient of determination, R^2 close to 1.0 between WB1 and WB2, meaning that wave conditions are strongly related to each other (i.e. 98% of the variation in WB2 can be explained by the variation in WB1). The pre-construction period consists of 554 events and a spread of wave heights up to around 6 m. The post-construction period consists of 636 events and wave heights mainly up to 3.5 m.

Wave heights at WB1 are marginally larger than equivalent events at WB2 (dotted trendline is towards WB1 from the 1:1 ratio – black line).

Based on the linear trends (y = a.x + b) for pre- and post-construction periods, the deduced relationship between WB1 and WB2 would lead to the following wave heights at WB1 (Table 23).

Table 23: WNW wave heights (in metres) based on statistical trendline relationships

WB1	WB2 pre-construction	WB2 post-construction	Difference at WB2
1.00	0.95	0.94	-0.01
2.00	1.91	1.89	-0.02
3.00	2.88	2.83	-0.05
4.00	3.84	3.77 extrapolated	-0.07

The difference in wave heights at WB1 between the two periods is small and in line with the statistical variance deduced for NW waves. There is no discernible scale of effect which can be attributed to the wind farm.

9. Summary

This review has examined the operational wave monitoring for possible wind farm effects. The relevance of the EIA wave modelling is also considered to provide a means to justify the further applicability of these models to support the EIA of Hornsea Four.

Developing this technical note has added to the evidence base by extending the baseline understanding for waves, substantiating the EIA methodology and assumptions, and establishing a dataset which helps quantify the effects of an offshore wind farm on waves, based on monopile options.

9.1 EIA considerations

The EIA for Hornsea Project One has been based on the Rochdale Envelope approach which is inherently conservative and presents a realistic worst-case option (or maximum design scenario) selected from a project design statement. The outcome typically adopts the greatest number and most closely spaced foundations with the largest diameter, generally gravity bases. This combination of design options leads to the highest level of potential blockage to waves passing the array. The same approach will be adopted for Hornsea Four meaning that the existing EIA assessments for adjacent projects will tend to be equivalent. The Rochdale Envelope approach assumes that any less conservative option will have a reduced level of impact.

What is noteworthy here is that, despite the conservative approach based on gravity base structures (GBS) for turbines in the array areas, the outcomes of wave modelling for Hornsea Project One, Hornsea Project Two and Hornsea Three each concluded that potential (blockage related) impacts to the shoreline, offshore sandbanks and the Flamborough Front would only result in effects of negligible or minor adverse significance (SMart Wind, 2013), (SMart Wind, 2015) and (Ørsted, 2018).

The consented schemes of Hornsea Project One and Hornsea Project Two that are now being delivered are based on final engineering decisions which are using a fewer number of smaller diameter foundations (i.e. monopile options rather than the consented option based on GBS). These decisions dramatically reduce the potential level of blockage on waves passing through these arrays. Table 24 provides a comparison between the consented assumption for foundations and the final foundation choice being constructed for Hornsea Project One and Hornsea Project Two. The area occupied by structures across each array is also provided to help develop a proxy for relative level of blockage. The relative level of reduction from the consented GBS case to the monopile options now being delivered in each case is 73 times less for Hornsea Project One and 52 times less for Hornsea Project Two (based on the conservative EIA case of a 10 m diameter monopile). Arguably, this level of reduction in blockage would expect to lead to negligible impacts on waves rather than any minor adverse level of significance.

Table 24: Comparison between consented maximum design scenario and final foundation options for Hornsea One and Two

Maximum Design Parameter	Hornsea Project One		Hornsea Project Two	
	Consented	Final	Consented	Final
Number of turbines	335	174.0	258	165
Minimum turbine spacing (m)	924	925.7	932	TBC
Foundation options	GBS	Monopile	GBS	Monopile
Foundation diameter (m)	50	8.1	58	10 (EIA)
Blockage (% of array area occupied by structures)	0.162	0.002	0.148	0.003

Apart from a much-reduced level of blockage between consented and final design options, an associated reduction is that the smaller diameter monopiles will generally fall below the threshold for diffraction and scattering type effects to be relevant, also implying a lesser level of wave interference across the array.

Hornsea Project One and Hornsea Project Two are adjacent projects to Hornsea Four with the potential to create a cumulative impact on waves from all three built projects acting together. However, since Hornsea Project One and Hornsea Project Two have now committed to scheme designs with much reduced levels of blockage than described in their respective ES, the approach to cumulative impacts needs to be rescaled according to the findings of the present review of operational wave monitoring from Hornsea Project One.

9.2 Operational wave monitoring

The available dataset for operational wave monitoring at Hornsea Project One has been associated to different phases of offshore activity which are used to define a pre- and post-construction period of wave conditions, representing a project which has installed 74% of planned monopile foundations at the time of developing this technical note.

The high level of similarity in the two datasets, WB1 – north and WB2 – south. (for wave height, period and direction) supports the case to generalise the measurements to be representative of the northern and southern boundaries of the array.

The dataset has been divided into 16 directional sectors which also align with the directional wave scenarios considered in the EIA assessment.

A statistical correlation of wave heights for each directional sector provides the most appropriate means to compare waves between WB1 and WB2 for pre- and post-construction periods. The approach is most effective when there is a high number of samples for both pre- and post-construction periods which are also reasonably spread in range.

The NW directional sector offers a unique opportunity to consider a situation where pre- and post-construction cases are effectively the same without any foundations. This is due to the western side of the project not being installed at the time of the review of operational wave monitoring data. The pre- and post-construction correlation relationships derived for the NW sector are similar but also quantify a level of statistical variance that cannot be attributed to any installed foundations.

The remaining directional sectors have then been examined for a scale of change that is detectable above the statistical variance from the NW sector. This is based on the derived linear trendline.

The sectors of N, NNW, NE, ENE and E are also contrasted to comparable information from the EIA wave modelling for context. The level of change quantified by the wave modelling does not include any statistical variance as like-for-like conditions are exactly represented for the baseline (pre-construction) and Layout 1 (post-construction) cases.

Some directional sectors are limited by a low number of samples and/or a restricted range, however, those that offer sufficient information for both pre- and post-construction periods suggest there is no additional scale of change detectable beyond the statistical variance shown for the NW sector. This outcome is not unsurprising since the monopile case uses substantially smaller level of blockage than EIA Layout 1 (i.e. fewer and smaller foundations which are more widely spaced, and which have a ration of $D/L < 0.2$. Any effects on waves due to the monopile foundations are therefore considered to be undiscernible from the statistical variance and must be at a low level which has minimal influence. These effects would not be important beyond the wind farm area.

10. Conclusions

This Technical Note presents important factual evidence of the unresolvable low-level of wave related interference from a monopile array of foundations; a typical configuration of the majority of operational Round 3 installations.

This finding is consistent with previous research conducted on a smaller Round 1 wind farm at Scroby Sands. This research project is widely known as AE1227: Assessment of the Significance of Changes to the Inshore Wave Regime as a consequence of an Offshore Wind Array (Cefas, 2005). The research selected Scroby Sands as a 'worst case' scenario for wave interaction from a Round 1 wind farm due to the relatively large pile diameter to water depth ratio, a situation more likely to cause stronger wave-structure interaction. As part of the study, field measurements of the wave regime down-wind of the wind farm array were collected in 2003 using X-band radar. The research concluded that slender monopiles at typical spacing (6-8 rotor diameters) do not have a significant potential to cause measurable wave reduction, diffraction or interference and therefore do not have a significant potential to modify local or far-field sediment transport processes.

The wave effects from Hornsea Project Two (based on the final monopile option) are now expected to be consistent with those considered in this Technical Note for Hornsea Project One, with the cumulative situation also considered to be unresolvable and impacts negligible.

Since, HOW04 is no longer considering the GBS option going forwards into PEIR, then the previous wave modelling of adjacent projects, based on the conservative GBS assumption, becomes less applicable. If the relative blockage effects of HOW04 can be shown in the PEIR to be closer to the built cases of Hornsea Project One and Two, then the findings of this Technical Note are considered to become more applicable evidence of the array related wave effects and offer the basis of asserting negligible impact.

11. References

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