FIVE ESTUARIES OFFSHORE WIND FARM

PRELIMINARY ENVIRONMENTAL INFORMATION REPORT

VOLUME 4, ANNEX 2.3: PHYSICAL PROCESSES TECHNICAL ASSESSMENT

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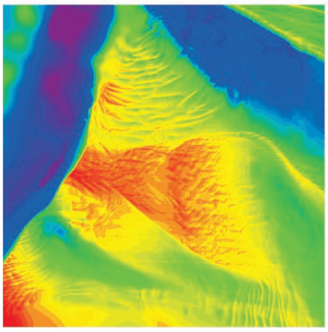
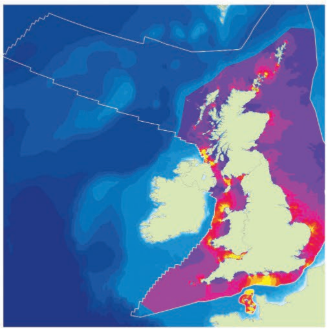
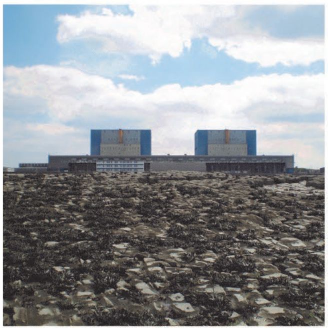
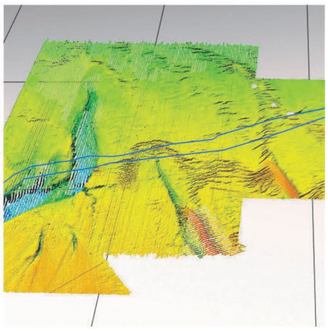
Five Estuaries Offshore Windfarm Environmental Impact Assessment

Volume 4, Annex 2.3: Physical Processes Technical Assessment

March 2023

Innovative Thinking - Sustainable Solutions

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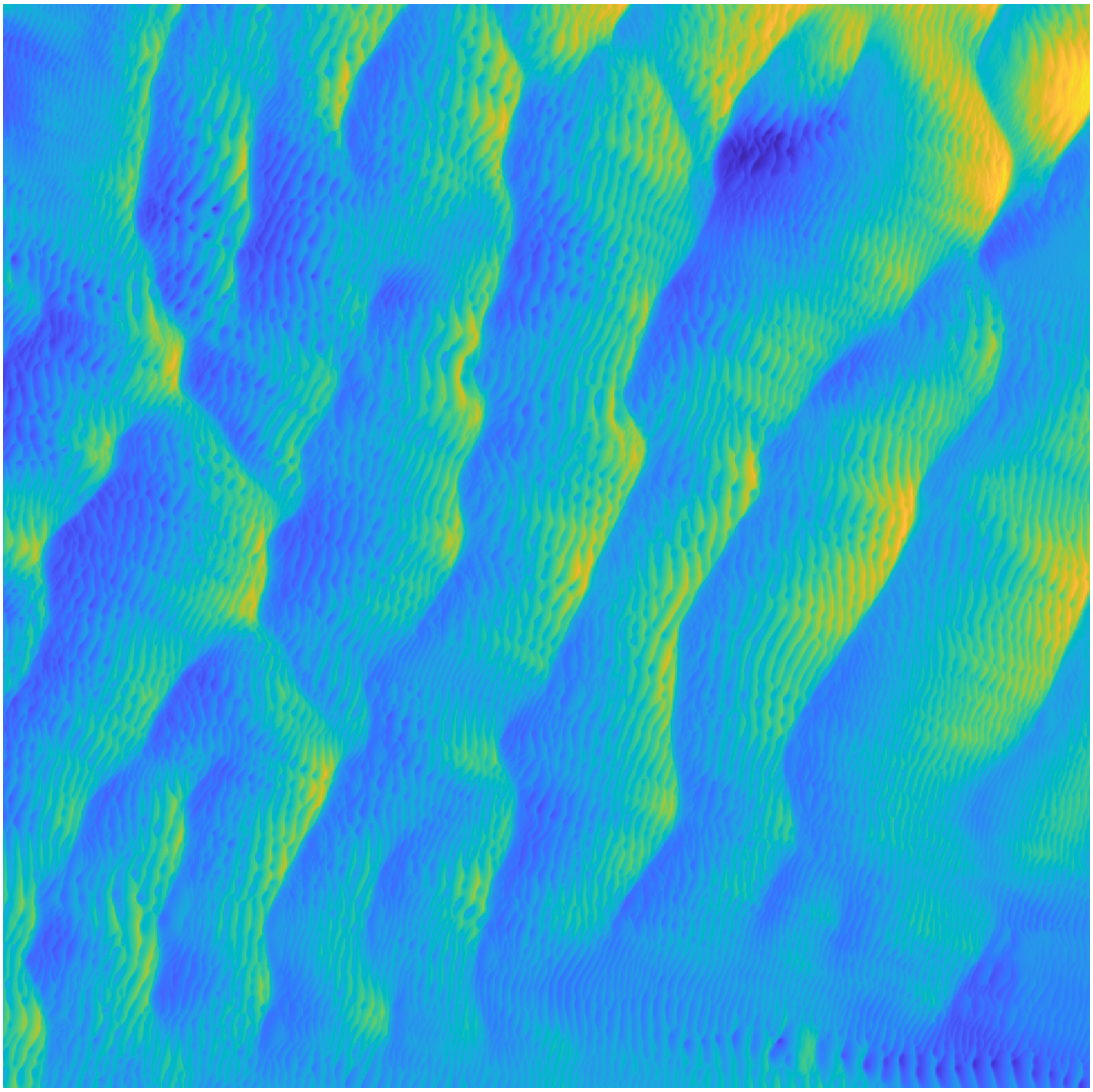

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Acronyms and abbreviations

BERR	Department for Business, Enterprise and Regulatory Reform
BMAPA	British Marine Aggregate Producers Association
CEA	Cumulative Effects Assessment
COWRIE	Collaborative Offshore Wind Research Into The Environment
D	Diameter
DNV	Det Norske Veritas
DTI	Department of Trade
ECC	Export Cable Corridor
EIA	Environmental Impact Assessment
EMODnet	European Marine Observation and Data Network
FM	Flexible Mesh (MIKE21 Model)
G/D	Gap to Pile Diameter Ratio
GoBe	GoBe Consultants Ltd
GOWF	Galloper Offshore Wind Farm
HD	Hydrodynamic (MIKE21 Model)
HDD	Horizontal Directional Drilling
Hs	Significant Wave Height
KC	Keulegan-Carpenter
LAT	Lowest Astronomical Tide
MDS	Maximum Design Scenario
MFE	Mass Flow Excavator
MW	Megawatt(s)
ºN	Degrees North
OSP	Offshore Substation Platform

OWF	Offshore Wind Farm
PEIR	Preliminary Environmental Information Report
RP	Return Period
RWE	RWE
SPM	Suspended Particulate Matter
SSC	Suspended Sediment Concentration
S_{extent}	Scour Extent
$S_{footprint}$	Scour Footprint
ST	Sand Transport (MIKE21 Model)
SW	Spectral Wave (MIKE21 Model)
TEDA	Thames Estuary Dredging Association
Tp	Peak Wave Period
TSHD	Trailing Suction Hopper Dredger
Tz	Zero Crossing Period
UK	United Kingdom
UTM	Universal Transverse Mercator
VE	Five Estuaries (Offshore Wind Farm)
VORF	Vertical Offshore Reference Frames
WGS	World Geodetic System
WTG	Wind Turbine Generator

Cardinal points/directions are used unless otherwise stated.

SI units are used unless otherwise stated.

1 Introduction

1.1 Overview

1.1.1 This Report

ABPmer has been commissioned to deliver the Marine Geology, Oceanography and Physical Processes Environmental Impact Assessment (EIA) for the proposed Five Estuaries Wind Farm (referred to here as VE) (Figure 1). This annex provides supporting technical analysis underpinning the following coastal processes assessments presented in the Preliminary Environmental Information Report (PEIR) Volume 2, Chapter 2.2: Marine Geology, Oceanography and Physical Processes:

- Changes to suspended sediment concentrations, bed levels and sediment type (Section 2);
- Changes to the wave regime (Section 3);
- Changes to the tidal regime and tidally driven sediment transport regime (Section 4); and
- Scour and seabed alteration (Section 6).

The assessments presented in this technical annex have been informed by:

- The collation and analysis of baseline information (as set out in Volume 4, Annex 2.1: Physical Processes Baseline Technical Report); and
- Hydrodynamic, wave and sediment plume modelling (the setup of which is set out in Volume 4, Annex 2.2: Physical Processes Model Design and Validation).

1.1.2 The Proposed Five Estuaries Offshore Wind Farm

The proposed Five Estuaries Array Areas and Offshore Export Cable Corridor are located in the southern North Sea, within the Approaches to the Outer Thames Estuary, on the east coast of England.

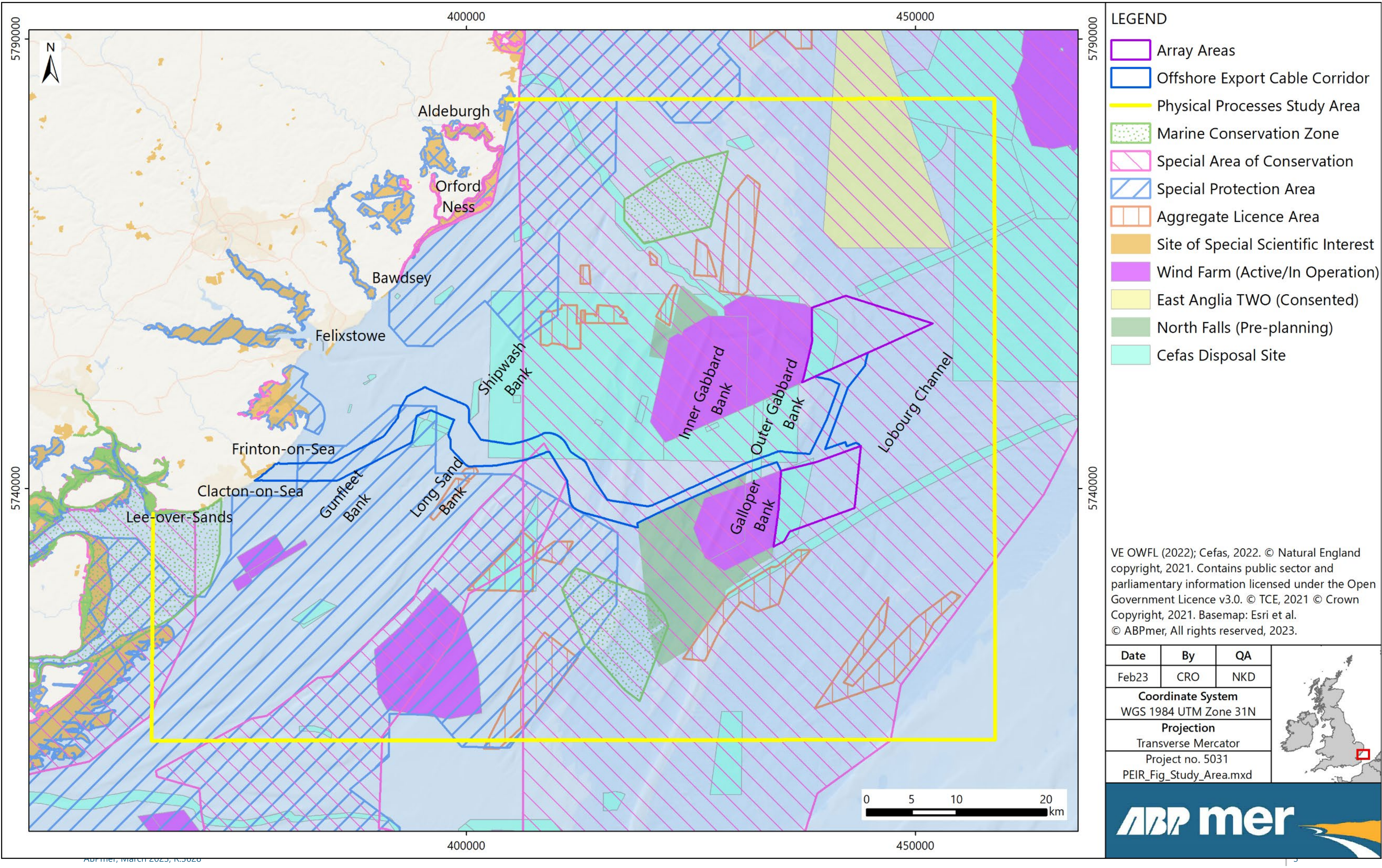
VE is a proposed sister project to the east of the Galloper offshore wind farm (GOWF, operational since 2018), approximately 30 km off the Suffolk coast. The Export Cable Corridor (ECC) runs approximately westward from the Five Estuaries Array Areas to a landfall area between Frinton-on-Sea and Holland Haven, on the Essex coast (Figure 1).

The study area shown in Figure 1 has been informed by expert judgement, based on (amongst other things) physical process understanding developed from work undertaken for the nearby (operational) Galloper and Greater Gabbard OWFs and analysis of prevailing wave direction and tidal excursion distance. Direct changes to the seabed will be confined to the array and ECC, with indirect changes (e.g. due to disruption of waves, tides or sediment pathways) experienced both inside and outside of the Project boundary. These indirect changes are expected to diminish with distance from the array and ECC.

1.2 Approach

In order to assess the potential changes relative to the baseline (existing) coastal and marine environment, a combination of complementary approaches have been adopted for the VE marine physical processes assessment. These include:

- The 'evidence base' containing monitoring data collected during the construction, and operation and maintenance of other offshore wind farm developments. The evidence base also includes results from numerical modelling and desk-based analyses undertaken to support other offshore wind farm EIAs, especially that used to support the consenting processes for the adjacent operational Galloper, and Greater Gabbard offshore wind farms.
- New numerical modelling to consider potential changes to hydrodynamics and waves and sediment transport in response to the construction, operation and decommissioning of VE;
- Analytical assessments of VE project-specific data, including the application of rule-based and spreadsheet based numerical models; and
- Standard empirical equations describing the relationship between (for example) hydrodynamic forcing and sediment transport or settling and mobilisation characteristics of sediment particles released during construction activities (e.g. Soulsby, 1997).


400000
450000
5790000
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N
Aldeburgh
Orford
Ness
Bawdsey
Felixstowe
Frinton-on-Sea
Clacton-on-Sea
Lee-over-Sands
Gunfleet Bank
Long Sand Bank
Shipwash Bank
Inner Gabbard Bank
Outer Gabbard Bank
Galloper Bank
Lobourg Channel
0 5 10 20 km
LEGEND
Array Areas
Offshore Export Cable Corridor
Physical Processes Study Area
Marine Conservation Zone
Special Area of Conservation
Special Protection Area
Aggregate Licence Area
Site of Special Scientific Interest
Wind Farm (Active/In Operation)
East Anglia TWO (Consented)
North Falls (Pre-planning)
Cefas Disposal Site
VE OWFL (2022); Cefas, 2022. © Natural England copyright, 2021. Contains public sector and parliamentary information licensed under the Open Government Licence v3.0. © TCE, 2021 © Crown Copyright, 2021. Basemap: Esri et al. © ABPmer, All rights reserved, 2023.
Date | By | QA
Feb23 | CRO | NKD
Coordinate System
WGS 1984 UTM Zone 31N
Projection
Transverse Mercator
Project no. 5031
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2 Changes to suspended sediment concentrations and seabed levels

This section outlines the assessment of potential changes to suspended sediment concentrations, seabed levels and sediment characteristics due to sediment disturbance caused by construction activities.

2.1 Overview

Local increases in suspended sediment concentration (SSC) may result from the disturbance of sediment by construction related activities, most notably due to:

- drilling of monopile foundations and pin piles for jacket foundations;
- seabed preparation by dredging prior to jacket suction bucket foundation installation;
- sandwave clearance (prior to cable burial);
- cable burial; and
- drilling fluid release during Horizontal Directional Drilling (HDD) at the landfall.

The mobilised material may be transported away from the disturbance location by the local tidal regime. According to the source-pathway-receptor model:

- disturbance and release of sediment is considered as the source of potential changes to SSC in the water column;
- tidal currents act as the pathway for transporting the suspended sediment; and
- the receptor is a feature potentially sensitive to any increase in suspended sediments and consequential deposition.

The magnitude, duration, rate of change and frequency of recurrence of changes to SSC and bed level are variable between operation types and in response to natural variability in the controlling environmental parameters.

2.2 Baseline conditions

A more detailed description of naturally varying SSC in the study area can be found in Volume 4, Annex 2.1: Physical Processes Baseline Technical Report. The following summary information is repeated here.

Within the offshore array areas, surface suspended particulate matter (SPM) concentration is approximately 1 to 3 mg/l during summer, increasing during winter to approximately 10 to 20 mg/l. Relatively higher values are anticipated during spring tides and storm conditions, and closer to the seabed. Within the offshore ECC, natural SSC increases towards the coastline due to higher tidal current speeds, and generally shallower water depths allowing greater seabed disturbance by wave action. During winter months, surface SPM concentration at the landward end of the ECC will normally exceed 100 mg/l and, as for the array areas, relatively higher values are anticipated during spring tides and storm conditions, and closer to the seabed.

2.3 Assessment methodology

This assessment of changes to suspended sediment concentration and associated deposition of sediment as a result of activities related to VE is informed by location and project specific numerical (spreadsheet) modelling (described below in Section 2.3.1). Based on previous stakeholder engagement and discussion with other EIA topics using the results of these assessments, the quantitative detail of the modelling results for all individual activities are reported as a descriptive summary and a series of spatial maps.

The theoretical basis for, and the results of, the following assessments are consistent with the results of observational (monitoring) evidence (e.g. BERR, 2008;), previous explicit numerical modelling of sediment plumes for analogous activities and environmental settings (e.g. TEDA, 2010; and by ABPmer for East Anglia ONE; Navitus Bay; Hornsea Four; Awel y Môr; Erebus), similar spreadsheet modelling for other wind farms by ABPmer (e.g. for Burbo Bank Extension; Walney Extension; Thanet Extension; Hornsea Three; Erebus), and results from other (various) consultants and wind farm EIAs, which normally also use a similar range of methodologies.

The maximum design scenario (MDS) for each activity type is determined using the information contained in the full offshore project design description (Volume 2, Chapter 1: Offshore Project Description). For each activity, the rate and duration of sediment disturbance and the total sediment volume is calculated for individual occurrences and for all occurrences of the activity, including the range of design permutations (e.g. a smaller number of larger foundations or a larger number of smaller foundations). Scenarios are identified that are likely to correspond to the realistic 'worst case' in terms of instantaneous and overall effects. The effect of all other options in the design envelope are therefore expected to be equal to or less than the results presented in this report.

2.3.1 Spreadsheet based numerical models

In order to inform the assessment of potential changes to SSC and bed levels arising from construction related activities, a number of spreadsheet based numerical models have been developed for use. Similar models were developed and used to inform the environmental impact assessments for similar activities at Burbo Bank Extension, Walney Extension, Navitus Bay, Thanet Extension, Hornsea Three and Erebus offshore wind farms (DONG Energy, 2013a,b; Navitus Bay Development Ltd, 2014; Vattenfall, 2018; Ørsted, 2018; and Blue Gem Wind, 2022, respectively).

The spreadsheet based numerical models used here are based upon the following information, assumptions and principles:

- Re-suspended coarser sediments (sands and gravels) will settle relatively rapidly to the seabed and their dispersion can therefore be considered on the basis of a 'snapshot' of the ambient conditions which are unlikely to vary greatly between the times of sediment release and settlement to the seabed. Re-suspended finer sediments may persist in the water column for hours or longer and so their dispersion is considered instead according to the longer-term net tidal current drift rate and direction in the area, which vary both temporally and spatially in speed and direction;

- A representative current speed for the array area is 0.5 m/s, which is representative of higher tidal flow conditions occurring on most flood and ebb cycles for a range of spring and neap conditions (See Section 4.2 and Appendix B Figures B1 and B2, for the full range across the area). Assuming a higher value will increase dispersion, decrease SSC and reduce the thickness of subsequent deposits and *vice versa*. In practice, a range of actual local conditions and outcomes are likely;

- Lateral dispersion of SSC in the plume is controlled by the horizontal eddy dispersion coefficient, Ke, estimated as Ke = $\kappa u^* z$ (Soulsby, 1997), where, z is the height above the seabed (a representative value of half the water depth is used), κ is the von Kármán coefficient (κ = 0.4) and u^* is the friction velocity ($u^* = \sqrt{(\tau/\rho)}$). Where ρ is the density of seawater (ρ = 1027 kg/m^3) and τ is the bed shear stress, calculated using the quadratic stress law ($\tau = \rho\ C_d$ U2, Soulsby, 1997) using a representative current speed for the Proposed Development site (U = 0.5 m/s) and a drag coefficient value for a rippled sandy seabed (C_d = 0.006);

- The interpreted geophysical data and sediment grab samples from the array area indicate that in general there are five characteristic surficial sediment types present, namely:
  - Sand;
  - Gravelly Sand;
  - Slightly gravelly Sand;
  - Slightly gravelly muddy Sand;
  - Muddy Sand.

- To estimate the time-scale in suspension, sediment is assumed to settle downwards at a calculated (theoretical) settling velocity for each grain size fraction (0.0001 m/s for fines, 0.05 m/s for (medium) sands and 0.5 m/s for gravels and generally coarser sediments, including clastic drill arisings).

The numerical model for SSC resulting from the release of sands and gravels is constructed as follows:

- The time required for sediment to settle at the identified settling velocity through a range of total water depths representative of the site is calculated, to yield the duration for settlement;

- The horizontal distance downstream that the plume is advected is found as the product of the representative ambient current speed and the duration for settlement;

- The horizontal footprint area of the plume at different water depths is calculated from the initial dispersion area, increasing at the horizontal dispersion rate over the elapsed time for the plume to reach that depth; and

- The estimate of SSC at different elevations is found by dividing the sediment mass in suspension at a given water depth (the product of the sediment release rate and the duration of the impact, divided by the water depth) by the representative plume volume at that depth (horizontal footprint area at that depth x 1 m).

The numerical model for sediment deposition thickness resulting from the release of sands and gravels is constructed as follows:

- The area over which sediment is deposited depends on the lateral spreading of the sediment plume footprint with depth, but also with tidal variation in current speed and direction, including the possibility of flow reversal. This is an important factor if the release occurs for more than tens of minutes as it affects the distance and direction which the plume is advected from the source;

- The width of the footprint of (instantaneous) deposition onto the seabed is estimated as the square root of the near-bed plume footprint area (calculated using the model for SSC above).

When drilling anchor piles, the point of sediment release is likely to be static and so the width of deposition is characterised based on the footprint of release and a small amount of lateral dispersion between surface and seabed prior to deposition;

- The length of the footprint of deposition onto the seabed over multiple tidal cycles is estimated as twice the advected distance of the plume at the representative current speed, representing the maximum length over consecutive flood and ebb tides. If the operation lasts less than 12.4 hours (one full tidal cycle), the length is reduced proportionally;
- The average seabed deposition thickness is calculated as the total volume of sediment released, divided by the footprint area (width times length) of deposition; and
- This model provides a conservative estimate of deposition thickness as it assumes that the whole sediment volume is deposited locally in a relatively narrow corridor. In practice, the deposition footprint on the seabed will probably be normally wider and frequently longer than is assumed, and the proportion of all sediment deposited locally will vary with the distribution in grain size (leading to a greater area but a correspondingly smaller average thickness).

The numerical model for SSC resulting from dispersion of fine sediment is constructed on the basis of the initial dispersion into the receiving waters, and then further dispersion of the plume as a whole, as per the following example for overspill for a trailing suction hopper dredger (TSHD):

- The vessel is likely to be stationary during precision dredging operations so the water movement relative to the vessel is dominantly tidal (at the representative current speed 0.5 m/s);
- Sediment is discharged at a representative rate (e.g. 30 kg/s for dredging over-spill) into a minimum volume of water 100 m^3 = 10 m x 10 m x 1 m deep;
- This volume of water will be refreshed every 20 seconds (10 m / 0.5 m/s);
- The total sediment input is 20 s x 30 kg/s = 600 kg;
- The resulting initial concentration in the receiving water is 600 kg / 100 m^3 = 6 kg/m^3 = 6,000 mg/l;
- The initial concentration plume would then be subject to turbulent dispersion both laterally and vertically. Given the starting mass of sediment and water volume above, levels of SSC will vary rapidly in proportion to the dilution of the same sediment mass as the plume dimensions and volume increase; and
- Assuming a faster current speed, faster vessel motion or larger footprint of release would reduce the mass of sediment introduced to the fixed volume of the receiving waters (and so SSC) at the point of initial dispersion, and *vice versa*.

2.4 Description of activities causing sediment disturbance

2.4.1 Drilling of monopile foundations and pin piles for jacket foundations

Overview

Monopile foundations and pin piles for jacket foundations will be installed into the seabed using standard piling techniques. In some locations, the particular geology may present some obstacle to piling, in which case, some or all of the seabed material might be drilled from within the pile footprint to assist in the piling process.

The impact of drilling operations mainly relates to the release of drilling spoil at or above the water surface which will put sediment into suspension and the subsequent re-deposition of that material to the seabed. The nature of this disturbance will be mostly determined by the rate and total volume of material to be drilled, the seabed and subsoil material type, and the drilling method (affecting the texture and grain size distribution of the drill spoil). The environmental conditions (total water depth, current speed and direction) over the period of active drilling will also affect the dimensions and concentration pattern of the plume and any subsequent deposition, to some extent; however, such conditions are likely to be continuously varying over time.

Evidence base

The evidence-base does not presently include many measurements of SSC resulting from drilling operations for monopile or pin pile installation. This is due to the relatively small number of occasions that such works have been necessary and the likely limited nature (extent, duration and SSC magnitude) of actual measurable impacts in practice.

Limited evidence from the field is provided by the during- and post-construction monitoring of monopile installation using drill-drive methods into chalk at the Lynn and Inner Dowsing offshore wind farms (CREL, 2008). Stiff London Clay is also present in the Five Estuaries Offshore Array Areas, it is recognised that the geological properties of the consolidated material (London Clay or chalk), the foundation dimensions and drilling apparatus will differ to some degree. In the Five Estuaries Offshore Array Areas, it is also not yet known how the drilled sub-soils will disaggregate as a result of the final chosen tools and method for drilling if and where needed. All of the above factors limit the extent to which the Lynn and Inner Dowsing monitoring evidence can be considered to be indicative of the proposed construction activities for Five Estuaries.

The installation of steel monopiles (4.7m diameter and up to 20m penetration depth) was assisted in some cases by a drill-drive methodology. The drill arisings were mainly in the form of rock (chalk) chippings that were released onto the seabed a short distance away in a controlled manner using a pumped riser. The particular concern in that case was the possibility of sub-surface chalk arisings leading to high levels of SSC of an atypical sediment type. The result of sediment trap monitoring (located as close as 100m from the operation) was that the chalk was not observed to collect in significant quantities. However, direct measurements of SSC were not possible at the time of the operation.

The dimensions of the chalk drill arisings deposit created was measured by geophysical survey and characterised as a conical mound, approximately 3m thick at the peak, extending laterally (from the peak to ambient bed level) up to 10m in what is assumed the downstream direction and 5m in the other. The volume of the deposit (measured as approximately 290m^3) was similar to the total volume of the drilled hole (347m^3) indicating that the majority of the total drill arisings volume had been deposited locally. The difference in volumes might be partially explained by different patterns of settling or transport leading to some material settling away from the main deposit location. It is also possible that the combination of drill and drive did not necessarily release a volume of material equivalent to 100% of the internal volume of the pile, or that the full burial depth may not have been achieved in this example. Seabed photographs indicate that the material in the deposit is clearly horizontally graded, with the largest clasts closer to the centroid of the deposit.

2.4.2 Seabed preparation by dredging prior to foundation and cable installation

Overview

To provide a stable footing for jacket foundations, standard dredging techniques may be used to remove or lower the level of the mobile seabed sediment veneer within a footprint slightly larger than the foundation base. Dredging may also be used to reduce the level of sandwaves where they are present in the footprint of foundations and in a narrow corridor where they intersect array, interconnector and export cable routes in the Offshore Array Areas. There are localised areas of sandwaves present in the export cable corridor as evidenced in [the Baseline Appendix Report].

Dredging has the potential to cause elevated SSC by, sediment over-spill at the water surface during dredging and by the subsequent release of the dredged material from the dredger during spoil disposal at a nearby location. The subsequent settlement of the sediment disturbed by dredging will lead to sediment accumulation of varying thickness and extent on the seabed. These changes are quantitatively characterised in this section using spreadsheet based numerical models.

Evidence base

The evidence-base with regards to dredging and elevated levels of SSC is broad and well established through a variety of monitoring and numerical modelling studies. The following text from the UK Marine SAC Project is representative of the wider evidence base.

"Dredging activities often generate no more increased suspended sediments than commercial shipping operations, bottom fishing or generated during severe storms (Parr et al., 1998). Furthermore, natural events such as storms, floods and large tides can increase suspended sediments over much larger areas, for longer periods than dredging operations (Environment Canada, 1994). It is therefore often very difficult to distinguish the environmental effects of dredging from those resulting from natural processes or normal navigation activities (Pennekamp et al., 1996).

…In general, the effects of suspended sediments and turbidity are generally short term (<1 week after activity) and near-field (<1 km from activity). There generally only needs to be concern if sensitive species are located in the vicinity of the maintained channel."

Dredging for construction aggregates is a common marine activity on the east coast of UK. The total mass of aggregate recovered from each region is reported annually by the British Marine Aggregate Producers Association (BMAPA, 2021). It is reported that, in 2020, approximately 1.35 million tonnes (0.85 million m^3) of construction aggregate were dredged from a total permitted licensed tonnage of approximately 3.6 million (2.26 million m^3) in the Thames Estuary region (within approximately 5 separate license area locations within the wider study area).

In comparison, the total volume of sediment that could potentially be dredged for foundation preparation in the Five Estuaries Offshore Array Areas is 1,193,598 m^3 (60 x 60 x 4 m for 79 WTG type gravity jacket foundations and 70 x 100 x 4 m for 2 Offshore Substation Platform (OSP) jacket foundations) over the whole duration of the construction period (equivalent to approximately 140 % of the annual volume of aggregate material actually recently extracted from licenced areas in the Thames Estuary region). The total volume of sediment that could potentially be dredged as part of sandwave clearance or levelling is up to 35 million m^3 in the Five Estuaries Offshore Array Areas and up to 64.75 million m^3 in the Five Estuaries Offshore ECC over the whole duration of the construction period. These estimates are highly conservative at this stage. Whilst seabed preparation and sandwave levelling represent a potentially large volume of sediment, the dredged material will be returned to the seabed and retained within the local sedimentary system, unlike marine aggregate dredging where the material is deliberately removed.

It is also noted that sediment dredged as part of construction activities for Five Estuaries will all be returned to the seabed nearby to the dredging location, whereas sediment dredged as part of aggregate extraction is removed permanently from the seabed.

2.4.3 Cable burial

Overview

The impact of cable burial operations mainly relates to a localised and temporary re-suspension and subsequent settling of sediments (BERR, 2008). The exact nature of this disturbance will be determined by the soil conditions within the Five Estuaries Offshore Array Areas and Offshore Cable Corridor, the length of installed cable, the burial depth and burial method. These changes are quantitatively characterised in this section for export, array and substation interconnector cables.

The impact of dredging sandwaves as part of cable burial is assessed in Section 2.4.2. There are localised areas of sandwaves present in the export cable corridor as evidenced in Volume 4, Annex 2.1 – Physical Processes Baseline Technical Report.

Evidence base

The evidence base with respect to cable burial activities is broad and includes a range of theoretical, numerical modelling and monitoring studies considering a range of installation methodologies, sediment types, water depths and other environmental conditions. The evidence base is widely applicable as the dimensions of the cables, the installation techniques used and the target depths of burial do not vary significantly with the scale of the development (small or large wind farm arrays) or the type of cable being installed (wind farm export, array or inter-connector cables, or non-wind farm electrical and communications cables).

SSC monitoring during cable laying operations has been undertaken at Nysted Wind Farm (ABPmer et al., 2007; BERR, 2008). During the works, both jetting and trenching were used, where the latter method involves pre-trenching and back-filling using back-hoe dredgers. Superficial sediments within the site were predominantly medium sands, approximately 0.5m to 3m in thickness, underlain by clay. SSC was recorded at a distance of 200m from jetting and trenching activities and the following values were observed:

- trenching – mean (14mg/l) and max (75mg/l); and
- jetting – mean (2mg/l) and max (18mg/l).

The higher sediment concentrations from the trenching activities were considered to be a result of the larger volume of seabed strata disturbed during operations and the fact that the material disturbed during trenching was lifted to the surface for inspection. This meant that the sediment was transported through the full water column before being placed alongside the trench (BERR, 2008).

Cable laying monitoring also took place at Kentish Flats where ploughing methods were used to install three export cables (EMU Limited, 2005). Cefas agreed pre-defined threshold limits against which SSC monitoring would be compared. The monitoring 500m down-tide (where the concentrations would be greatest) of the cable laying activities showed:

- marginal, short-term increases in background levels (approximately nine times increase to the background concentrations); and

- peak concentrations occasionally reaching 140mg/l (equivalent to peaks in the naturally occurring background concentrations).

The observations at Nysted and Kentish Flats provide confidence that cable laying activities do not create a long-term, significant disruption to the background sediment concentrations. Furthermore, it also illustrates that there is little sediment dispersal, indicating that there is unlikely to be much deposition on the seabed other than immediately adjacent to the cable route.

Reach (2007) describes plume dispersion studies for a cable laying jetting operation in Hong Kong with an assumption that 20% of a trench cross-section of 1.75m² would be disturbed by the jetting process and the speed of the jetting machine would be 300m/hour (0.083m/s). ASA (2005) describes similar studies for a cable laying operation near Cape Cod in the USA and assumed that 30% of a trench cross-section of 3m² would be disturbed by the jetting process and the speed of the jetting machine would be 91m/hour (0.025m/s). This latter study also assumed that any sand particles would quickly return to the bed and only the fine sediment particles (particles with a diameter less than 63µm) would form a plume in the water column.

SeaScape Energy (2008) describes cable installation plume dispersion monitoring studies carried out at the Burbo offshore wind farm in Liverpool Bay, UK.

- three export cables were installed to a target depth of approximately 3m by vertical injector ploughing while array cables were installed to a similar depth by jetting assisted ploughing.
- the monitoring demonstrated clearly that both cable installation techniques had only small scale impacts on localised SSC. Changes were measurable to a few hundreds of metres only and suspended sediment levels were not elevated more than five times background. Suspended sediment levels never approached the threshold level (3,000mg/l) agreed with regulatory authorities beforehand, even in very close proximity to the works (less than 50m).
- local changes in SSC over a relatively fine sediment seabed area (most likely to lead to plume impacts) was in the region of 250 to 300mg/l within 200m of the operation, falling to the measured baseline level (100mg/l) by 700m downstream. It is assumed, therefore, that coarser sediments were associated with even lower levels.

The post-burial impacts of cable burial on sandy seabed morphology were also considered by BERR (2008) with reference to a wide range of desktop and monitoring studies. The report concludes that impacts will also be limited in terms of both the thickness of re-deposited sediments and the potential for affecting the surficial sediment type:

"The low levels of sediment that are mobilised during cable laying mean that there will be only low levels of deposition around the cable route. The finer material will generally remain in suspension for longer but will settle and remobilise on each tide with no measurable material left in place. Coarser sediments are expected to settle within a few metres of the cable route and following disturbance is likely to recover rapidly, given similar communities in the vicinity." (BERR, 2008).

2.4.4 Drilling fluid release during HDD at the landfall

Horizontal Directional Drilling (HDD) is the preferred option to transition the Five Estuaries offshore export cable to the onshore grid at a landfall between Frinton-on-Sea and Holland Haven, on the Essex coast (Figure 1). The drill punch-out location will be either in the intertidal area, or below the lowest astronomical tidal water level. The drill length may be up to approximately 1,100m, with a diameter of up to approximately 1.3m.

The release of drilling fluid (a suspension of natural bentonite clay in water) into the coastal waters at the punch-out location may cause a sediment plume in the nearshore area.

Up to 5 HDD conduits might be required, with up to 4,940m^3 of drilling fluid potentially released per conduit (up to 24,700m^3 total for all conduits). Lesser amounts are more likely, depending on the final drilling method, length and diameter required.

Drilling fluid is a composite made of bentonite and water with the following functions:

- to remove cuttings from in front of the drill bit;
- power the mud motor;
- to transport cuttings from the drill face through the annular space towards the surface;
- lubricate the drill string during drilling phases and HDPE strings during pull back;
- cooling the reamers (cutting tools);
- hole stabilisation; and
- creation of a filter cake against the wall of the hole to minimize the risk of loss of drilling fluid or influx of groundwater penetration into the borehole.

The drilling fluid typically consists of a low concentration bentonite – water mixture. Depending on the formation to be drilled through, the concentration is typically between 13 litres (30kg) and 35 litres (80kg) of dry bentonite clay per m^3 of water (30,000 to 80,000mg/l).

The use of bentonite has limited potential to cause environmental impacts:

- it is a natural material, so has no chemical constituents;
- it is recyclable;
- it is on the OSPAR List of Substances Used and Discharged Offshore which Are Considered to Pose Little or No Risk to the Environment (PLONOR); and
- owing to the large diameter pipe and long length, the total volume of fluid used may be relatively large, but, owing to the low concentration, the total amount of bentonite used is limited.

At the point of 'punch out' some of the total volume of drilling fluid may be released into the surrounding seawater by residual pressure in the system, and further movement of the equipment. The size of the plume will be initially very small in extent and localised to the end of the drill bit and borehole (order of a few metres diameter); the SSC of the undiluted drilling fluid at this point will be very high (30,000 to 80,000mg/l). The free end of the plume will be advected (transported passively) at the speed and direction of the ambient tidal current at the time of the release and the narrow plume will gradually grow in length for the (limited) period of time that drilling fluid continues to be released.

The plume will be subject to turbulent dispersion over time and distance as it is advected. The width and the height of the plume will gradually increase, but the SCC within the plume will rapidly decrease in proportion to the increase in volume.

Bentonite clay grains are very small and so are likely to stay in suspension for long periods of time (days to weeks or longer) in the relatively turbulent marine environment. As a result, the Bentonite clay in the drilling fluid is expected to become progressively dispersed to very low concentrations (not measurably different from ambient natural turbidity levels) over periods of hours to days, and will therefore not settle or accumulate onto the seabed in measurable thickness in any location more than a few tens of metres from the main point of release.

2.5 Cumulative changes

Overview

A Cumulative Effects Assessment (CEA) has been undertaken to consider the impact associated with Five Estuaries together with other projects and plans. Each project on the CEA long list[1] has been considered on a case-by-case basis for scoping in or out of the coastal processes chapter, based upon data confidence, effect-receptor pathways and the spatial/temporal scales involved.

In terms of the potential for cumulative changes to SSC, bed levels and sediment type, the screening approach described above was informed using modelled spring tidal excursion ellipses. This is because meaningful sediment plume interaction generally only has the potential to occur if the activities generating the sediment plumes are located within one spring tidal excursion ellipse from one another and occur at the same time.

Given the length and orientation of tidal excursion ellipses in the vicinity of Five Estuaries, it is the case that the potential for sediment plume interaction will be limited to instances in which Five Estuaries construction activities occur simultaneously with:

- construction activities in the proposed North Falls offshore wind farm; and
- aggregation extraction operations.

It is considered unlikely that activities causing sediment disturbance will occur in the adjacent operational Galloper and Gabbard Bank offshore wind farms, in a manner that would cause cumulative impacts, during the installation phase of Five Estuaries. Galloper and Gabbard Bank offshore wind farms are already constructed, so will not foreseeably require large scale dredging, drilling or trenching works; minor repairs and maintenance do not disturb large volumes of sediment.

It is also generally unlikely that activities in the Five Estuaries and Galloper or Gabbard or North Falls developments would be sufficiently closely aligned with respect to the tidal axis, and close enough in distance for overlapping plumes to occur. If such overlapping or interacting plumes should occur, they will be similar in nature or magnitude to that described above for other cumulative scenarios. The potential for cumulative change is discussed in this section.

Five Estuaries and North Falls offshore wind farm construction activities

The North Falls array areas and ECC are located to the side (to the west) of the VE Offshore Array Areas with respect to the tidal axis, which means that overlap and interaction between plumes created by activities at North Falls and activities in the Offshore Array Areas are very unlikely.

Overlap and interaction between plumes created in the ECCs for the two projects are possible, but only if actual active trenching work is conducted simultaneously and at locations that are aligned with respect to the tidal axis, which is very unlikely.

Five Estuaries and other aggregate dredging activities

Only a small number of active aggregate dredging license areas (namely, Areas: 524; 507/1/4; 508; 509/1/2/3; 510/1/2; 524; 528/2) are sufficiently close to the Five Estuaries project (within one tidal excursion distance) that an overlapping plume effect is at all likely.

With the exception of Area 524, the aggregate dredging sites are located to the side of the Offshore Array Areas with respect to the tidal axis, which means that overlap and interaction between plumes

[1] Volume 1, Annex 3.1: Cumulative Effects Assessment, sets out the full definition of the tiers. Tier 1: high level of certainty or information availability (including under construction or where a planning application has been approved or is awaiting decision). Tier 2: medium level of certainty or information (such as developments on PINS Programme of Projects where a Scoping Report has been submitted). Tier 3: low level of certainty or information available (no planning applications submitted or identified for potential future development only).

created by aggregate dredging and activities in the Offshore Array Areas are very unlikely. Some overlap of plumes might occur in relation to export cable burial in the central section of the export cable corridor only, however, as assessed in Sections 2.4.3 and 2.6, the extent and duration of sediment plumes from cable burial are very limited.

Any cumulative increase in either the spatial footprint or peak concentration of sediment plumes are therefore likely to be indistinguishable from background levels. Any associated cumulative changes in bed level (different to that already assessed for Five Estuaries alone) are also unlikely to be measurable in practice.

2.6 Assessment of changes to suspended sediment concentrations, bed levels and sediment type

This section provides a description of the realistically possible combinations of magnitude and extent of impact for local increases in SSC and seabed deposition, due to sediment disturbance potentially caused by:

- drilling of monopile foundations and pin piles for jacket foundations;
- seabed preparation by dredging prior to jacket suction bucket foundation installation;
- sandwave clearance (prior to cable burial);
- cable burial; and
- drilling fluid release during HDD at the landfall.

The actual magnitude and extent of such impacts will depend in practice on a range of factors, such as the actual total volumes and rates of sediment disturbance, the local water depth and current speed at the time of the activity, the local sediment type and grain size distribution, the local seabed topography and slopes, etc. There will be a wide range of possible combinations of these factors and so it is not possible to predict specific dimensions with complete certainty. To provide a robust assessment, a range of realistic combinations have been considered, based on conservatively representative location (environmental) and project (MDS) specific information, including a range of water depths, heights of sediment ejection/initial resuspension, and sediment types.

This wider range of results can be summarised broadly in terms of four main zones of effect, based on the distance from the activity causing sediment disturbance. These zones are consistent with the results of observational (monitoring) evidence and recent numerical modelling of analogous activities (e.g. BERR, 2008; TEDA, 2010; Navitus Bay Development Ltd, 2014; Awel y Môr Offshore Wind Farm Ltd, 2022):

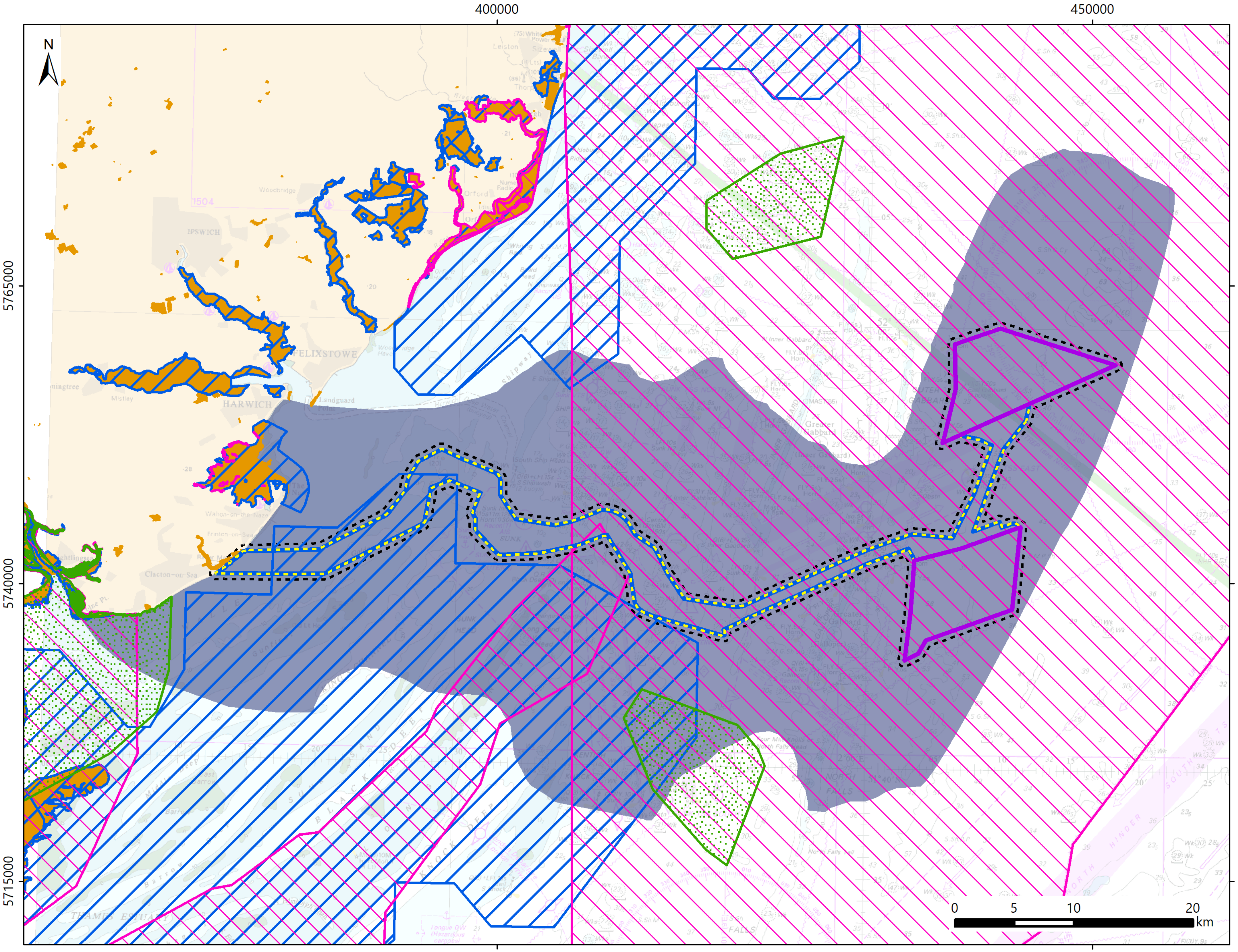
- 0 to 50m – zone of highest SSC increase and greatest likely thickness of deposition. All gravel sized sediment likely deposited in this zone, also a large proportion of sands that are not resuspended high into the water column, and also most or all dredge spoil in the active phase. Plume dimensions and SSC, and deposit extent and thickness, are primarily controlled by the volume of sediment released and the manner in which the deposit settles.
  - at the time of active disturbance - very high SSC increase (tens to hundreds of thousands of mg/l) lasting for the duration of active disturbance plus up to 30 minutes following end of disturbance; sands and gravels may deposit in local thicknesses of tens of centimetres to several metres; fine sediment is unlikely to deposit in measurable thickness.
  - more than one hour after the end of active disturbance – no change to SSC; no measurable ongoing deposition.

- 50 to 500m – zone of measurable SSC increase and measurable but lesser thickness of deposition. Mainly sands that are released or resuspended higher in the water column and resettling to the seabed whilst being advected by ambient tidal currents. Plume dimensions and SSC, and deposit extent and thickness, are primarily controlled by the volume of sediment released, the height of resuspension or release above the seabed, and the ambient current speed and direction at the time.

  - at the time of active disturbance - high SSC increase (hundreds to low thousands of mg/l) lasting for the duration of active disturbance plus up to 30 minutes following end of disturbance; sands and gravels may deposit in local thicknesses of up to tens of centimetres; fine sediment is unlikely to deposit in measurable thickness.
  - more than one hour after end of active disturbance – no change to SSC; no measurable ongoing deposition.

- 500m to the tidal excursion buffer distance – zone of lesser but measurable SSC increase and no measurable thickness of deposition. Mainly fines that are maintained in suspension for more than one tidal cycle and are advected by ambient tidal currents. Plume dimensions and SSC are primarily controlled by the volume of sediment released, the patterns of current speed and direction at the place and time of release and where the plume moves to over the following 24 hours.
  - at the time of active disturbance – low to intermediate SSC increase (tens to low hundreds of mg/l) as a result of any remaining fines in suspension, only within a narrow plume (tens to a few hundreds of metres wide, SSC decreasing rapidly by dispersion to ambient values within one day after the end of active disturbance; fine sediment is unlikely to deposit in measurable thickness.
  - one to six hours after end of active disturbance – decreasing to low SSC increase (tens of mg/l); fine sediment is unlikely to deposit in measurable thickness.
  - six to 24 hours after end of active disturbance – decreasing gradually through dispersion to background SSC (no measurable local increase); fine sediment is unlikely to deposit in measurable thickness. No measurable change from baseline SSC after 24 to 48 hours following cessation of activities.
- beyond the tidal excursion buffer distance or anywhere not tidally aligned to the active sediment disturbance activity – there is no expected impact or change to SSC nor a measurable sediment deposition.

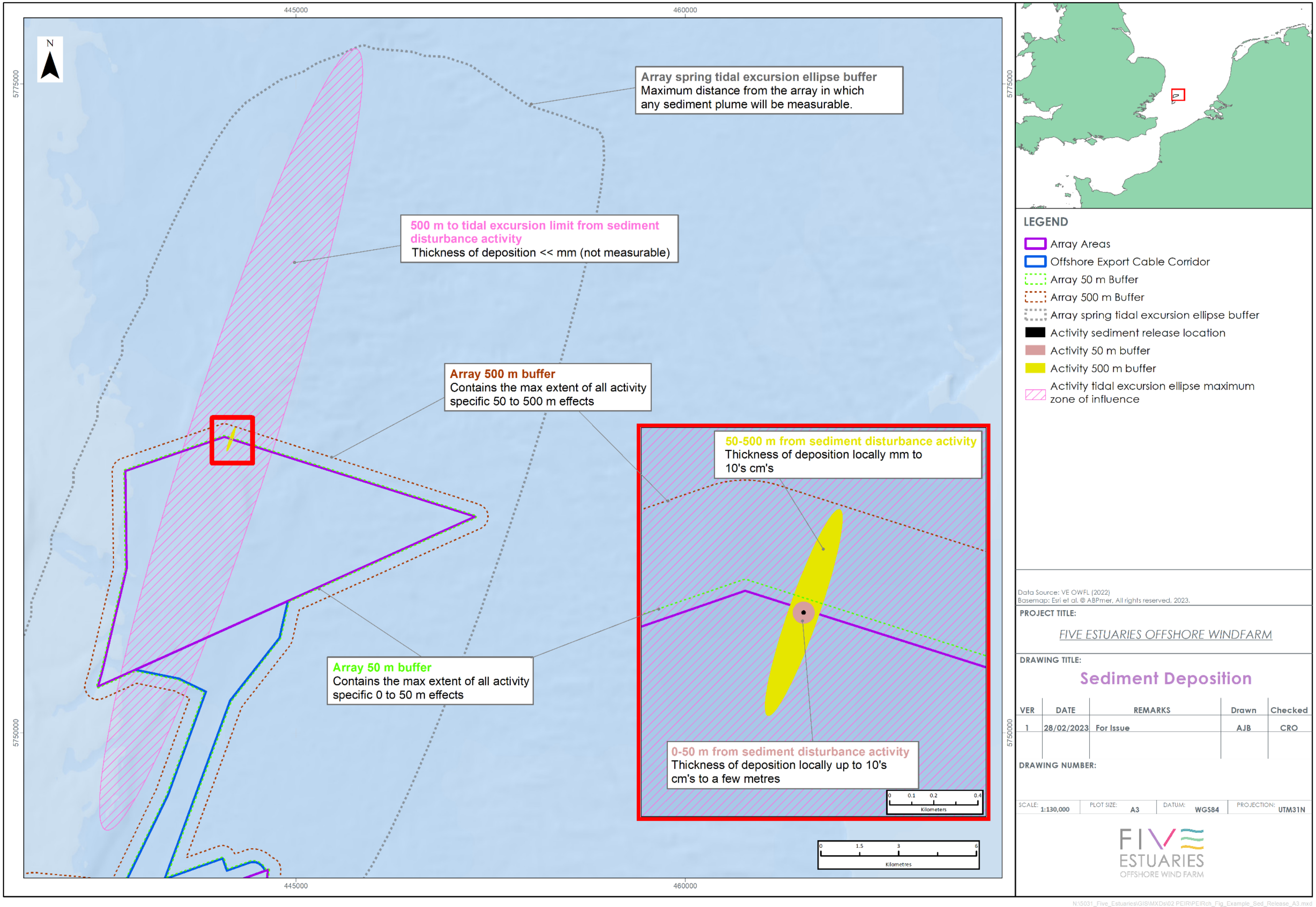
Figure 2 provides a summary of the maximum spatial extent of these zones in relation to the whole of the Five Estuaries Offshore Array Area and ECC, and in relation to selected receptors in the surrounding area. Figure 3 provides an example schematic illustration of the footprint of effect for a single occurrence of an activity causing local sediment disturbance. In practice the MDS impact will be a limited number of discrete areas of effect (similar to that shown in the example), separated by areas of lesser impact.



LEGEND

- Array Areas
- Offshore Export Cable Corridor
- 50m Buffer
- 500m Buffer
- Marine Conservation Zone
- Special Area of Conservation
- Special Protection Area
- Site of Special Scientific Interest
- Spring tidal excursion ellipse buffer

Date	By	QA
Feb23	CRO	NKD
Coordinate System WGS 1984 UTM Zone 31N		
Projection Transverse Mercator		
Project no. 5031 PEIR_Fig_Buffers_TechAnnex.mxd		



445000
460000
5775000
5750000
N
Array spring tidal excursion ellipse buffer
Maximum distance from the array in which any sediment plume will be measurable.
500 m to tidal excursion limit from sediment disturbance activity
Thickness of deposition << mm (not measurable)
Array 500 m buffer
Contains the max extent of all activity specific 50 to 500 m effects
Array 50 m buffer
Contains the max extent of all activity specific 0 to 50 m effects
50-500 m from sediment disturbance activity
Thickness of deposition locally mm to 10's cm's
0-50 m from sediment disturbance activity
Thickness of deposition locally up to 10's cm's to a few metres
0 0.1 0.2 0.4
Kilometers
0 1.5 3 6
Kilometres
LEGEND
Array Areas
Offshore Export Cable Corridor
Array 50 m Buffer
Array 500 m Buffer
Array spring tidal excursion ellipse buffer
Activity sediment release location
Activity 50 m buffer
Activity 500 m buffer
Activity tidal excursion ellipse maximum zone of influence
Data Source: VE OWFL (2022)
Basemap: Esri et al. © ABPmer, All rights reserved, 2023.
PROJECT TITLE:
FIVE ESTUARIES OFFSHORE WINDFARM
DRAWING TITLE:
Sediment Deposition
VER | DATE | REMARKS | Drawn | Checked
1 | 28/02/2023 | For Issue | AJB | CRO
DRAWING NUMBER:
SCALE: 1:130,000
PLOT SIZE: A3
DATUM: WGS84
PROJECTION: UTM31N
FIVE ESTUARIES OFFSHORE WIND FARM
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3 Changes to the Wave Regime

3.1 Overview

This section sets out the assessment of changes to the wave regime within the study area, based on spectral wave modelling of the PEIR MDS for blockage due to foundations within the VE array.

The wave model has been built using the MIKE21FM Spectral Wave (SW) module, which simulates the development, propagation and dispersion of wave energy throughout the model domain.

More detailed information about the design and validation of the wave model may be found in Volume 4, Annex 2.2: Physical Processes Model Design and Validation.

The wave model creates discrete simulations of wave height, period and direction throughout the domain, for a representative range of selected every day and extreme wave conditions (return periods and directions). The wave condition scenarios considered by the model for the assessment are:

- Wave coming directions (from N, NE, E, SE, and S); and
- Return periods (50% non-exceedance, 0.1 yr; 1 yr; 10 yr; 50 yr; 100 yr).

The details of each condition as defined in a central location of the eastern offshore wave boundary, approximately 10 km east of VE, are presented in Table 1.

Table 1. Wave and wind boundary conditions for each of the directional return period seastate conditions tested

Directional Sector	Case (Return Period)	Significant Wave Height (m)	Peak Wave Period (Tp, s)	Mean Wave Direction (°N)	Wind Speed @10 m (m/s)	Wind Direction (°N)
N	50% no exc	0.9	3.8	0	7.2	0
	0.1 yr RP	2.5	6.4	0	14.1	0
	1 yr RP	4.1	8.2	0	19.5	0
	10 yr RP	6.0	9.9	0	24.1	0
	50 yr RP	6.8	10.6	0	26.3	0
	100 yr RP	7.0	10.8	0	26.9	0
NE	50% no exc	1.0	4.0	45	7.6	45
	0.1 yr RP	1.9	5.6	45	11.8	45
	1 yr RP	3.0	7.1	45	15.8	45
	10 yr RP	4.7	8.8	45	21.3	45
	50 yr RP	5.5	9.5	45	23.1	45
	100 yr RP	5.8	9.7	45	24.1	45
E	50% no exc	0.9	3.7	90	7.0	90
	0.1 yr RP	1.5	4.7	90	10.6	90
	1 yr RP	2.2	5.8	90	13.1	90
	10 yr RP	3.5	7.3	90	17.4	90
	50 yr RP	4.0	7.8	90	19.0	90
	100 yr RP	4.2	7.9	90	19.5	90

Directional Sector	Case (Return Period)	Significant Wave Height (m)	Peak Wave Period (Tp, s)	Mean Wave Direction (°N)	Wind Speed @10 m (m/s)	Wind Direction (°N)
SE	50% no exc	0.8	3.3	135	6.2	135
	0.1 yr RP	1.5	4.4	135	10.6	135
	1 yr RP	2.1	5.4	135	12.7	135
	10 yr RP	3.9	7.2	135	18.4	135
	50 yr RP	4.7	8.0	135	21.1	135
	100 yr RP	5.0	8.2	135	21.7	135
S	50% no exc	1.0	3.6	180	7.6	180
	0.1 yr RP	2.3	5.5	180	13.5	180
	1 yr RP	3.8	7.1	180	18.5	180
	10 yr RP	5.6	8.7	180	23.3	180
	50 yr RP	6.0	8.9	180	24.1	180
	100 yr RP	6.1	9.0	180	24.3	180

3.2 Baseline conditions

Plots showing the spatial distribution of wave height and direction for each of the baseline wave conditions without any wind farm infrastructure present are shown in Appendix A (Figure A1 to Figure A5).

As shown in the following assessment section and confirmed in baseline model scenarios that include the effect of nearby operational windfarms (not shown), the monopile foundations installed in the nearby operational windfarms cause no measurable difference (<2.5 % wave height, <0.1 s wave period and <3 deg wave direction) to the natural baseline condition (excluding the effect of any wind farm infrastructure).

3.3 Assessment

3.3.1 Five Estuaries Wind Farm foundation type and number

The VE design envelope includes a range of wind turbine generator (WTG) and OSP foundation types, numbers and dimensions. The MDS is identified as the combination of options presenting the greatest total potential blockage to waves passing through the array area.

The MDS for VE is:

- 79 x smaller WTGs on conical gravity bases
    - 55 m base diameter at seabed, tapering to 15 m at sea surface;
    - Scour protection 2 m high, 137.5 m diameter, tapering to seabed level with 1:2 slope, total diameter 146 m; and
    - Combined equivalent blockage width 41.5 m per foundation.
- 2 x OSP on jacket foundations with suction buckets
    - 6 legs, 3.5 m diameter;
    - Cross bracing, 1.5 m diameter;
    - Base dimensions at seabed 100 m x 60 m;
    - Suction bucket diameter 20 m, height above seabed 5 m;
    - Scour protection 2 m high, up to 80 m diameter around each leg; and
    - Combined equivalent blockage width 62.0 m per foundation.

Any other combination of foundation type and number would result in a smaller total blockage.

3.3.2 Other nearby operational wind farm foundation type and number

The actual built dimensions of individual foundations in a wind farm are not normally publicly listed in detail and, in any case, may vary slightly within an array to account for differences in water depth or ground conditions. Conservatively representative WTG foundation details for the nearby existing operational offshore wind farms are used as follows.

Galloper:

- 56 x WTGs on monopile foundations
  - 7.5 m diameter

Greater Gabbard:

- 140 x WTGs on monopile foundations
  - 6.3 m diameter

Gunfleet Sands (all phases):

- 50 x WTGs on monopile foundations
  - 5.0 m diameter

London Array:

- 175 x WTGs on monopile foundations
  - 5.7 m diameter

No scour protection is included in the above combined equivalent blockage width calculations for nearby existing operational offshore wind farms. The details of scour protection (height, slope, diameter, armour type, proportion and location of foundations applied to) are not publicly available. Although (very) conservatively included for VE, nearbed scour protection is unlikely to make a measurable change to local water depths or contribution to blockage of waves (or currents) for the purposes of this assessment.

3.3.3 Other nearby proposed wind farm foundation type and number

East Anglia TWO:

- Up to 75 x WTGs on gravity base foundations
  - 60 m base diameter at seabed, tapering to 15 m at sea surface;
  - Scour protection 2 m high, 150 m diameter;
  - Combined equivalent blockage width 44.7 m per foundation;
  - An assumed layout is used, with uniform similar spacing and orientation to the related East Anglia OWF site. Foundations are concentrated in the southern part of the array area, which is a MDS assumption for cumulative impacts with VE.

North Falls:

- Up to 71 x WTGs on gravity base foundations
  - 65 m base diameter at seabed, tapering to 15 m at sea surface;
  - Scour protection 2 m high, 162 m diameter;
  - Combined equivalent blockage width 47.9 m per foundation;
  - An assumed layout is used, with uniform similar spacing and orientation to the adjacent Greater Gabbard site.

3.3.4 Foundation layouts

For VE, the indicative layout pattern for smaller WTGs and areas of likely locations for the OSPs are used in conjunction with the MDS type and number of foundations (shown in Figure 4). This layout is considered to be realistically representative of any that might be eventually considered.

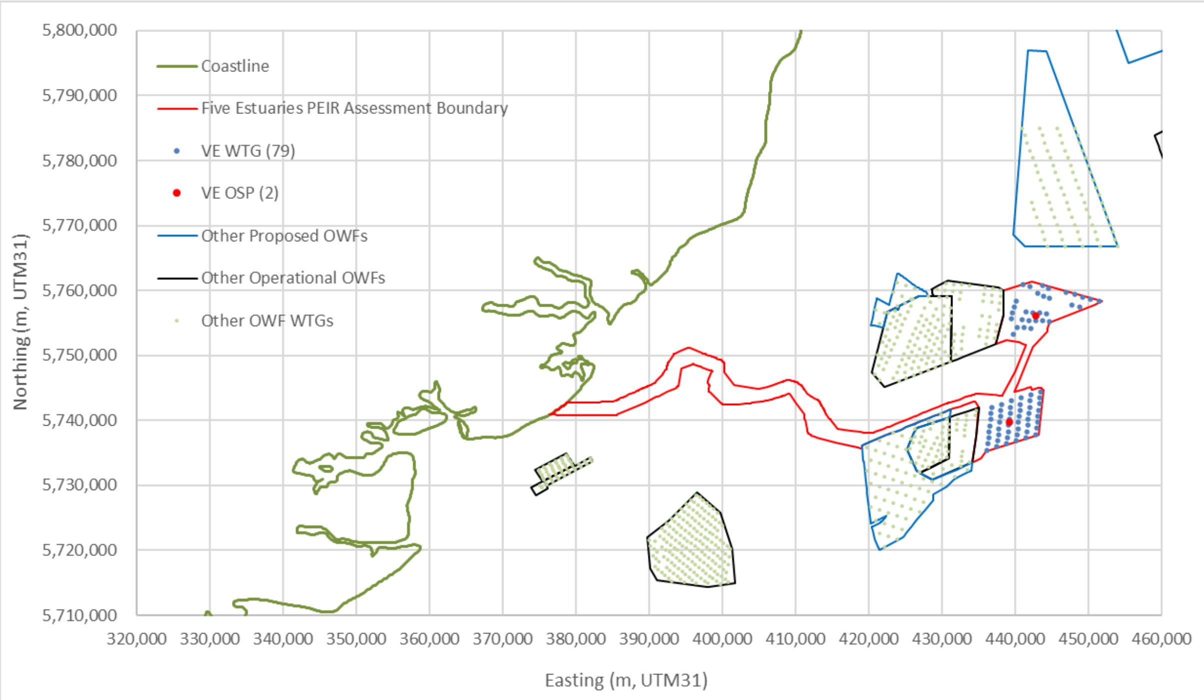

Figure 4. Layout of VE MDS foundations, also showing the as built foundation locations for the operational Galloper, Greater Gabbard, London Array and Gunfleet Sands OWFs, and indicative MDS foundation locations assumed for the proposed East Anglia TWO and North Falls OWFs.

The actual location of all foundations in other relevant nearby operational wind farms (namely Greater Gabbard, Galloper, London Array and Gunfleet Sands) are known and used directly in the model.

3.3.5 Changes to the wave regime

Plots showing the spatial distribution of changes to wave height for each of the baseline wave conditions as a result of MDS foundation type, number and layout for VE are shown in Appendix A (Figure A6 to Figure A10), and separately also including other nearby proposed and operational wind farms, (Figure A11 to Figure A15). In the figures, solid black lines show the array area extents for each included wind farm.

Changes less than 5 % of the baseline wave height would be indistinguishable from natural variability both within the seastate (difference between individual waves) and compared to normal rates of change (over timescales of one hour or less); such small differences would not be measurable in practice. Changes less than 2.5 % are also less than the reasonably expected accuracy of the model and so are excluded from the colour scale.

The images show that wave height is progressively decreased with distance through the array area in the direction from which the waves are coming. As a result, the maximum reduction in wave height is found downwind of individual WTGs in the central downwind part of the southern array area (5 to 7.5 %). The maximum reduction in a very localised and limited extent outside of the array area is only 2.5 to 5 % for the full range of wave directions and return periods considered. The scale of the change is dependent on the nature of the wave height/period condition, and the main direction of the wave energy with respect to the shape/thickness of the array and the alignment of the foundations. The maximum corresponding changes to wave period and wave direction (not shown) are less than 0.1 s and 3 deg respectively, at all locations, in all cases.

Wave height begins to recover immediately downwind of the array area. Recovery occurs mainly due to a wave energy spreading from areas to the side less or unaffected by interaction with the wind farm. For smaller seastates, recovery of the dominant wave condition can also occur as a result of ongoing wind energy input.

In the area where changes to wave height are greatest (typically within and immediately to the west or south-west of the array area), water depths are also relatively large (30 to 35 m Lowest Astronomical Tide (LAT)), with an additional 1.3 to 2.6 m depth depending on the state of the tide). In such water depths, a minimum wave period (approximately 6 s and larger in 30 m depth) is required to penetrate deeply enough to cause any water movement at the seabed. Even longer waves in conjunction with a sufficient wave height are needed to cause sufficient motion at the seabed to contribute to sediment transport.

As the wave period will not be affected (by more than 0.1 s), the ability of individual waves to reach the seabed will be unaffected. Where an individual wave is large enough to reach the seabed, the predicted change in wave height (proportional to the resulting amplitude of water movement) is locally only up to 5 to 10 %. The difference is therefore unlikely to result in a measurably different motion of water.

Differences in waves are very small in absolute and relative terms, and not measurable in practice (< 1 % wave height, <0.1 s wave period, <1 deg wave direction) at all other notable sedimentary and morphological features in the study area, including all sandbanks, nearshore areas (up to 5 km from the coast) and adjacent coastlines.

Further discussion of the results is provided in the PEIR impact assessment (Volume 2, Chapter 2.2: Marine Geology, Oceanography and Physical Processes).

4 Changes to the Tidal Regime

4.1 Overview

This section sets out the assessment of changes to the tidal regime within the study area, based on hydrodynamic modelling of the PEIR MDS for blockage due to foundations within the VE array.

The hydrodynamic model has been built using the MIKE21FM Hydrodynamic (HD) module, which simulates the development, propagation and dispersion of the tidal wave, and associated movements of water, throughout the model domain.

More detailed information about the design and validation of the hydrodynamic model may be found in Volume 4, Annex 2.2: Physical Processes Model Design and Validation.

The hydrodynamic model creates a continuous simulation of tidal water level, depth average current speed and current direction throughout the domain, for a representative spring-neap cycle (approximately mean neap and mean spring conditions) during the model validation period.

4.2 Baseline conditions

Plots showing the spatial distribution of current speed and direction for representative neap and spring conditions, during low water, peak flood, high water and peak ebb periods, without any wind farm infrastructure present are shown in Appendix B (Figure B1 and Figure B2).

As shown in the following assessment section and as confirmed in baseline model scenarios that include the effect of nearby operational windfarms (not shown), the monopile foundations installed in the nearby operational windfarms cause no measurable difference (<0.01 m water level, <0.01 m/s current speed and <1 deg current direction) at the resolution of the model (200 m) in the baseline condition. As discussed below in Section 4.3.2, although not explicitly resolved by the model, a very localised narrow wake is expected behind (and similar to the width of) individual foundations, associated with a small reduction in average current speed and a corresponding small increase in turbulence intensity. The wake signature will dissipate and recover with distance downstream, becoming indistinguishable to ambient conditions within tens to a few hundreds of metres. The dimensions and characteristics of such wake features were measured (by ABPmer) in the Burbo Offshore Wind Farm, reported in SeaScape Energy (2008).

4.3 Assessment

4.3.1 Five Estuaries and other operational wind farm foundation type, number and layout

The VE design envelope includes a range of WTG and OSP foundation types, numbers and dimensions. The MDS is identified as the combination of options presenting the greatest total potential blockage to water movement through the array area.

The PEIR MDS foundation type, number and layout for VE is the same as described and used for modelling changes to the wave regime in Section 3.3, with 79 smaller WTGs on conical gravity bases, and 2 OSP multi-leg jacket foundations with suction buckets. Any other combination of foundation type and number in the design envelope for PEIR would result in a smaller total blockage.

Conservatively representative WTG foundation type, dimensions and numbers are used for the nearby existing operational offshore wind farms, as described and used for modelling changes to the wave regime in Section 3.3. The actual location of all foundations in other nearby operational wind farms are known and used directly in the model (see Figure 4).

4.3.2 Changes to the tidal regime

Further discussion of the results is also provided in the ES impact assessment (Volume 2, Chapter 2.2: Marine Geology, Oceanography and Physical Processes).

Instantaneous current speed and direction

Plots showing the spatial distribution of changes to current speed for selected tidal conditions as a result of the PEIR MDS foundation type, number and layout for VE, are shown in Appendix B (Figure B3 and Figure B4).

The model has a relatively high spatial resolution of 200 m in the study area including VE and all of the nearby operational wind farms. The model accounts for the changes to currents caused by the individual foundations, and the propagation and recovery of wake features, at a similar scale. In practice, at a more local sub-grid scale (order of metres), slightly greater changes might be expected in the very local flow field of the individual foundations (e.g. a narrow wake approximately as wide as the foundation, and turbulent eddies in the order of tens of centimetres to a few metres within the wake footprint, all recovering to ambient conditions within the order of tens to a few hundreds of metres downstream).

Changes less than 5% of the baseline condition (i.e. approximately 0.13 m based on mean spring tidal range, 0.05 m/s current speed or 5 deg current direction) would be largely indistinguishable from frequent and high natural rates of change over various timescales, including: flood/ebb/slack; spring-neap; solstice-equinox; interannual/metonic cycle; and, variability in response to meteorological (surge) influence. Such small absolute differences would also not be accurately measurable in practice. Changes less than 0.01 m/s are also less than the reasonably expected accuracy of the model and so are excluded from the colour scale.

The result figures show that changes to current speed at the resolution of the model (at length scales greater than 200 m) will be less than 0.05 m/s, which is very small in both absolute and relative terms, within the range of natural variability, and not measurable in practice. Corresponding changes to current direction (not shown in the figures) are less than 1 deg.

Residual current speed and direction

The timeseries of current speed and direction throughout the model domain was also used to determine the residual current speed and direction (the long term drift rate and direction of water over one spring neap cycle). The results are equally valid for baseline conditions with or without the nearby other built wind farms which cause no measurable difference in currents (<0.01 m water level, <0.05 current speed locally and <1 deg current direction) at the resolution of the model (approximately 200 m). Consistent with the very limited scale of change in instantaneous current speed and direction described above as a result of the MDS foundation type, number and layout for VE (MDS foundation type, number and layout for VE), no measurable change in residual current speed or direction is predicted either within the VE array, or elsewhere.

A plot showing the spatial distribution of residual current speed and direction for selected tidal conditions are shown in Appendix B (Figure B5).

5 Changes to the Sediment Transport Regime

5.1 Overview

This section sets out the assessment of changes to the sediment transport regime, as a result of the assessed changes to the tidal regime within the study area, based on hydrodynamic and sediment transport modelling, including the PEIR MDS for blockage due to foundations within the VE array area. Other built windfarms present in the present-day baseline condition and also in the future 'with VE' scenario cause no net change in this assessment.

The hydrodynamic model has been built using the MIKE21FM Hydrodynamic (HD) module, which simulates the development, propagation and dispersion of the tidal wave, and associated movements of water, throughout the model domain. The sediment transport model has been built using the MIKE21FM Sand Transport (ST) module, which simulates a time series of total load transport rate and direction for 250 μm diameter quartz sand in response to the shear stress caused by the tidal currents described by the hydrodynamic model.

More detailed information about the design and validation of the hydrodynamic model and associated sediment transport model may be found in Volume 4, Annex 2.2: Physical Processes Model Design and Validation.

The hydrodynamic and sediment transport models together create a continuous simulation of tidal water level, depth average current speed and current direction and resulting instantaneous sediment transport rate throughout the domain, for a representative spring-neap cycle (approximately mean neap and mean spring conditions) during the model validation period.

5.2 Baseline Conditions

Further discussion and information about baseline sediment transport conditions and patterns is also provided in Volume 4, Annex 2.1: Physical Processes Baseline Technical Report.

5.2.1 Tidally driven sediment transport rate and direction

The baseline sediment transport model results were used to determine the instantaneous and residual sand transport rate and direction (the long term transport rate and direction for representative medium to fine sand over one spring neap cycle).

Plots showing the spatial distribution of baseline instantaneous sediment transport rate and direction for selected tidal conditions, and the residual sediment transport rate and direction over a representative spring-neap cycle, are shown in Appendix C (Figures C1, C2, C5 and C6).

5.2.2 Wave effects on sediment transport rate and direction

Sediment transport by waves alone in deep water results in a to-and-fro motion with minimal net transport. In conjunction with tidal currents, waves increase the overall rate of sediment transport, but the combined net transport rate and direction is largely controlled by the speed and direction of the coincident tidal current.

No explicit modelling of wave alone, or wave-current driven sediment transport has been undertaken on this basis.

5.3 Assessment

5.3.1 Changes to the tidally driven sediment transport rate

Additional hydrodynamic and sediment transport model results were created for a scenario including the PEIR MDS for blockage due to foundations within the VE array area (described in Section 4.3). The MDS results were compared with the corresponding baseline results to determine the impact of the foundations on instantaneous and residual sand transport rate and direction (the long term transport rate and direction for representative medium to fine sand over one spring neap cycle).

Plots showing the spatial distribution of absolute difference in instantaneous sediment transport rate for selected tidal conditions, and the absolute and relative difference in residual sediment transport rate over a representative spring-neap cycle, are shown in Appendix C (Figure C3, C4, and C7 to C10).

Consistent with the very limited scale of change in instantaneous current speed and direction described in Section 4.3.2 and associated images in Appendix B, no measurable change in residual sand transport rate or direction is predicted either within the VE array, or elsewhere, at the resolution of the model (approximately 200 m). Localised narrow wake features not resolved by the model may have a similarly localised effect on the texture (but not the morphology) of the seabed within their footprint; the wake is only likely to result in changes to seabed morphology immediately around the foundation base in the form of scour (described in Section 6).

Further discussion of these results is provided in the PEIR impact assessment (Volume 2, Chapter 2.2: Marine Geology, Oceanography and Physical Processes).

5.3.2 Changes to any wave driven component of the sediment transport rate

Wave model results quantifying the difference in wave height as a result of the PEIR MDS for blockage due to foundations within the VE array area are described in Section 3.3.5, and associated images in Appendix A.

The differences in wave height, period and direction described in Section 3.3.5. are small in absolute and relative terms and (as a small additional contribution to the tidally dominated transport) could only cause an even smaller change to overall instantaneous sediment transport rates or directions. The differences would not be measurable in practice and are easily within the range of natural variability in wave height from wave to wave, from hour to hour during the passage of a storm, and in the context of seasonal and interannual variation of wave climate.

Further discussion of these results is provided in the PEIR impact assessment (Volume 2, Chapter 2.2: Marine Geology, Oceanography and Physical Processes).

6 Scour and Seabed Alteration

6.1 Overview

The purpose of this section is to conservatively and quantifiably estimate the area of seabed that will be altered during the operational phase of the wind farm as a result of sediment scour that may develop adjacent to turbine foundations (in the absence of any scour protection).

The term scour refers here to the development of pits, troughs or other depressions in the seabed sediments around the base of turbine foundations. Scour is the result of net sediment removal over time (typically in the order of hours to days from installation in mobile sediments) due to the complex three-dimensional interaction between the foundation and ambient flows (currents and/or waves). Such interactions result in locally accelerated time-mean flow and locally elevated turbulence levels that enhance sediment transport potential in the area of influence. The resulting dimensions of the scour features and their rate of development are, generally, dependent upon the characteristics of the:

- Obstacle (dimensions, shape and orientation);
- Ambient flow (depth, magnitude, orientation and variation including tidal currents, waves, or combined conditions); and
- Seabed sediment (geotextural and geotechnical properties).

Based on the existing literature and evidence base, an equilibrium depth and pattern of scour can be empirically approximated for given combinations of these parameters. Natural variability in the above parameters means that the predicted equilibrium scour condition may also vary over time on, for example, spring-neap, seasonal or annual time-scales. The time required for the equilibrium scour condition to initially develop is also dependant on these parameters and may vary from hours to years.

Scour assessment for EIA purposes is considered here for three foundation types: monopiles; piled jacket foundations (a four-legged version); and gravity base foundation structures. Each foundation type may produce different scour patterns therefore monopiles, gravity base foundations and jacket foundations have all been considered. Suction caisson foundations (for monopods and jackets) have not been considered in the assessment below because these will fall within the envelope of change associated with the other three foundation types.

The concerns under consideration include the seabed area that may become modified from its natural state (potentially impacting sensitive receptors through habitat alteration) and the volume and rate of additional sediment resuspension, as a result of scour. The seabed area directly affected by scour may be modified from the baseline (pre-development) or ambient state in several ways, including:

- A different (coarser) surface sediment grain size distribution may develop due to winnowing of finer material by the more energetic flow within the scour pit;
- A different surface character will be present if scour protection (e.g. rock protection) is used;
- Seabed slopes may be locally steeper in the scour pit; and
- Flow speed and turbulence may be locally elevated.

The magnitude of any change will vary depending upon the foundation type, the local baseline oceanographic and sedimentary environments and the type of scour protection implemented (if needed). In some cases, the modified sediment character within a scour pit may not be so different from the surrounding seabed; however, changes relating to bed slope and elevated flow speed and turbulence close to the foundation are still likely to apply. No direct assessment is offered within this document as to the potential impact on sensitive ecological receptors.

The assessment presented here is not intended for use in detailed engineering design. However, methodologies similar to those recommended for the design of offshore wind foundations (e.g., DNV, 2016) have been used in some cases where they are applicable. The methods applied to assess scour are set out in Appendix A.

6.2 Baseline conditions

Where obstacles are not present on the seabed, normal sediment transport processes can cause spatial and temporal variations in seabed level and sediment character in the baseline environment. Scour is a similar but localised change resulting from particular local patterns of sediment transport. Scour may also occur in the baseline environment in response to natural obstacles such as rocky outcrops or boulders. Key features of the baseline environment pertinent to the assessment of scour due to the presence of wind farm infrastructure are summarised below:

- The VE array is characterised by the presence of coarse-grained unconsolidated sediments with both sand and sandy gravel particularly prevalent;
- Surficial sediment units are of variable thickness: the greatest thicknesses (typically 5 to 14 m, up to 15 to 17 m locally) are found on and around the slope between the shallower western part and deeper eastern part of each array area (Fugro, 2022a). In other large parts of the site, the thickness of erodible Holocene sediments is relatively thin or absent (typically limited to up to 0.5 to 2 m), and underlain by relatively erosion resistant pre-Holocene (London Clay) material; and
- Based on the prevalence and potential mobility of bedform features of varying scale in the available geophysical survey data, the seabed level is expected to vary naturally on hourly or longer timescales to the order of centimetres to decimetres. This is primarily due to the movement of small scale bedforms due to the action of tidal currents and occasionally wave induced orbital currents. It will also vary over longer-timescales (order of decimetres to metres) in response to the migration of larger sand wave features, where present and mobile.

6.3 Evidence base

The adjacent operational Greater Gabbard and Galloper offshore wind farms were constructed using 140 monopile foundations of 6.3 m, and 56 monopile foundations of 7.5 m diameter, respectively. The seabed is typically a relatively thin veneer of sandy sediment overlying London Clay, which is similar to some parts of the VE array areas. All of the foundations in the Galloper foundations, but relatively few of the Greater Gabbard foundations had scour protection installed. Monitoring evidence for Greater Gabbard and Galloper offshore wind farms is currently not publicly available and therefore the extent to which (i) scour has developed in locations without scour protection; and/or (ii) secondary scour has developed in locations with scour protection is not presently known in detail. It is however assumed that the limited requirement for scour protection, which would no doubt be applied if excessive scour was identified during routine monitoring, is indicative of a limited potential for scour to occur in similar parts of the adjacent VE site.

Whitehouse (1998) provides a synthesis of a range of research papers, industry reports, monitoring studies and other evidence available at that time, describing the patterns and dimensions of scour that result from a variety of obstacle shapes, sizes and environmental conditions. Building upon a theoretical understanding of the processes involved, the accepted methods for the prediction of scour mainly rely on stochastic relationships and approaches (i.e. relationships that are based on and describe the available evidence). As such, scour analysis is an evidence-based science where suitable analogues provide the most robust basis for prediction.

Since the publication of Whitehouse (1998), evidence continues to be collected and other predictive relationships have been developed and reported by the research community. In general, more recent observations have confirmed the approaches (and associated ranges of uncertainty) presented in Whitehouse (1998). As the evidence base has grown, additional approaches and relationships have been developed to better predict scour for a wider range of more specific obstacle shapes, sizes and environmental conditions.

Monitoring evidence regarding scour development around unprotected wind farm monopile installations is provided by HR Wallingford *et al.* (2007) and ABPmer *et al.*, (2010) in a series of monitoring data synthesis reports for the Department of Trade and Industry (DTI) and Collaborative Offshore Wind Research Into The Environment (COWRIE). HR Wallingford *et al.*, (2007) note that the available data support the view that scour is a progressive process that can occur where the seabed sediment is potentially erodible and there is an adequate thickness of that sediment for scouring to occur. Where the seabed comprises consolidated pre-Holocene sedimentary units, the scour will be slower to develop and limited in depth. For instance, geotechnical surveys at Kentish Flats offshore wind farm (Outer Thames) show that the seabed consists of non-cohesive sands over more resistant London Clay. The post construction monitoring evidence generally indicates that maximum scour rates around the monopiles (of diameter 4.3 m) occurred during the first year from installation and then rapidly slowed with near stability occurring by the third anniversary of the works. Scour depths ranged from 1.5 to 1.9 m at the monitoring locations and the results indicate that the scour depth is restricted by the cohesive underlying London Clay formation. In many ways, this is a close analogue for the VE environmental setting.

A research paper by Whitehouse *et al.*, (2011) provides a summary of the field evidence for scour around gravity base foundations in the North Sea used in oil and gas projects. This review emphasized the sensitivity of scour to foundation shape, with foundations in very close proximity sharing similar hydrodynamic/ sedimentary environments displaying markedly different scour characteristics. This review also described field evidence for scour around a rectangular gravity base foundation (75 m by 80 m by 16 m high) located within the North Sea in 42 m water depth. Scour was measured as 2.5 to 3.5 m deep in 0.15 mm (i.e. fine) sand.

Scour protection is evidently a mature engineering concept and by design will both prevent primary scour and minimise secondary scour. The evidence base supporting the design of scour protection is therefore strong but is not relevant to this assessment. The evidence base concerning the environmental impacts of scour protection is more limited. Multi-layered gravel and rock scour protection is being successfully used at the Thornton Bank offshore wind farm in conjunction with six gravity base foundations in a sandy environment with water depths of approximately 28 m (ABPmer *et al.*, 2010).

6.4 Assessment

6.4.1 Outline of structures considered in assessment

The following foundation structures have been considered within the assessment presented in this section:

- Monopile foundations:
  - 15 m diameter;
- Jacket foundations:

    - 40 m x 40 m to 45 m x 45 m base dimensions with four primary 3.5 m diameter legs and various secondary bracing; and
- Gravity base foundations:
    - 55 m diameter base.

Jacket foundations can be built upon pin pile, suction bucket or gravity plinth foundations. This assessment considers the scour caused by the main jacket structure. Scour dimensions for specific foundation sub-types are expected to fall within the envelope of the assessment. In all cases, some form of bed preparation, potentially including an erosion resistant gravel bed or additional scour protection are part of the maximum design scenario; these would largely prevent the formation of scour if and where applied.

6.4.2 Factors affecting equilibrium scour depth

Engineering controls – scour protection

The greatest preventative influence on local scour depth would arise from the installation of scour protection. If correctly designed and installed, scour protection will essentially prevent the development of local primary scour as described in this section. The dimensions and nature of scour protection may vary between designs but, given its purpose, would likely cover an area of seabed approximately similar to the predicted extent of the scour.

Interaction between ambient currents and the scour protection may lead to the development of secondary scour at its edges. The local dimensions of secondary scour are highly dependent upon the specific shape, design and placement of the protection. These parameters are highly variable and so there is no clear quantitative method or evidence base for accurately predicting the dimensions of secondary scour. However, as for foundations, the approximate scale of the scour depth and extent is likely to be proportional to the much smaller size of the individual elements comprising the protection.

Natural controls

As summarised in Whitehouse (1998), a number of natural factors are known to influence equilibrium scour depth for monopiles, contributing to the range of observed equilibrium scour depths. These factors include the:

- Frequency and magnitude of ambient sediment transport;
- Ratio of monopile diameter to water depth;
- Ratio of monopile diameter to peak flow speed;
- Ratio of monopile diameter to sediment grain size;
- Sediment grain size, gradation and the geotechnical properties of sedimentary units; and
- The thickness of erodible sediment overlying more erosion resistant sublayers.

The influence of these factors where they do apply is to generally reduce the depth, extent and volume of the predicted scour, hence providing a less conservative estimate. For example, a greater frequency and magnitude of sediment transport can actually reduce the equilibrium scour depth, as the scour hole is also simultaneously being (partially) in-filled by ambient sediment transport.

The above factors have been considered in the context of the VE array area and most (except the thickness of erodible sediment) were not found to significantly or consistently reduce the predicted values for the purposes of EIA. The thickness of erodible sediment in large parts of the VE array area is limited to a thin veneer (<2 m thick). The underlying London Clay material is more erosion resistant and would not be expected to continue scouring after the overlying sediment veneer has been locally eroded. In practice, this will fundamentally limit maximum potential scour depth in most of the array area. The following assessment conservatively assumes that foundations will be located in areas of deeper erodible sediment where the full equilibrium scour depth might eventually occur.

6.4.3 Time for scour to develop around the foundation options

Scour depth can vary significantly under combined current and wave conditions through time (Harris *et al*. 2010). Monitoring of scour development around monopile foundations in UK offshore wind sites suggest that the time-scale to achieve equilibrium conditions can be of the order of 60 days in environments with a potentially mobile seabed (Harris *et al*., 2011). However, as previously stated, equilibrium scour depths may not be reached for a period of several months or even a few years where erosion resistant sediments/ geology are present. These values account for tidal variations as well as the influence of waves. (Near) symmetrical scour will only develop following exposure to both flood and ebb tidal directions.

Under waves or combined waves and currents an equilibrium scour depth for the conditions existing at that time may be achieved over a period of minutes, whilst typically under tidal flows alone equilibrium scour conditions may take several months to develop.

6.4.4 Spatial extent of scour

At the Scroby Sands offshore wind farm, narrow, elongated scour features have been observed to extend over tens or hundreds of metres from individual foundations, leading to a more extensive impact than would normally be predicted. The development of elongate scour features at Scroby Sands is considered to have occurred due to the strongly rectilinear nature of the tidal currents (a very well defined tidal current axis with minimal deviation during each half tidal cycle) which allows the narrow turbulent wake behind each foundation to persist over the same areas of seabed for a greater proportion of the time, leading to net erosion in these areas. Due to a relatively higher rate of tidal rotation, the development of such elongate scour features is less likely to occur within the VE array area.

6.4.5 Results

Table 2 and Table 3 summarize the key results of the first-order scour assessment undertaken using the methodological approach set out in Appendix A. Results conservatively assume maximum equilibrium scour depths are symmetrically present around the perimeter of the structure in a uniform and frequently mobile sedimentary environment. Derivative calculations of scour extent, footprint and volume assume an angle of internal friction = 32°. Scour extent is measured from the structure's edge. Scour footprint excludes the footprint of the structure. Scour pit volumes for gravity base foundation structures are calculated as the volume of an inverted truncated cone, minus the structure volume; scour pit volume for the jacket foundations are similarly calculated but as the sum of that predicted for each the corner piles.

Table 2. Summary of predicted maximum scour dimensions for largest individual wind turbine foundation structures

Parameter		Foundation type		
		Monopile (15 m diameter)	Multi-leg Jacket (45 m base, 4 x 3.5 m legs)	Gravity Base (55 m diameter)
	Steady current	19.5	4.6	2.1

Parameter		Foundation type		
		Monopile (15 m diameter)	Multi-leg Jacket (45 m base, 4 x 3.5 m legs)	Gravity Base (55 m diameter)
Equilibrium Scour Depth (m)^	Waves	Insufficient for scour	Insufficient for scour	2.2
	Waves and current	19.5	4.6	3.5
	Global scour	N/A	1.4	N/A
Extent from foundation* (m)	Local scour	31.2	7.3	3.3
	Global scour	N/A	45.0	N/A
Footprint* (m^2)	Structure alone	177	38	2,376
	Local scour (exc. Structure)	4,530	987	606
	Global scour (exc. Structure)	N/A	6,323	N/A
Volume* (m^3)	Local scour (exc. Structure)	34,224	1,739	615
	Global scour (exc. local scour and structure)	N/A	8,853	N/A

^ Results assume erodible bed and absence of geological controls

* Based upon the scour depth for steady currents. Footprint and volume values are per foundation.

Table 3. Total seabed footprint of the different foundation types with and without scour

Parameter	Foundation type		
	Monopile (15 m diameter)	Multi-leg Jacket (WTG 45 m base, 4 x 3.5 m legs; OSP 100 x 60 m base, 6 x 3.5 m legs)	Gravity Base (55 m diameter)
Maximum number of foundations	79 WTG + 2 OSP	79 WTG + 2 OSP	79 WTG + 2 OSP
Seabed footprint of all foundations (m^2)	14,314	3,156	192,442
Proportion of array area* (%)	0.01	0.00	0.15
Seabed footprint of all local scour (m^2)	366,930	80,896	49,126
Proportion of array area* (%)	0.29	0.06	0.04
Seabed footprint of all foundations + local scour (m^2)	381,244	84,052	241,568
Proportion of array area* (%)	0.30	0.07	0.19
Seabed footprint of all global scour (m^2)	NA	515,129	NA
Proportion of array area* (%)	NA	0.40	NA
Seabed footprint of all scour protection (m^2)	423,945	121,528	1,202,826
Proportion of array area* (%)	0.33	0.09	0.94
Seabed footprint of all foundations + scour protection (m^2)	438,259	124,684	1,395,268
Proportion of array area* (%)	0.34	0.10	1.09

* Combined array area = 128.03 km^2, comprising the northern and southern array areas (66.88 km^2 and 61.15 km^2, respectively)

In the following section, the term 'local scour' refers to the local response to individual structure members. 'Global scour' refers to a region of shallower but potentially more extensive scour associated with a multi-member foundation resulting from the change in flow velocity through the gaps between members of the structure and turbulence shed by the entire structure. Global scour does not imply scour at the scale of the wind farm array.

Key findings are summarised below:

- Overall, scour development within the VE array area is expected to be dominated by the action of tidal currents;
- In practice, the thickness of unconsolidated (and more easily erodible) surficial Holocene sediment is spatially variable across the VE array, with the greatest thicknesses found in central and eastern areas of the array (Fugro, 2022a). In the west, pre-Holocene material is at or close to the surface and may limit the extent to which scour can occur. (Detailed geotechnical information is not currently available so the extent to which this is the case remains unknown at this stage);
- Of all the turbine foundation options under consideration, a 15 m diameter monopile foundation has the potential to cause the greatest equilibrium local scour depth (19.5 m), footprint (4,530 m^2) and volume (up to 34,224 m^3), but only in areas where the seabed is potentially erodible by the action of scour to that depth;
- The greatest individual turbine foundation global scour footprint is associated with the (45 m base length) jacket foundation (6,323 m^2), although with a relatively small average depth (1.4 m);
- For the VE array as a whole, the greatest total turbine foundation local scour footprint is associated with an array of 79 smaller WTG monopile foundations (15 m diameter) and two OSP monopile foundations (15 m diameter) (366,930 m^2, equivalent to only approximately 0.29% of the combined array area); and
- For the VE array as a whole, the greatest total turbine foundation global scour footprint is associated with an array of 79 smaller WTG piled jacket foundations (45 m base length) and two OSP piled jacket foundations (100 x 60 m base length) (515,129 m^2), equivalent to only approximately 0.40% of the combined array area.

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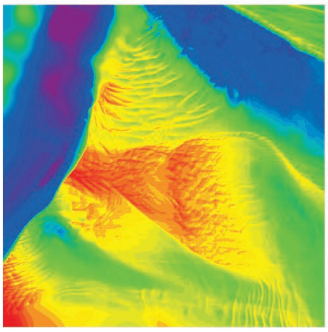
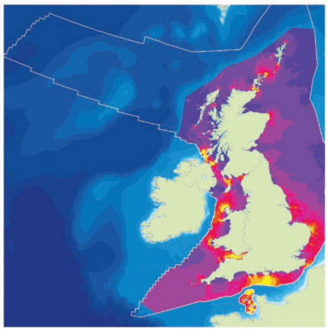
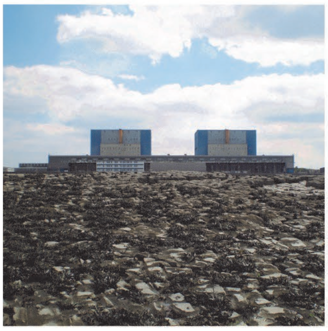
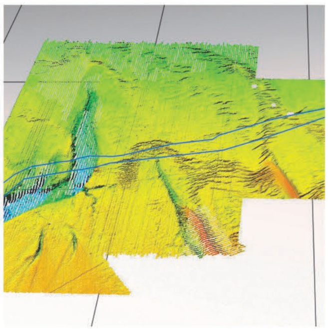
Appendices

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A Wave Model Baseline and Results Figures

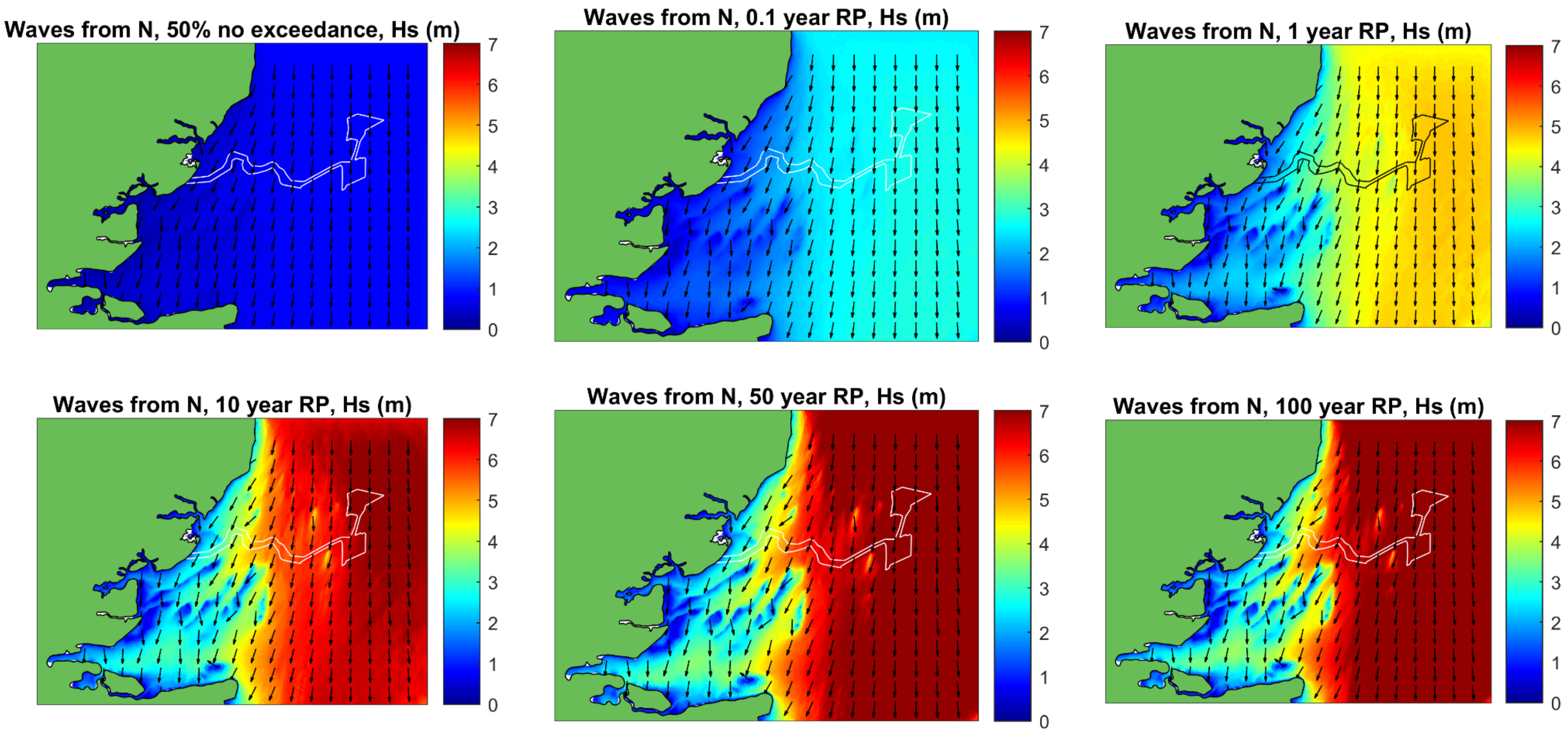


Figure A1. Baseline significant wave height, waves from the north, all return periods

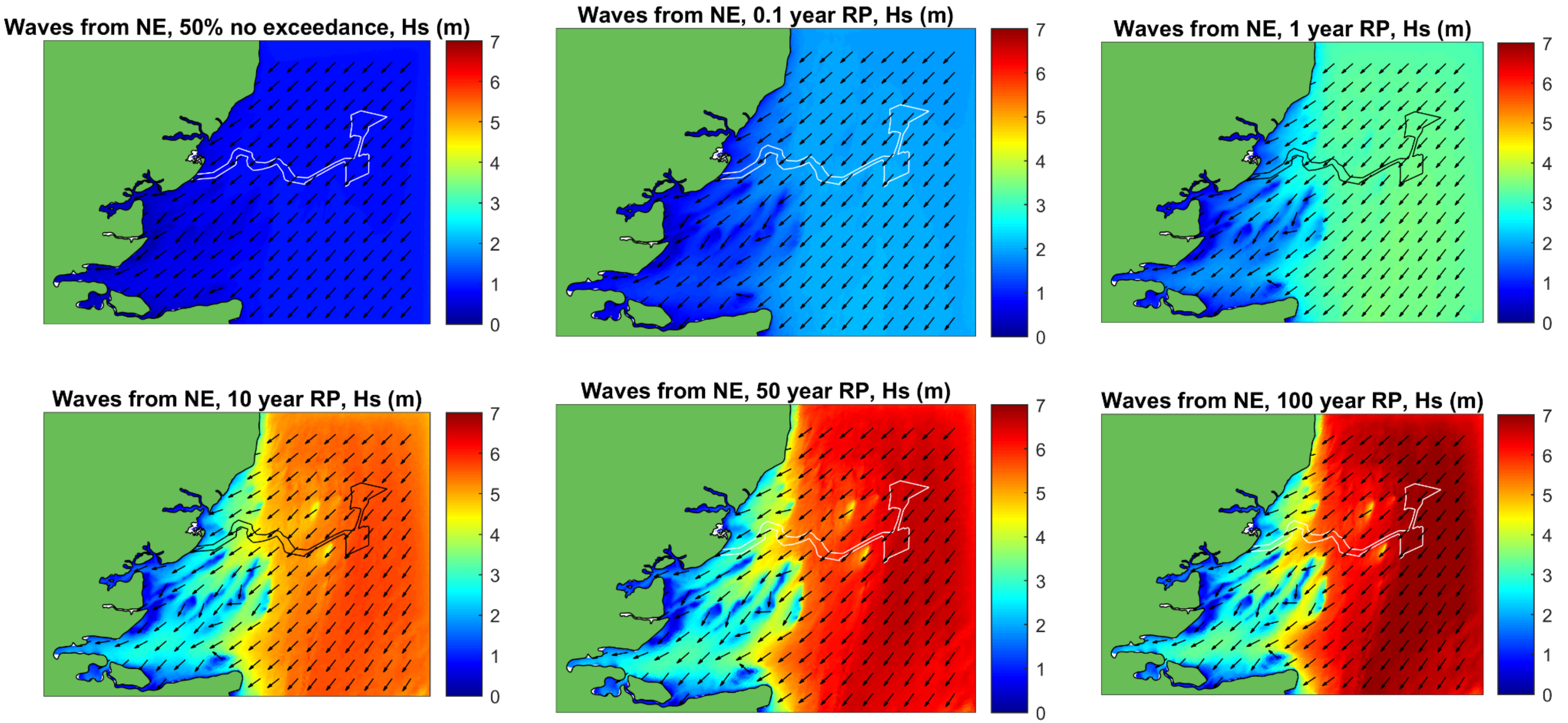


Figure A2. **Baseline significant wave height, waves from the north-east, all return periods**

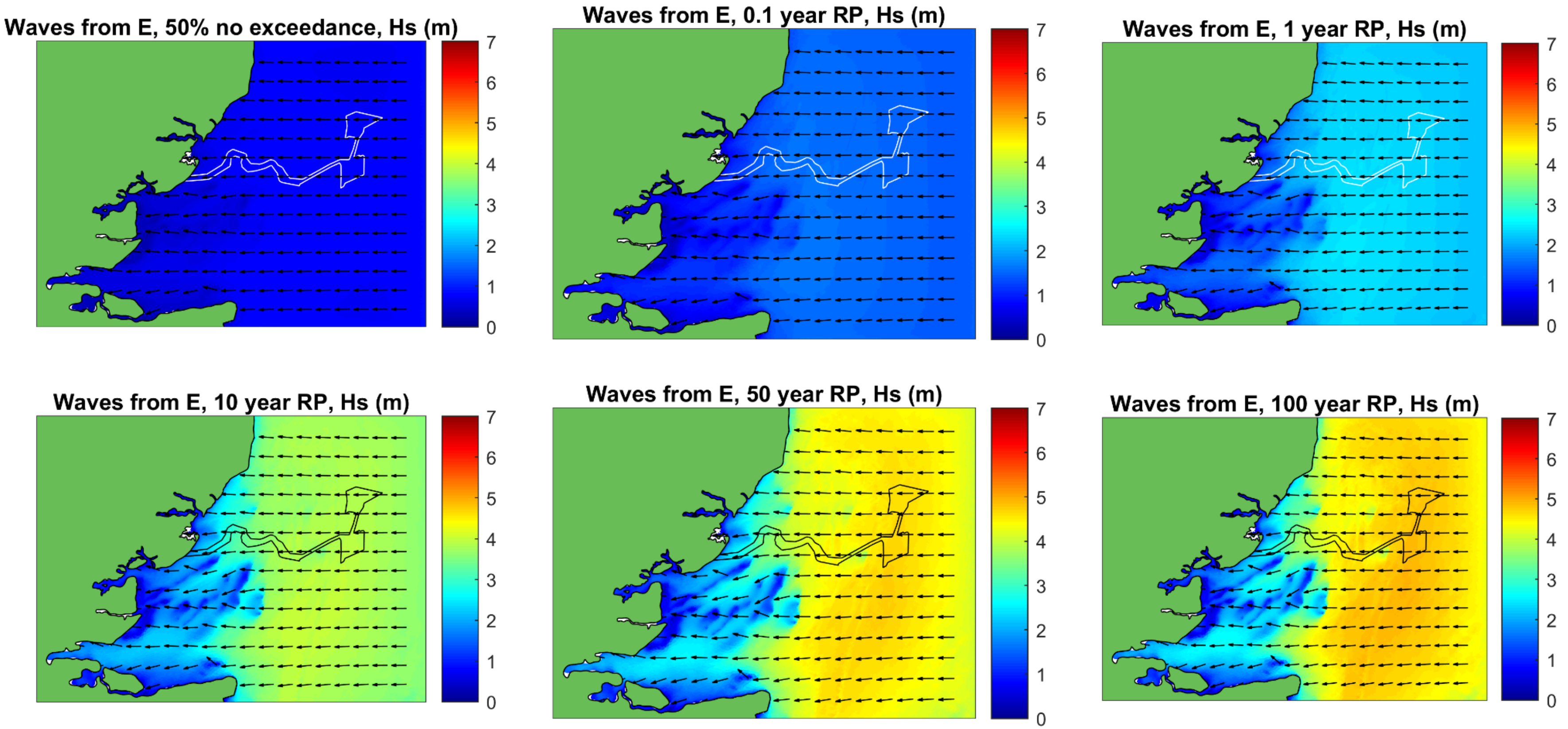


Figure A3. **Baseline significant wave height, waves from the east, all return periods**

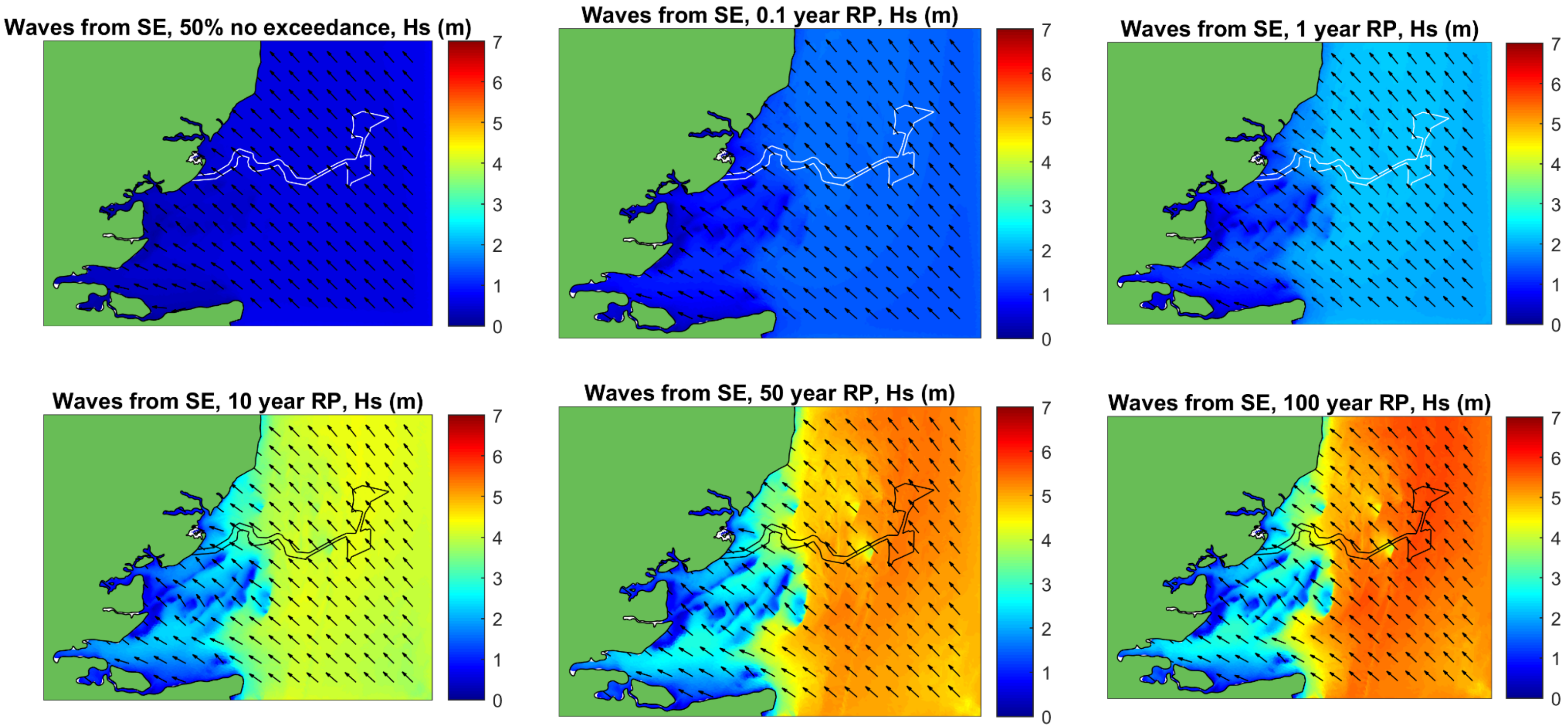


Figure A4. Baseline significant wave height, waves from the south-east, all return periods

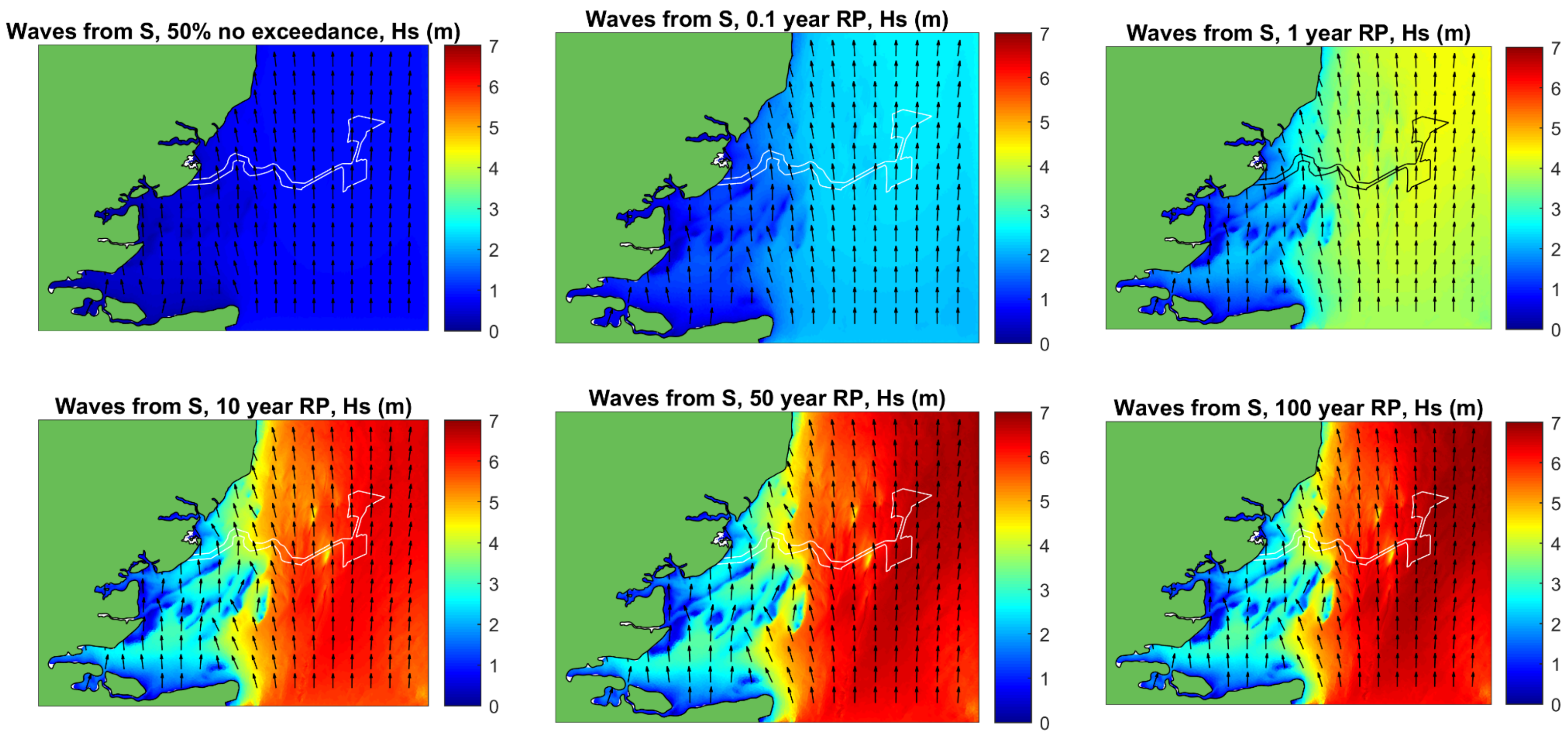


Figure A5. Baseline significant wave height, waves from the south, all return periods

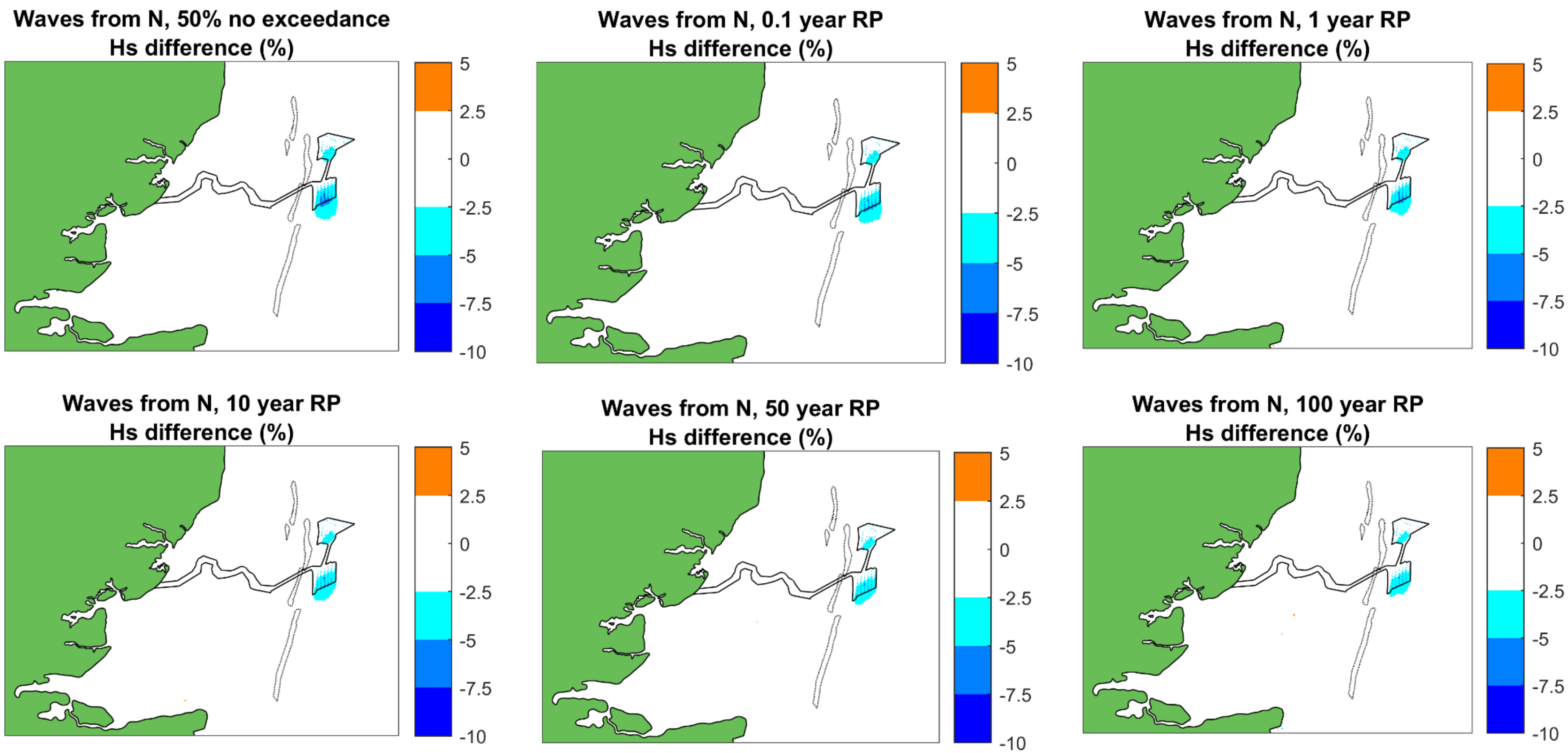


Figure A6. **Percentage difference in significant wave height (scheme minus baseline as a proportion of baseline values), operational phase, waves from the north, all return periods. Negative values are a reduction in wave height as a result of the MDS infrastructure for Five Estuaries alone. The extent of Annex I sandbank features nearby to VE are indicated as dotted lines.**

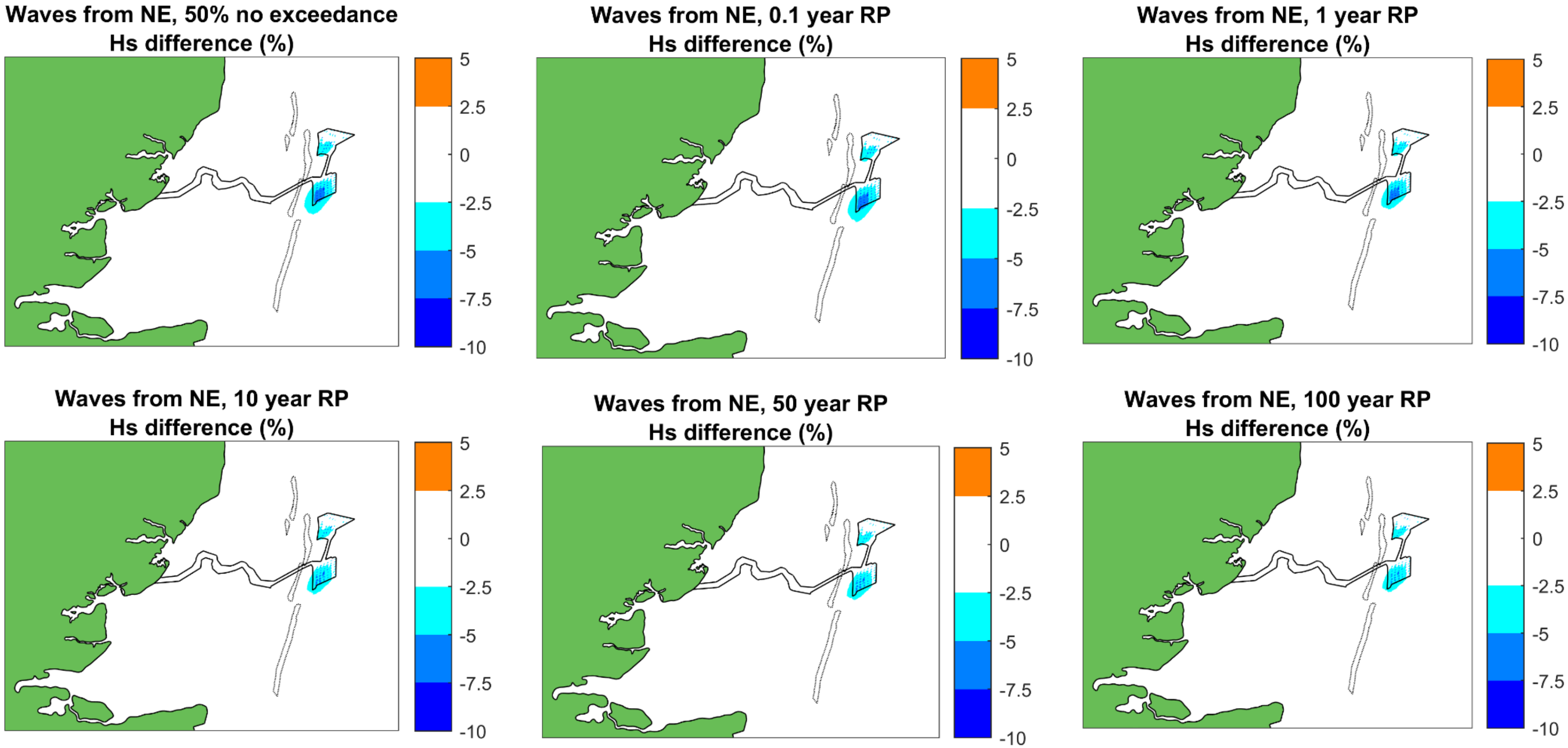


Figure A7. Percentage difference in significant wave height (scheme minus baseline as a proportion of baseline values), operational phase, waves from the north-east, all return periods. Negative values are a reduction in wave height as a result of the MDS infrastructure for Five Estuaries alone. The extent of Annex I sandbank features nearby to VE are indicated as dotted lines.

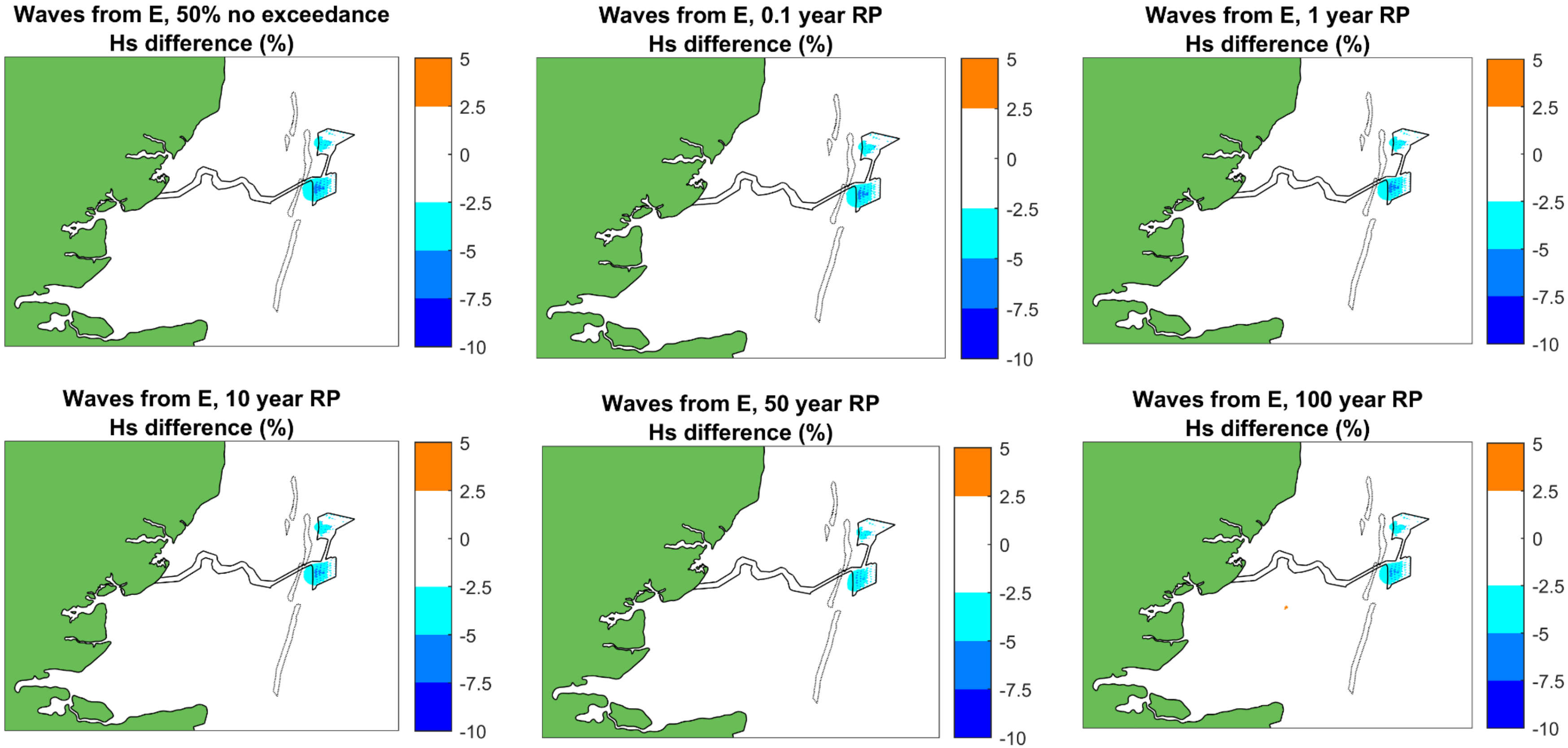


Figure A8. **Percentage difference in significant wave height (scheme minus baseline as a proportion of baseline values), operational phase, waves from the east, all return periods. Negative values are a reduction in wave height as a result of the MDS infrastructure for Five Estuaries alone. The extent of Annex I sandbank features nearby to VE are indicated as dotted lines.**

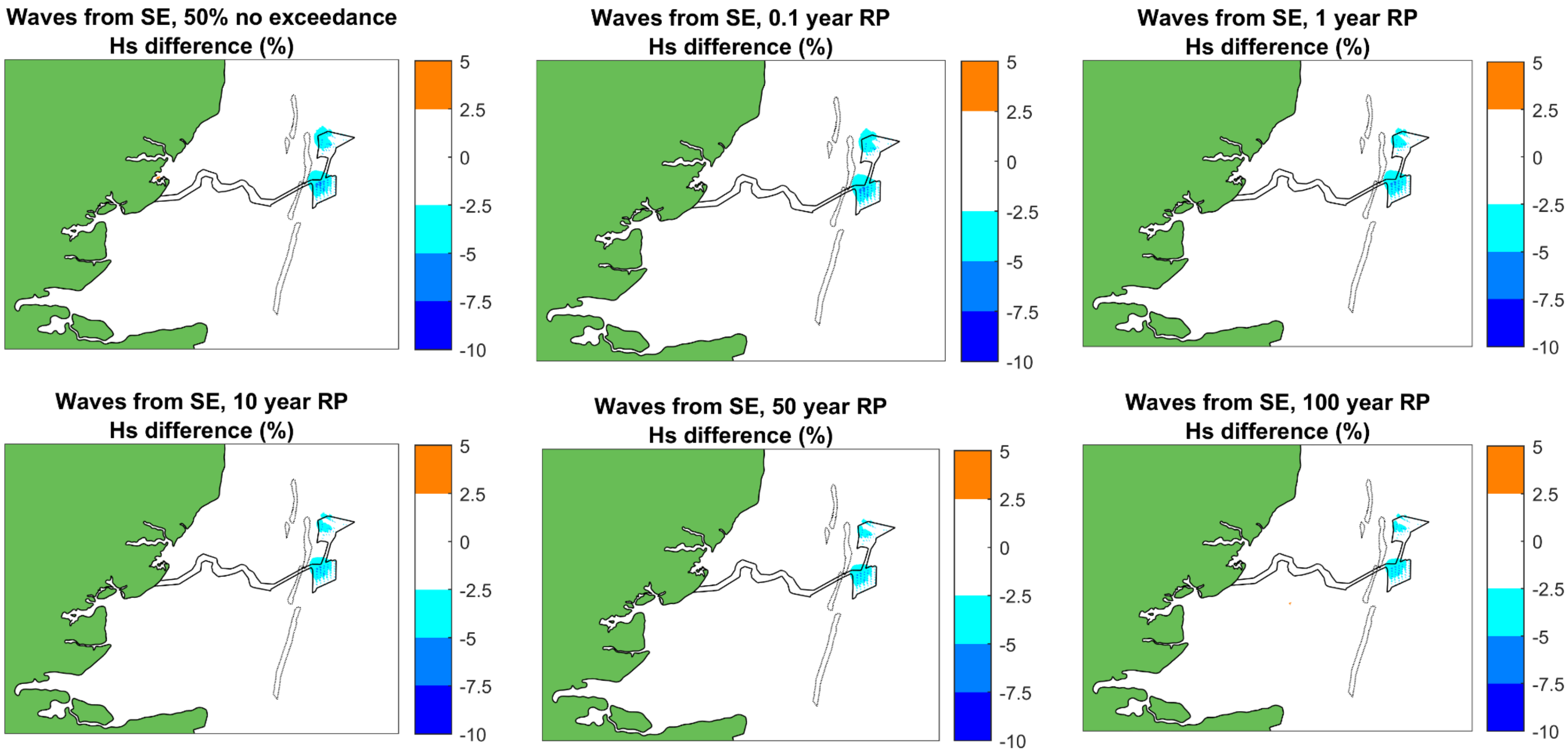


Figure A9. **Percentage difference in significant wave height (scheme minus baseline as a proportion of baseline values), operational phase, waves from the south-east, all return periods. Negative values are a reduction in wave height as a result of the MDS infrastructure for Five Estuaries alone. The extent of Annex I sandbank features nearby to VE are indicated as dotted lines.**

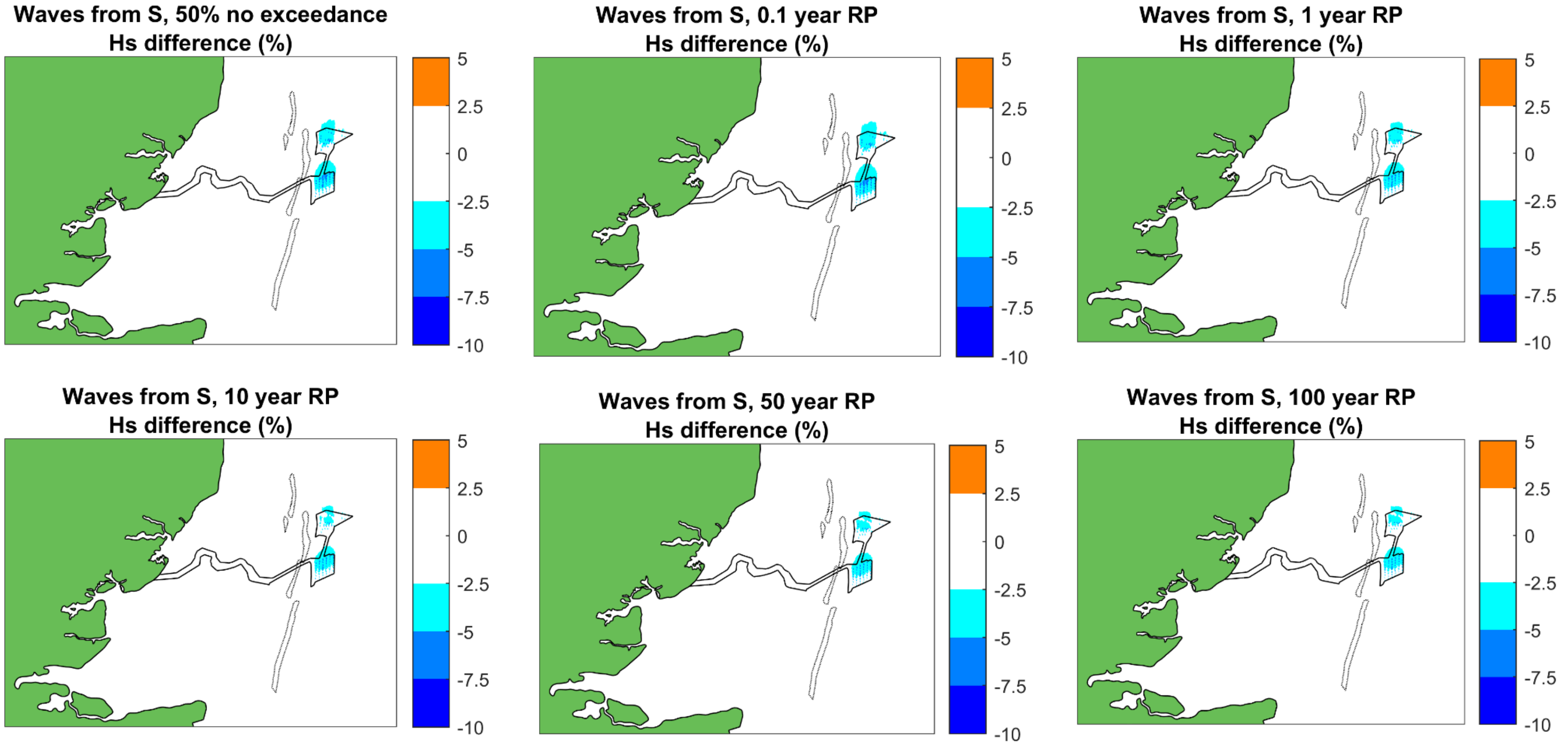


Figure A10. **Percentage difference in significant wave height (scheme minus baseline as a proportion of baseline values), operational phase, waves from the south, all return periods. Negative values are a reduction in wave height as a result of the MDS infrastructure for Five Estuaries alone. The extent of Annex I sandbank features nearby to VE are indicated as dotted lines.**

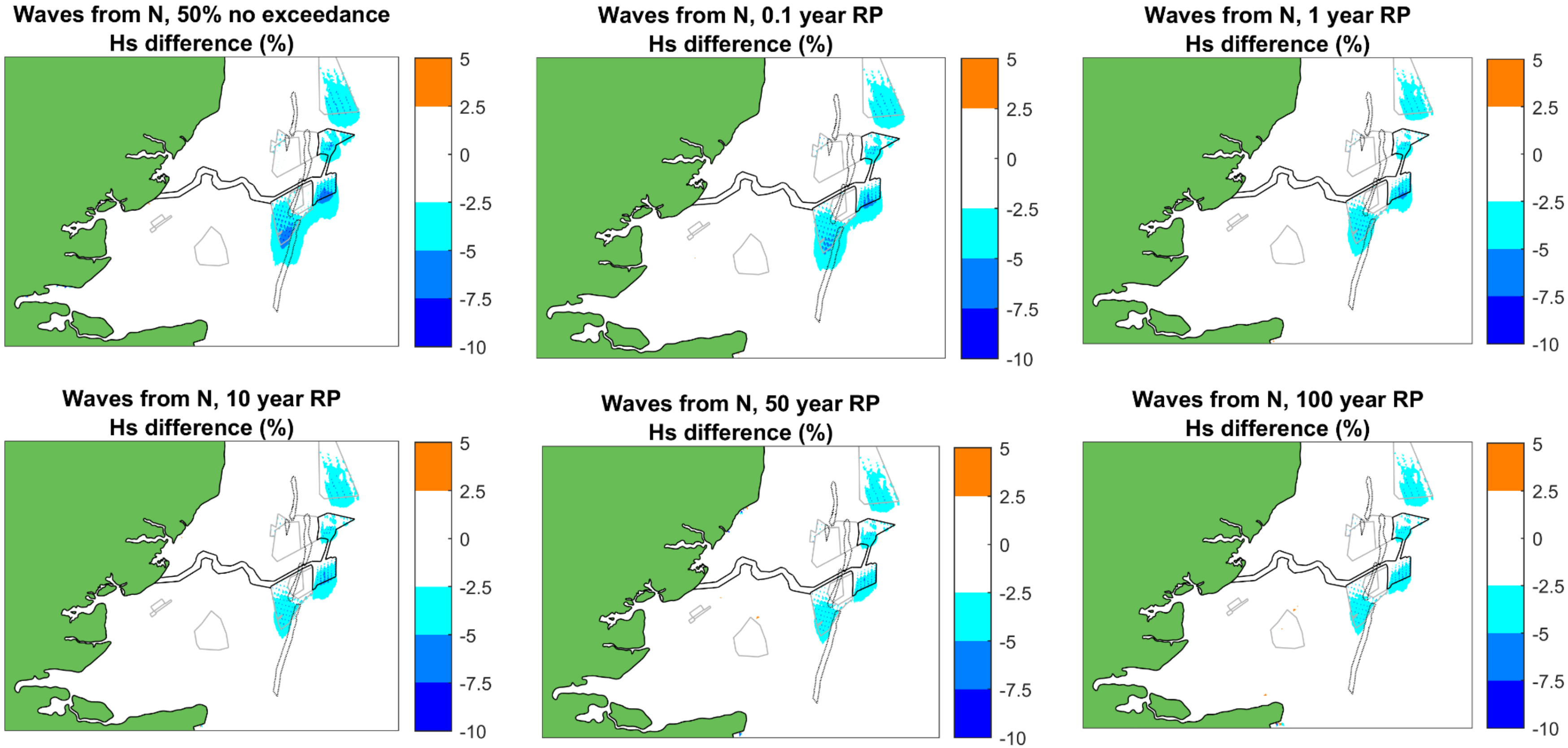


Figure A11. **Percentage difference in significant wave height (scheme minus baseline as a proportion of baseline values), operational phase, waves from the north, all return periods. Negative values are a reduction in wave height as the cumulative result of the MDS infrastructure for Five Estuaries, assumed MDS infrastructure for North Falls and East Anglia TWO, and as built infrastructure for other wind farms in the study area. The extent of Annex I sandbank features nearby to VE are indicated as dotted lines.**

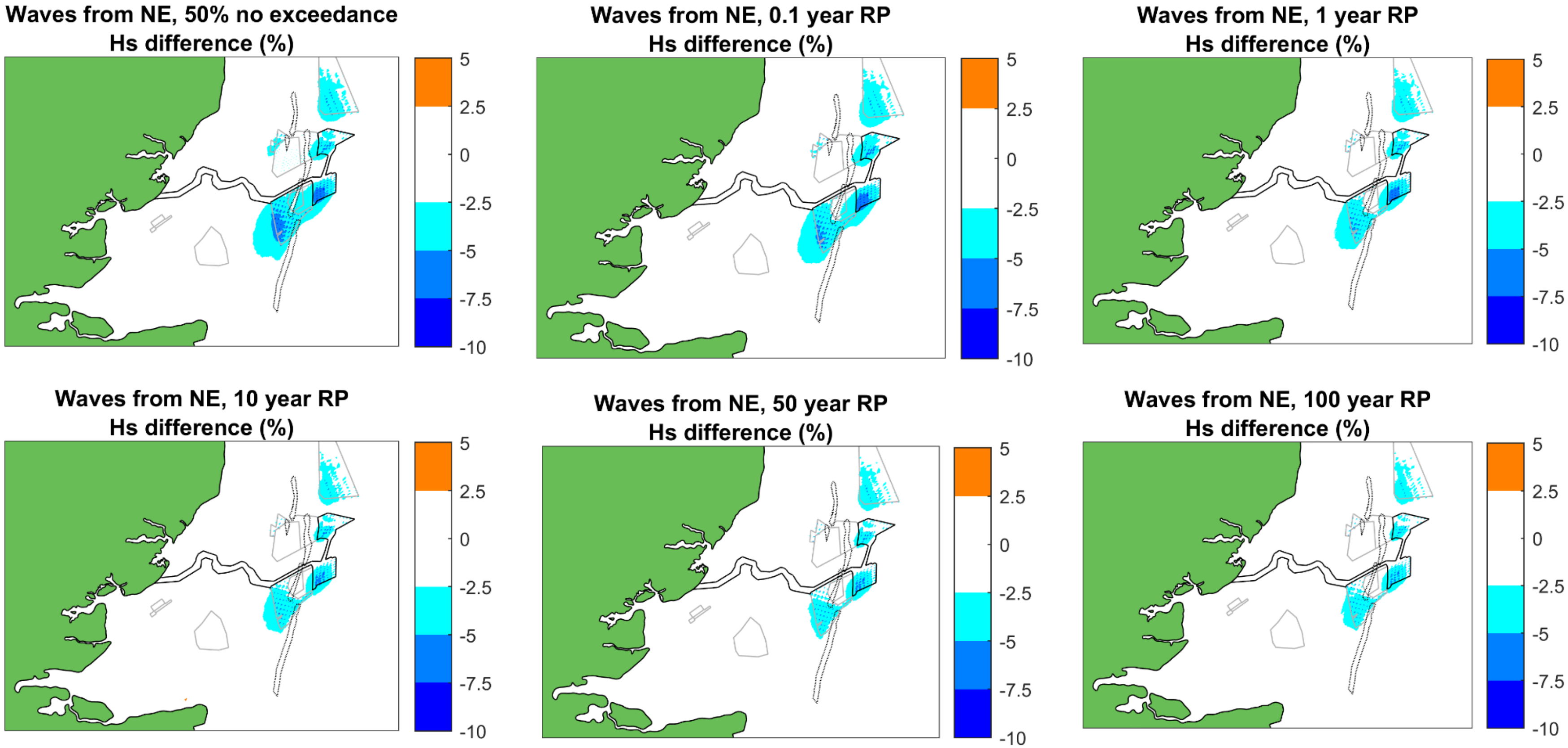


Figure A12. **Percentage difference in significant wave height (scheme minus baseline as a proportion of baseline values), operational phase, waves from the north-east, all return periods. Negative values are a reduction in wave height as a result of the MDS infrastructure for Five Estuaries, assumed MDS infrastructure for North Falls and East Anglia TWO, and as built infrastructure for other wind farms in the study area. The extent of Annex I sandbank features nearby to VE are indicated as dotted lines.**

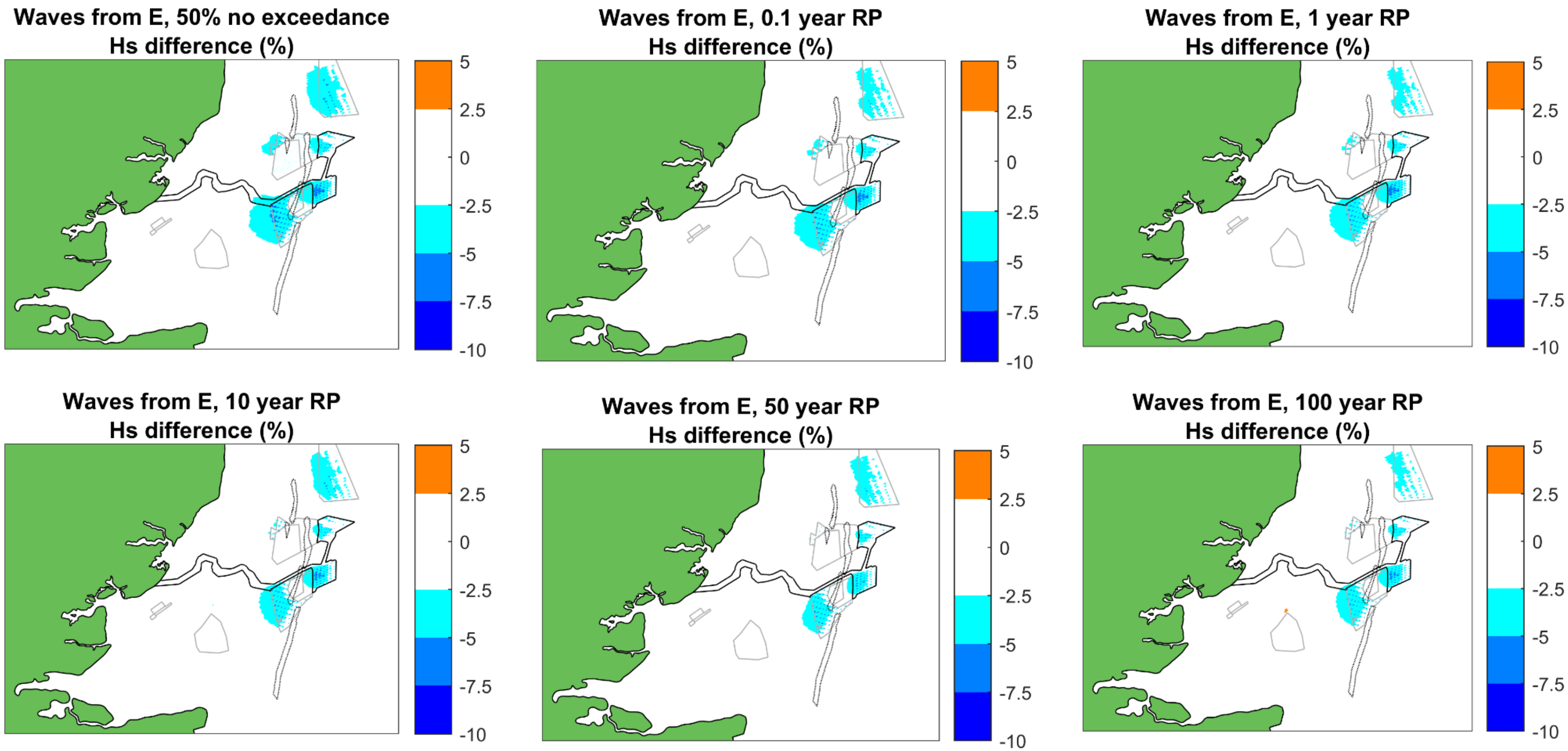


Figure A13. Percentage difference in significant wave height (scheme minus baseline as a proportion of baseline values), operational phase, waves from the east, all return periods. Negative values are a reduction in wave height as a result of the MDS infrastructure for Five Estuaries, assumed MDS infrastructure for North Falls and East Anglia TWO, and as built infrastructure for other wind farms in the study area. The extent of Annex I sandbank features nearby to VE are indicated as dotted lines.

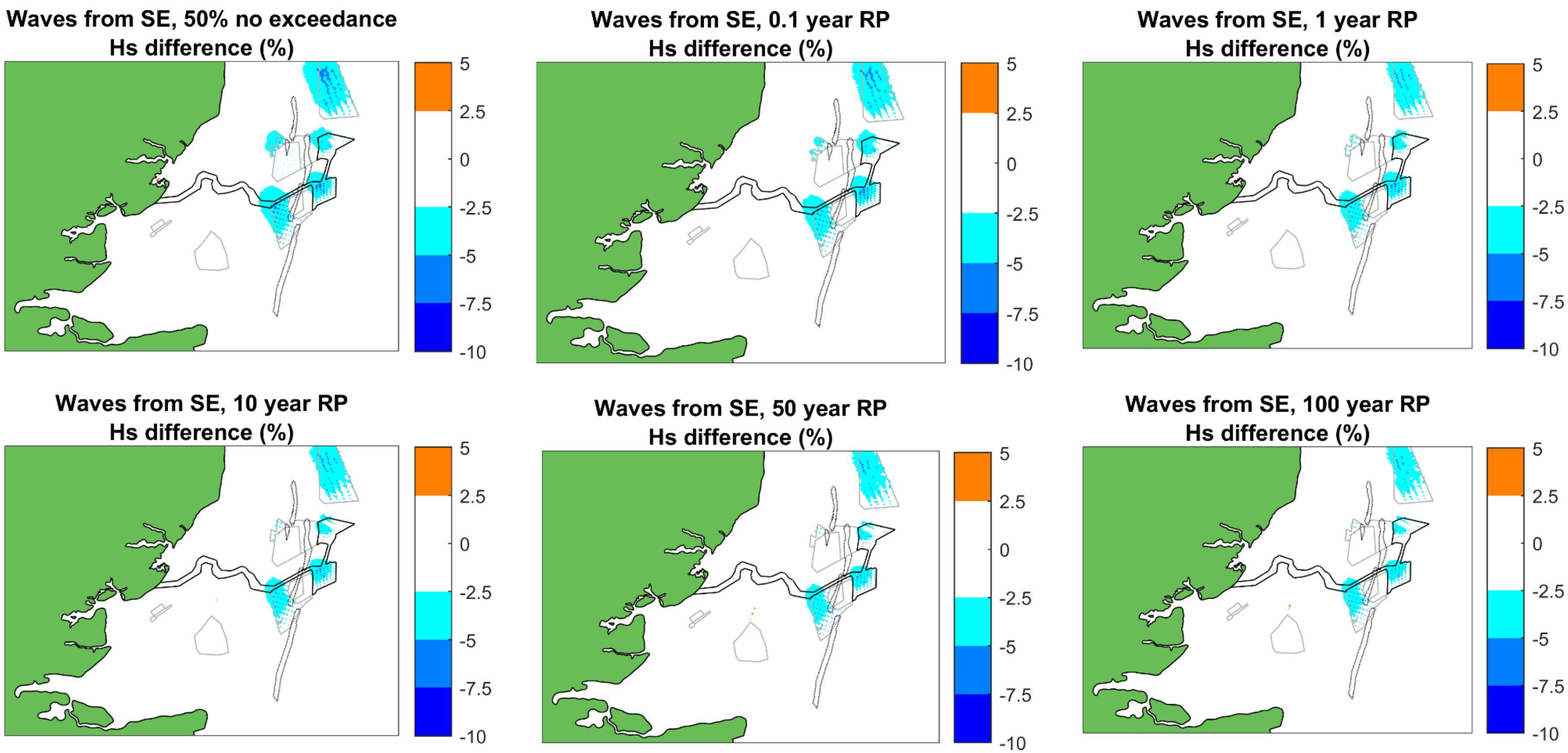


Figure A14. Percentage difference in significant wave height (scheme minus baseline as a proportion of baseline values), operational phase, waves from the south-east, all return periods. Negative values are a reduction in wave height as a result of the MDS infrastructure for Five Estuaries, assumed MDS infrastructure for North Falls and East Anglia TWO, and as built infrastructure for other wind farms in the study area. The extent of Annex I sandbank features nearby to VE are indicated as dotted lines.

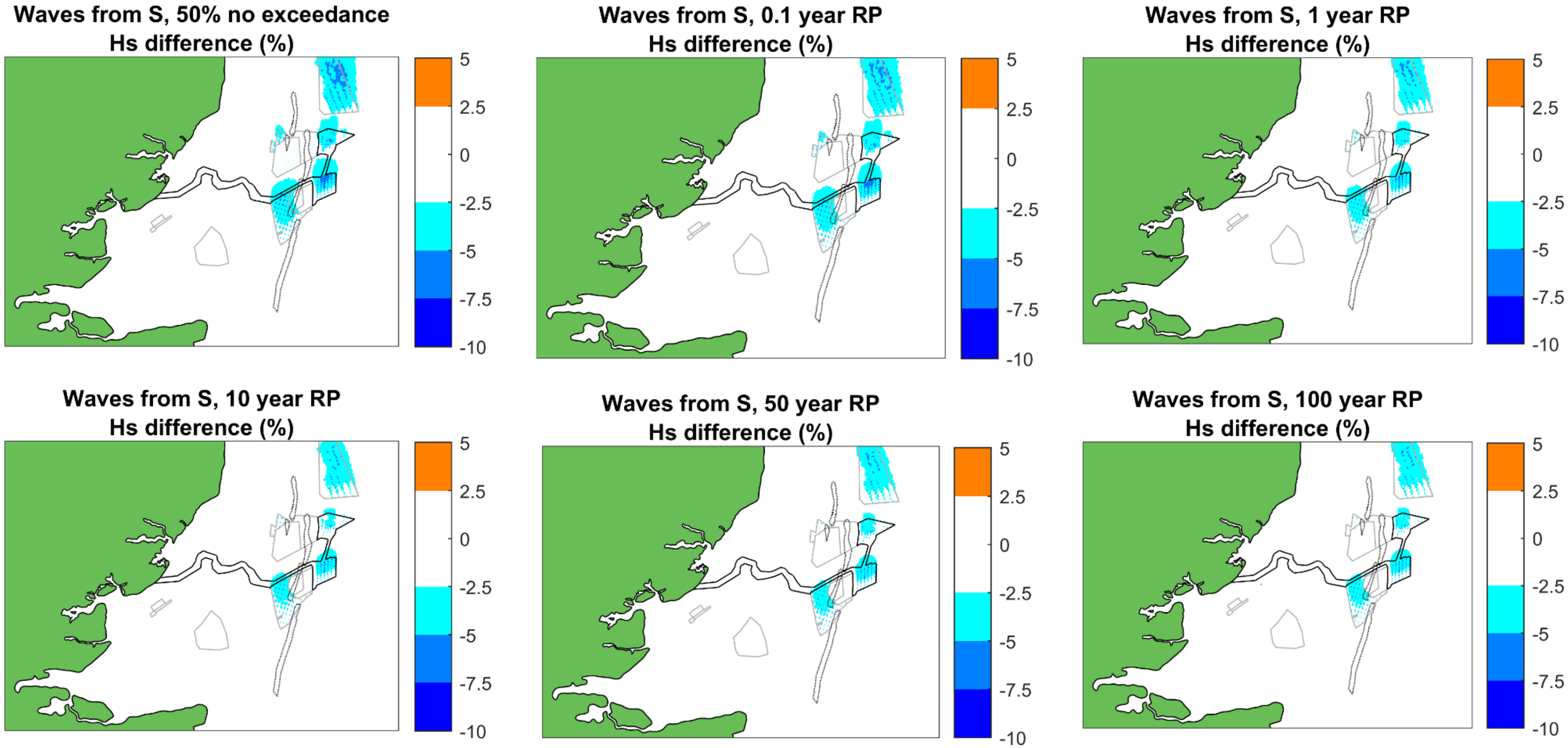


Figure A15. **Percentage difference in significant wave height (scheme minus baseline as a proportion of baseline values), operational phase, waves from the south, all return periods. Negative values are a reduction in wave height as a result of the MDS infrastructure for Five Estuaries, assumed MDS infrastructure for North Falls and East Anglia TWO, and as built infrastructure for other wind farms in the study area. The extent of Annex I sandbank features nearby to VE are indicated as dotted lines.**

B Tidal Model Baseline and Results Figures

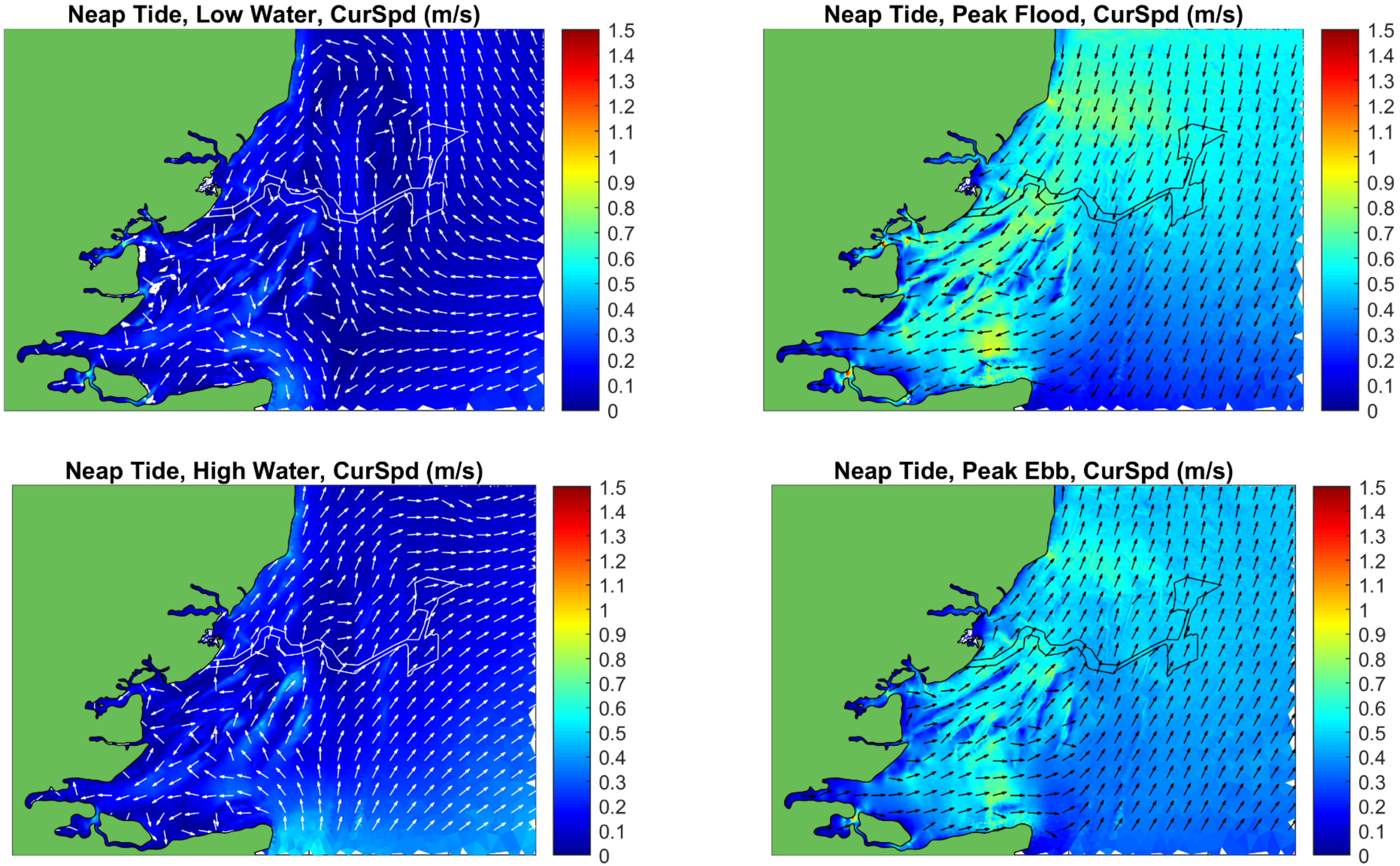


Figure B1. Baseline tidal current speed and direction during a representative neap tidal condition

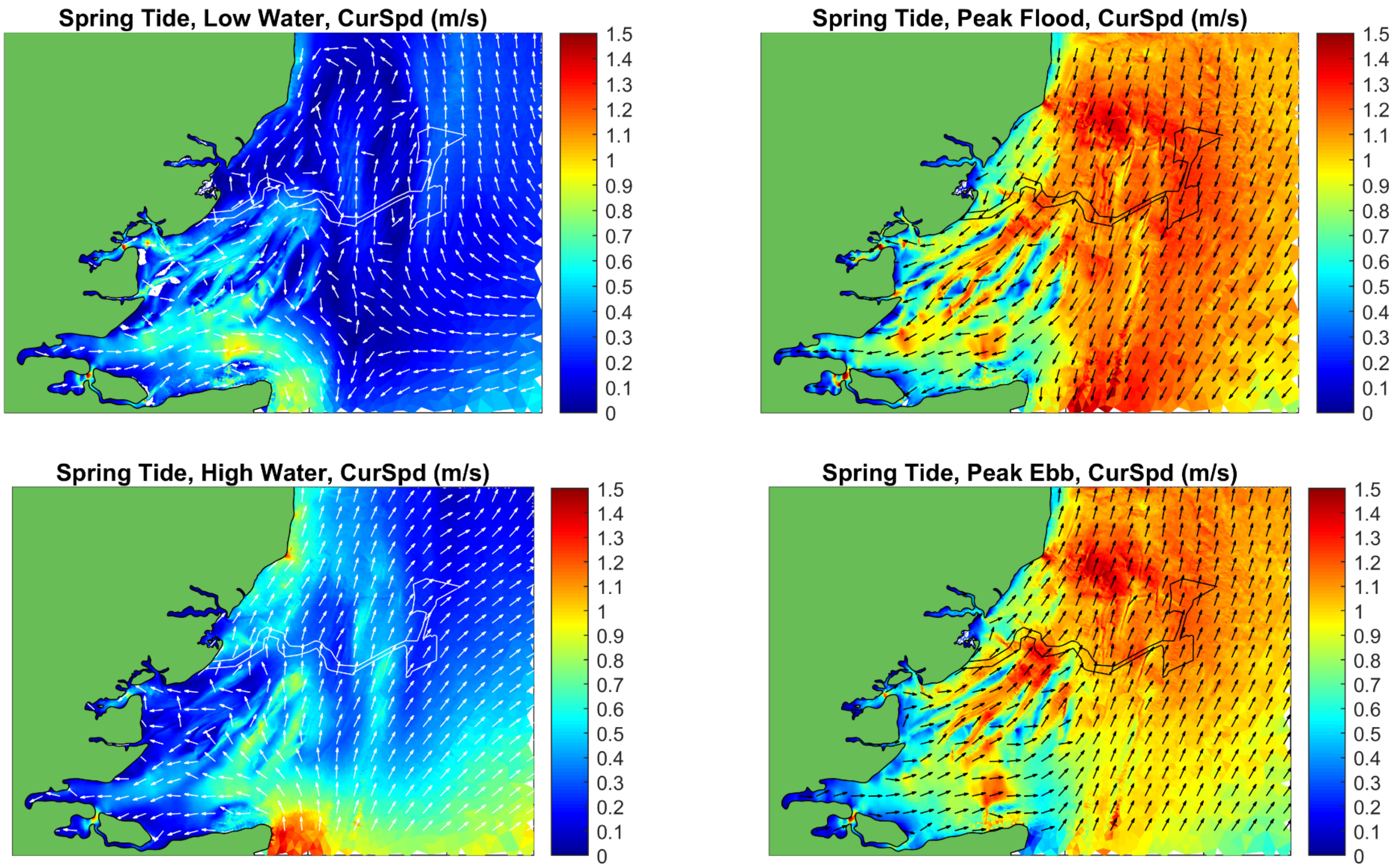


Figure B2. Baseline tidal current speed and direction during a representative spring tidal condition

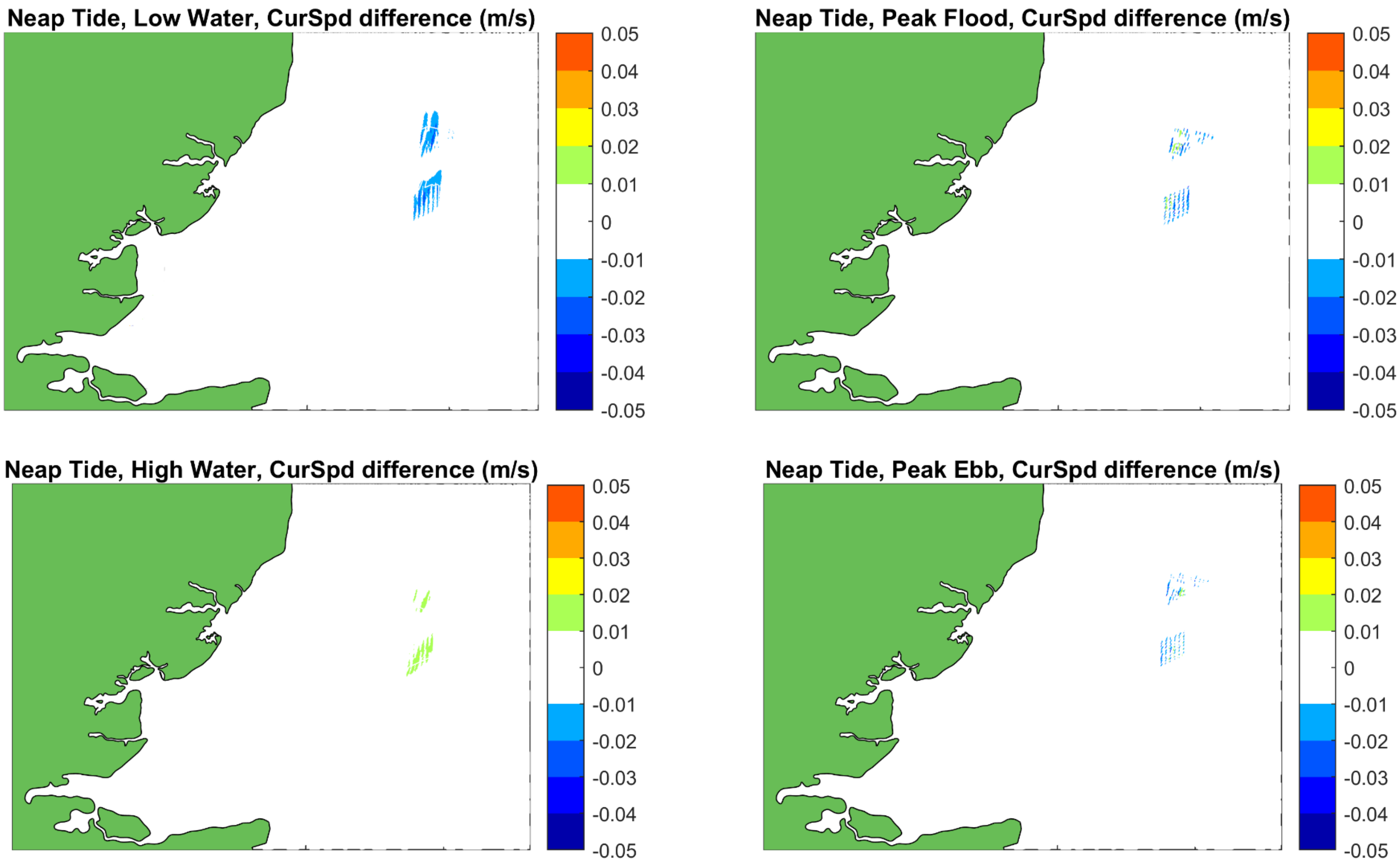


Figure B3. **Absolute difference in tidal current speed (scheme minus baseline), operational phase, during a representative neap tidal condition. Negative and positive values are a reduction or increase in time average current speed, respectively, as a result of the MDS infrastructure for Five Estuaries.**

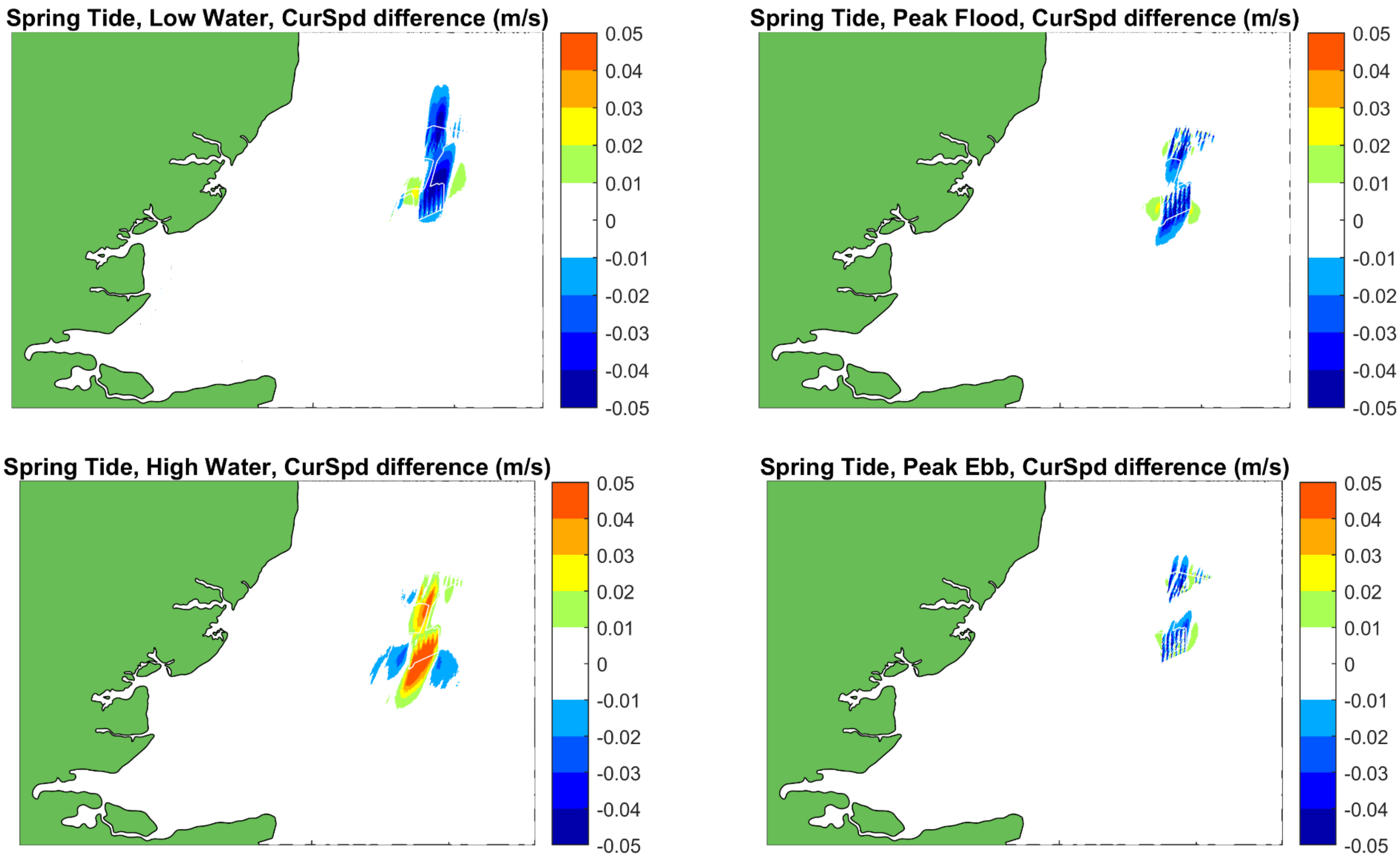
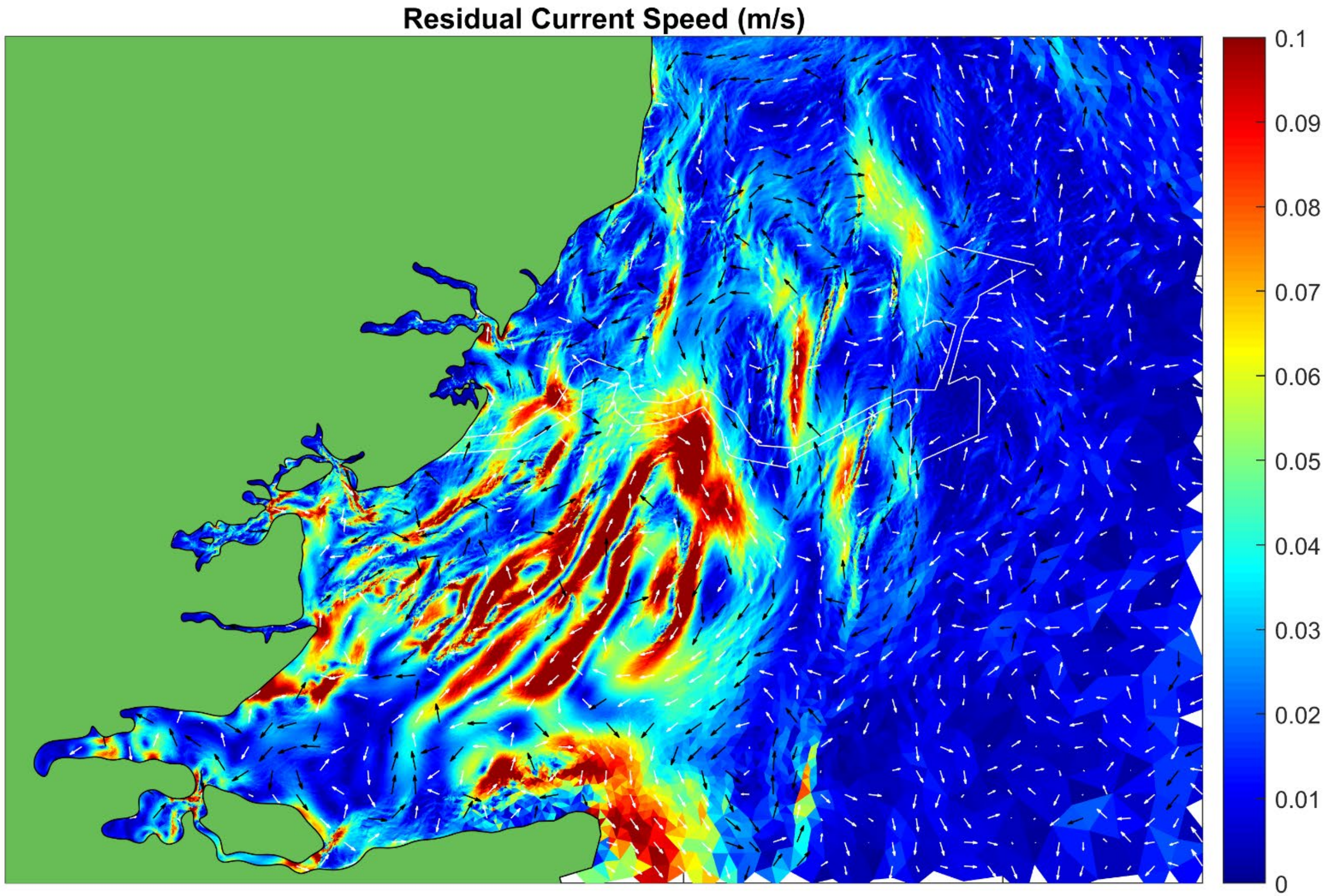


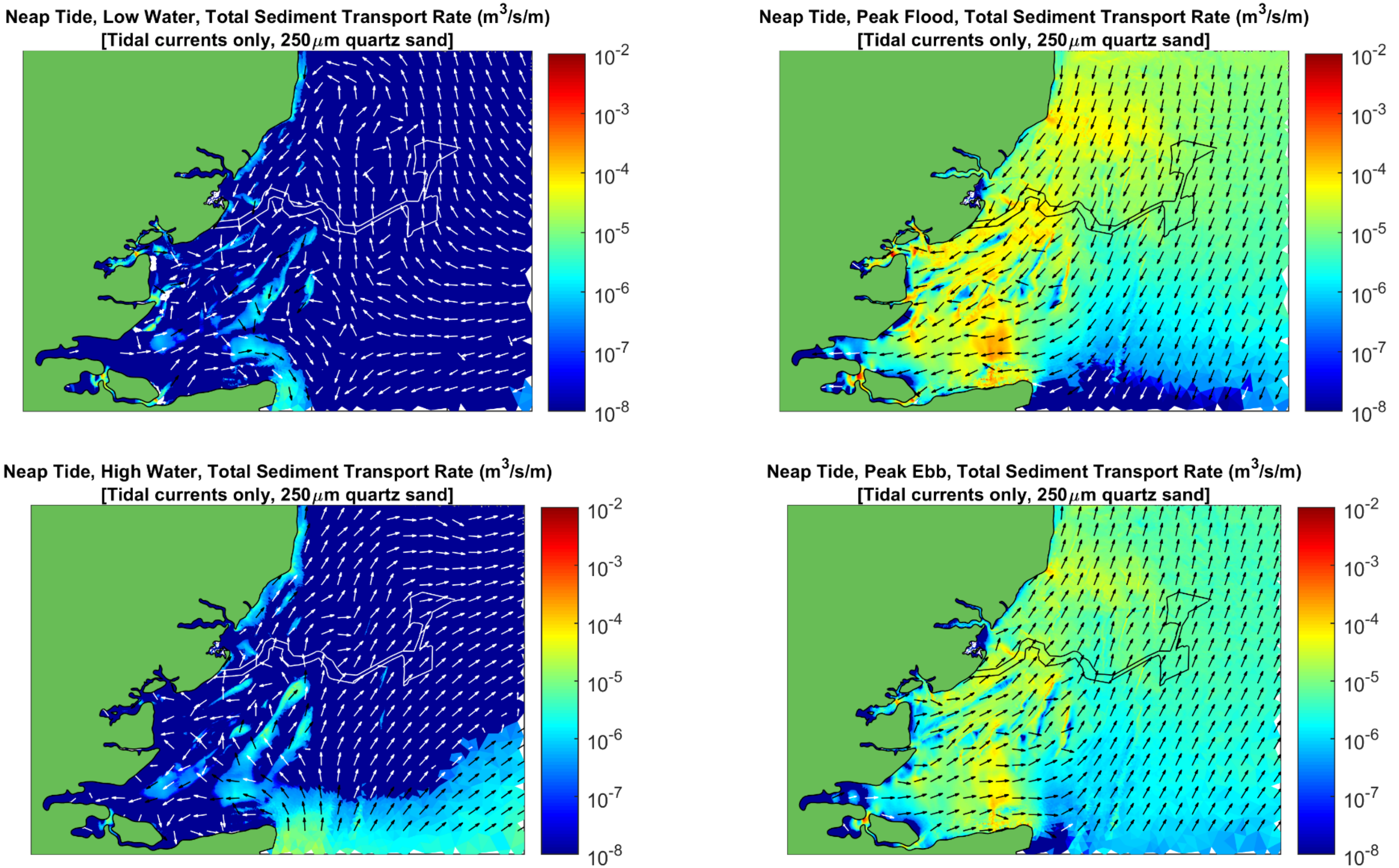
Figure B4. **Absolute difference in tidal current speed (scheme minus baseline), operational phase, during a representative spring tidal condition. Negative and positive values are a reduction or increase in time average current speed, respectively, as a result of the MDS infrastructure for Five Estuaries.**



A combination of black and white site outlines and vector arrows are used to improve visual contrast against the underlying colourmap. Direction is not indicated for residual current speeds less than 0.01 m/s.

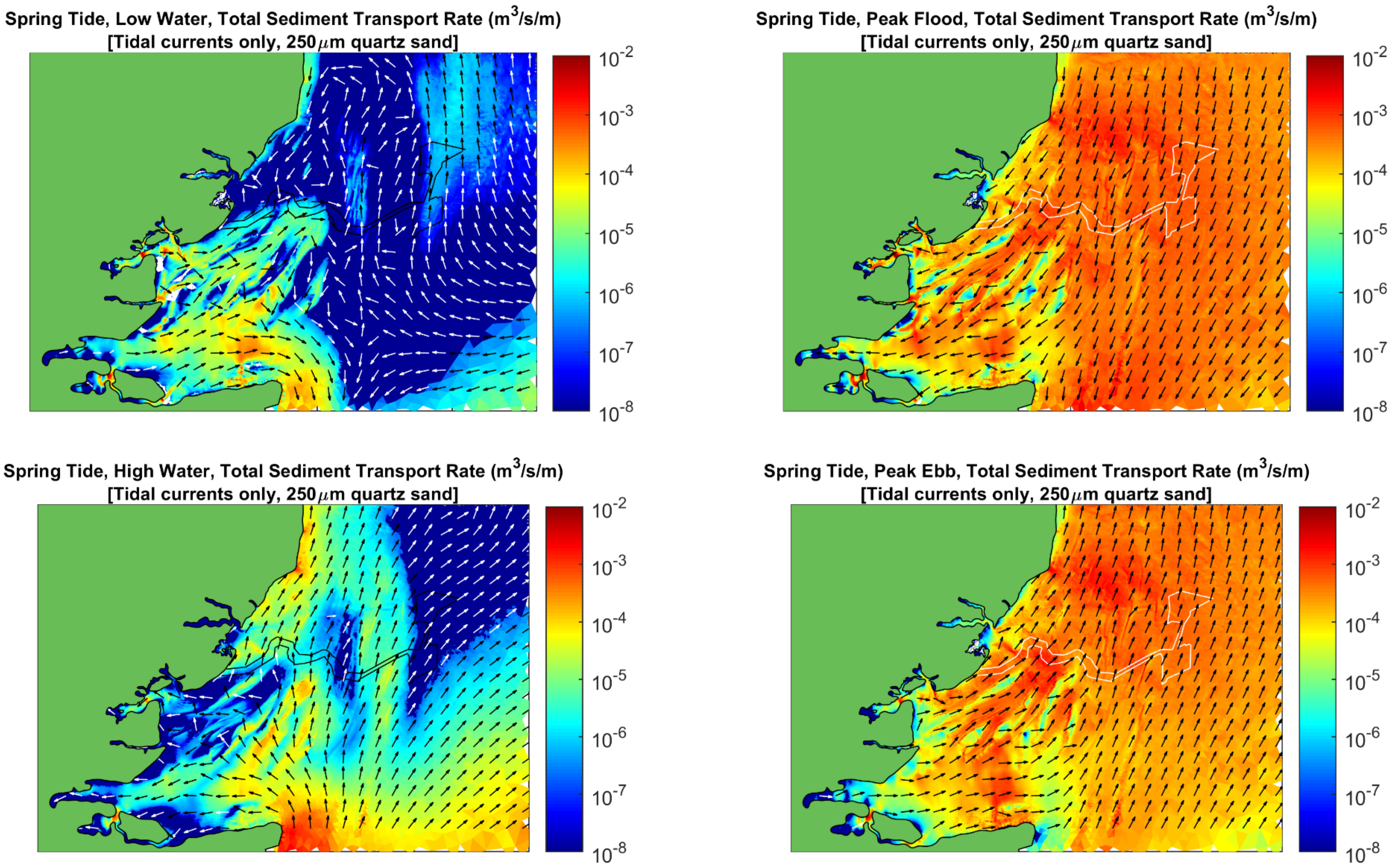
Figure B5. Baseline residual tidal current speed and direction measured over a representative spring-neap tidal period

C Sediment Transport Model Baseline and Results Figures



A combination of black and white site outlines and vector arrows are used to improve visual contrast against the underlying colourmap.

Figure C1. Baseline instantaneous sediment transport rate and direction, for 250 µm quartz sand, during a representative neap tidal condition



A combination of black and white site outlines and vector arrows are used to improve visual contrast against the underlying colourmap.

Figure C2. Baseline instantaneous sediment transport rate and direction, for 250 µm quartz sand, during a representative spring tidal condition

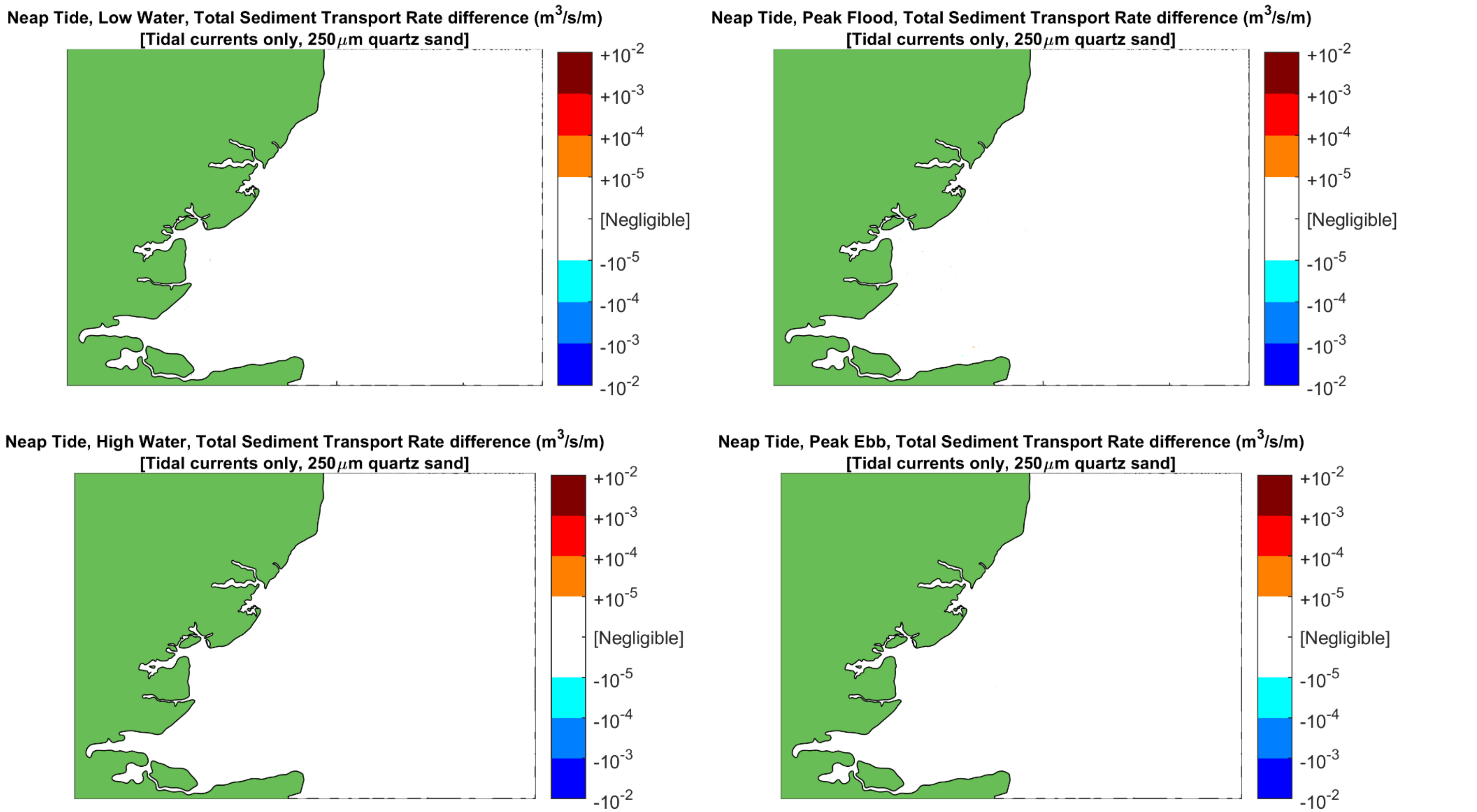


Figure C3. **Absolute difference in instantaneous sediment transport rate (scheme minus baseline), operational phase, during a representative neap tidal condition. Negative and positive values are a reduction or increase in the instantaneous sediment transport rate, respectively, as a result of the MDS infrastructure for Five Estuaries.**

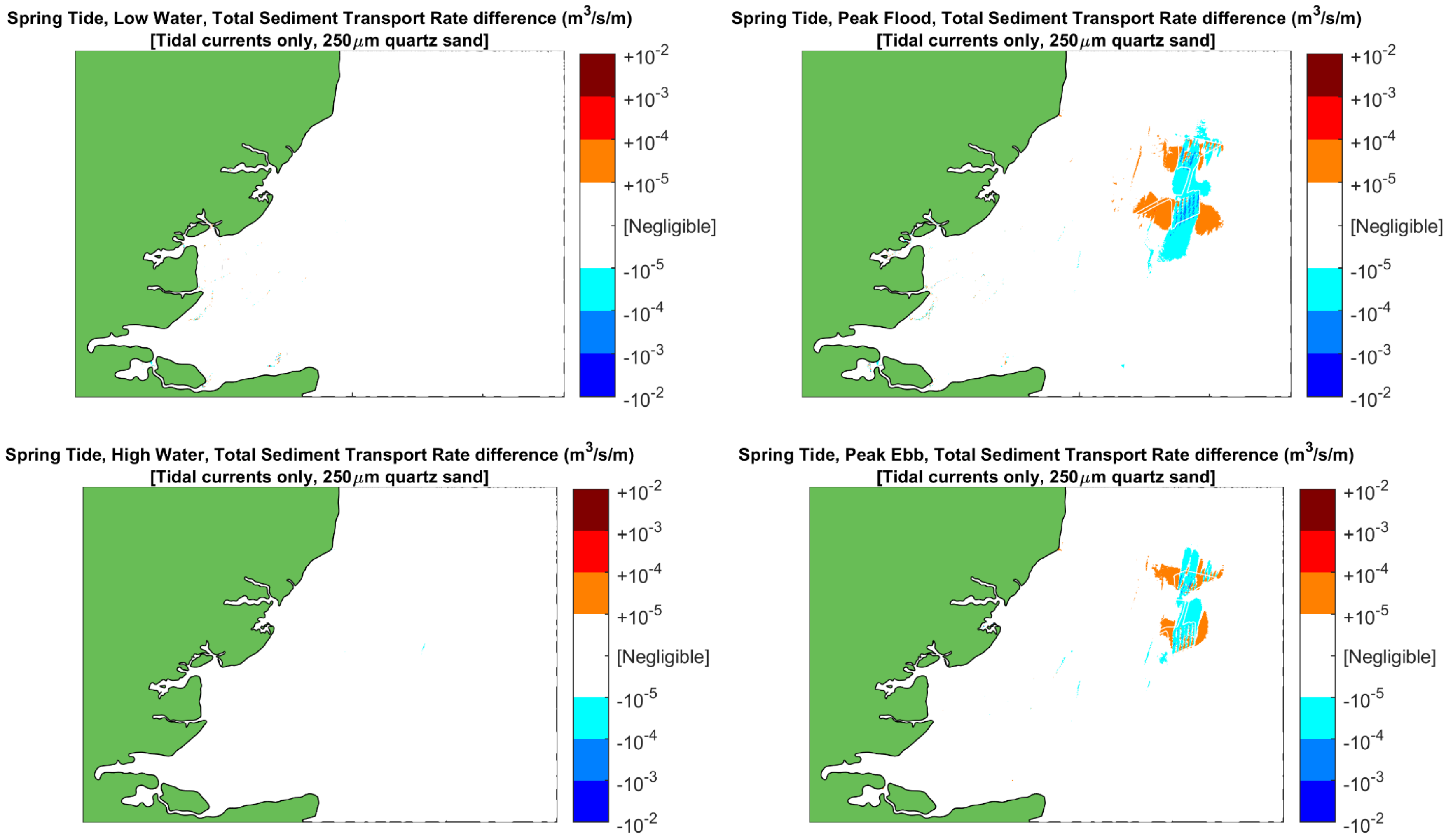
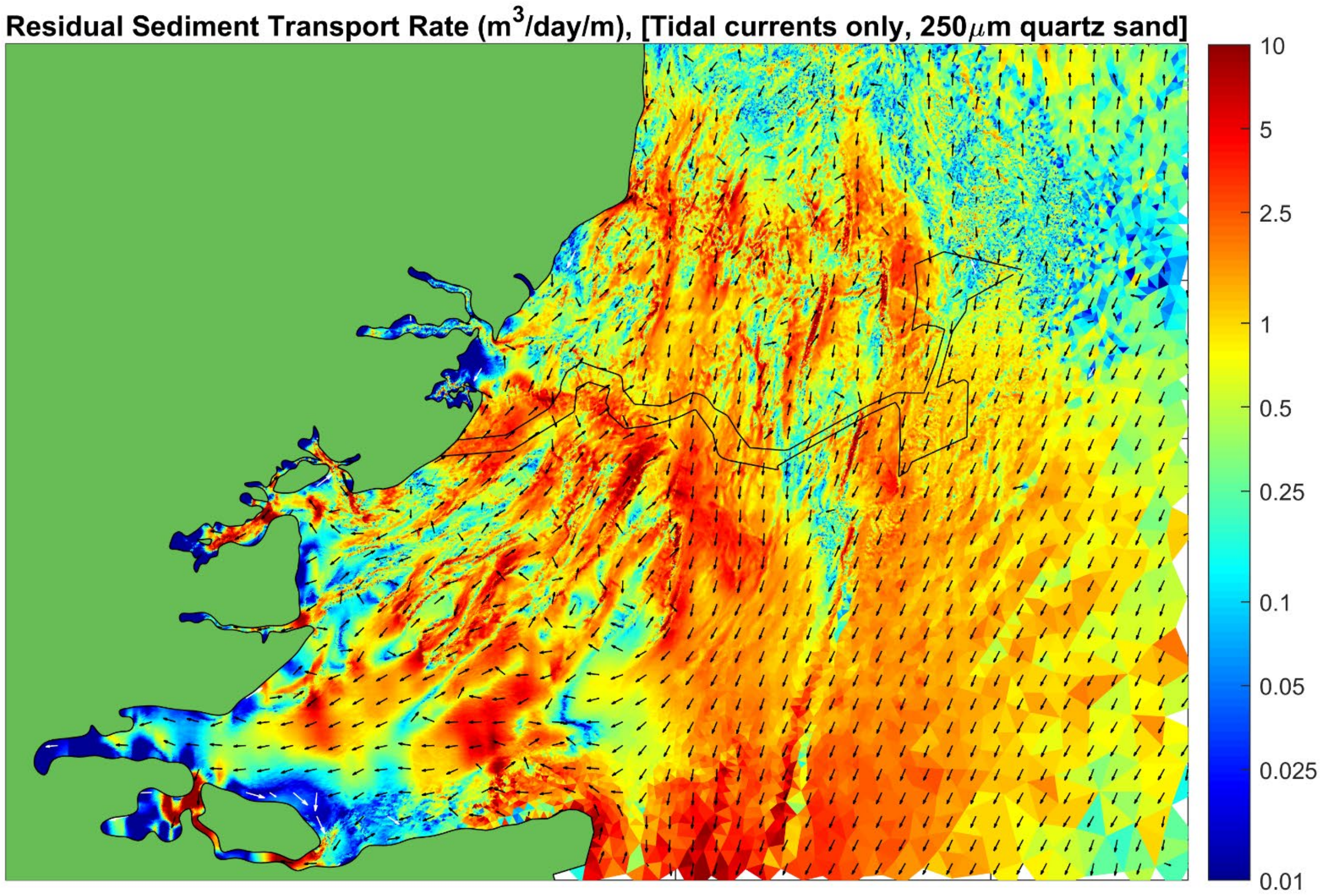
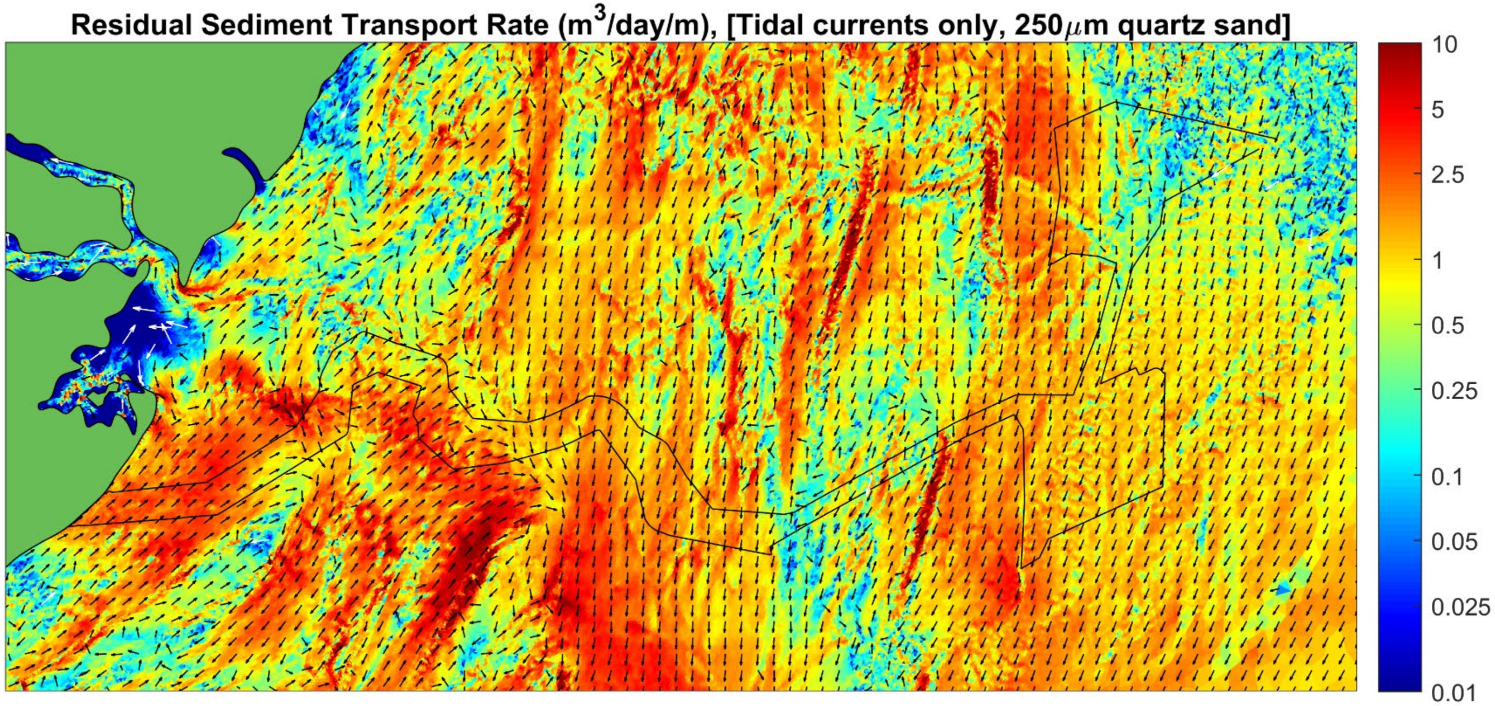


Figure C4. Absolute difference in instantaneous sediment transport rate (scheme minus baseline), operational phase, during a representative spring tidal condition. Negative and positive values are a reduction or increase in the instantaneous sediment transport rate, respectively, as a result of the MDS infrastructure for Five Estuaries.



A combination of black and white site outlines and vector arrows are used to improve visual contrast against the underlying colourmap.

Figure C5. Baseline residual sediment transport rate and direction, for 250 μm quartz sand, measured over a representative spring-neap tidal period, regional view.



A combination of black and white site outlines and vector arrows are used to improve visual contrast against the underlying colourmap.

Figure C6. **Baseline residual sediment transport rate and direction, for 250 μm quartz sand, measured over a representative spring-neap tidal period, detailed view.**

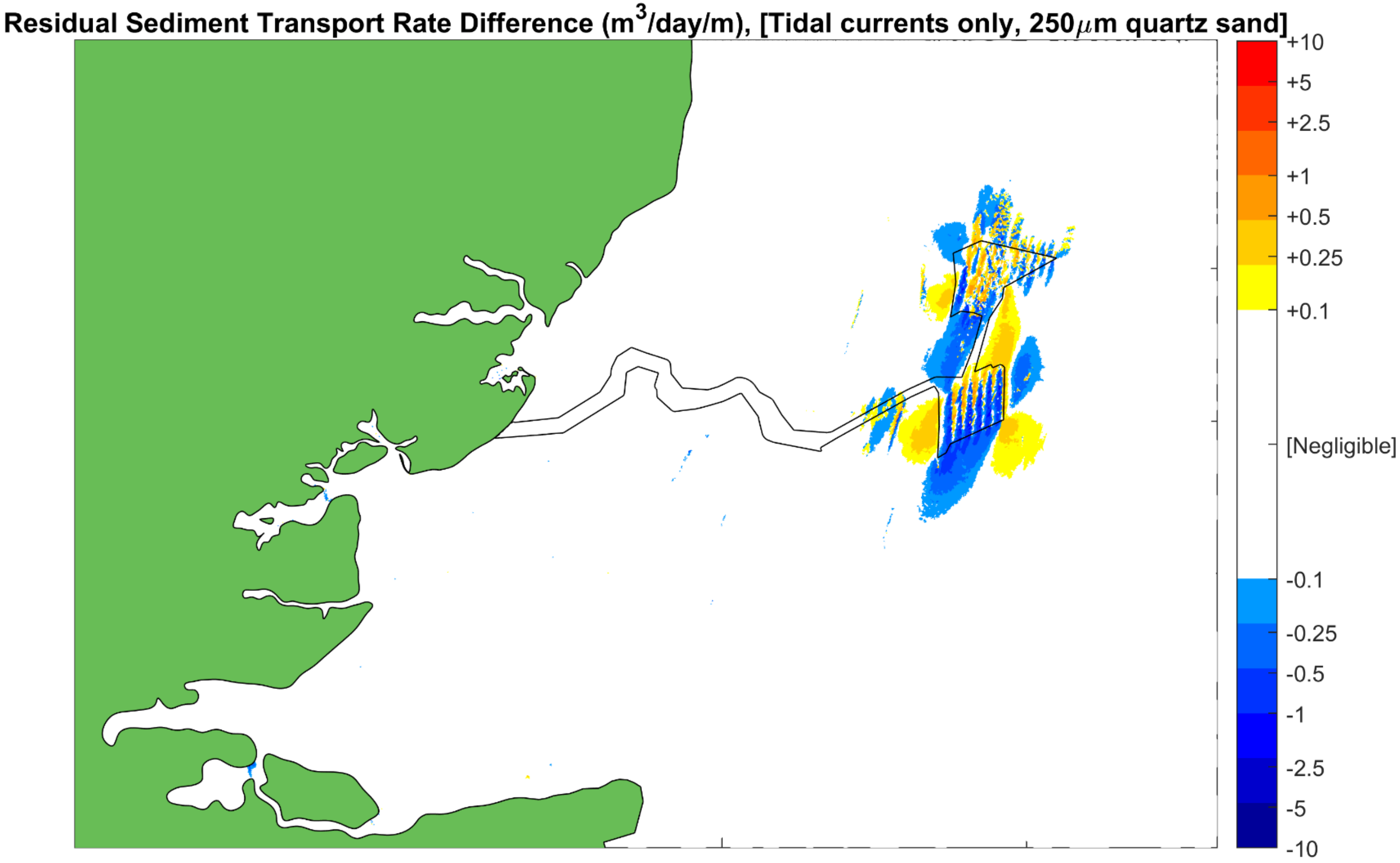


Figure C7. Absolute difference in residual sediment transport rate and direction, for 250 µm quartz sand, measured over a representative spring-neap tidal period, regional view. Negative and positive values are a relative reduction or increase in the residual sediment transport rate, respectively, as a result of the MDS infrastructure for Five Estuaries.

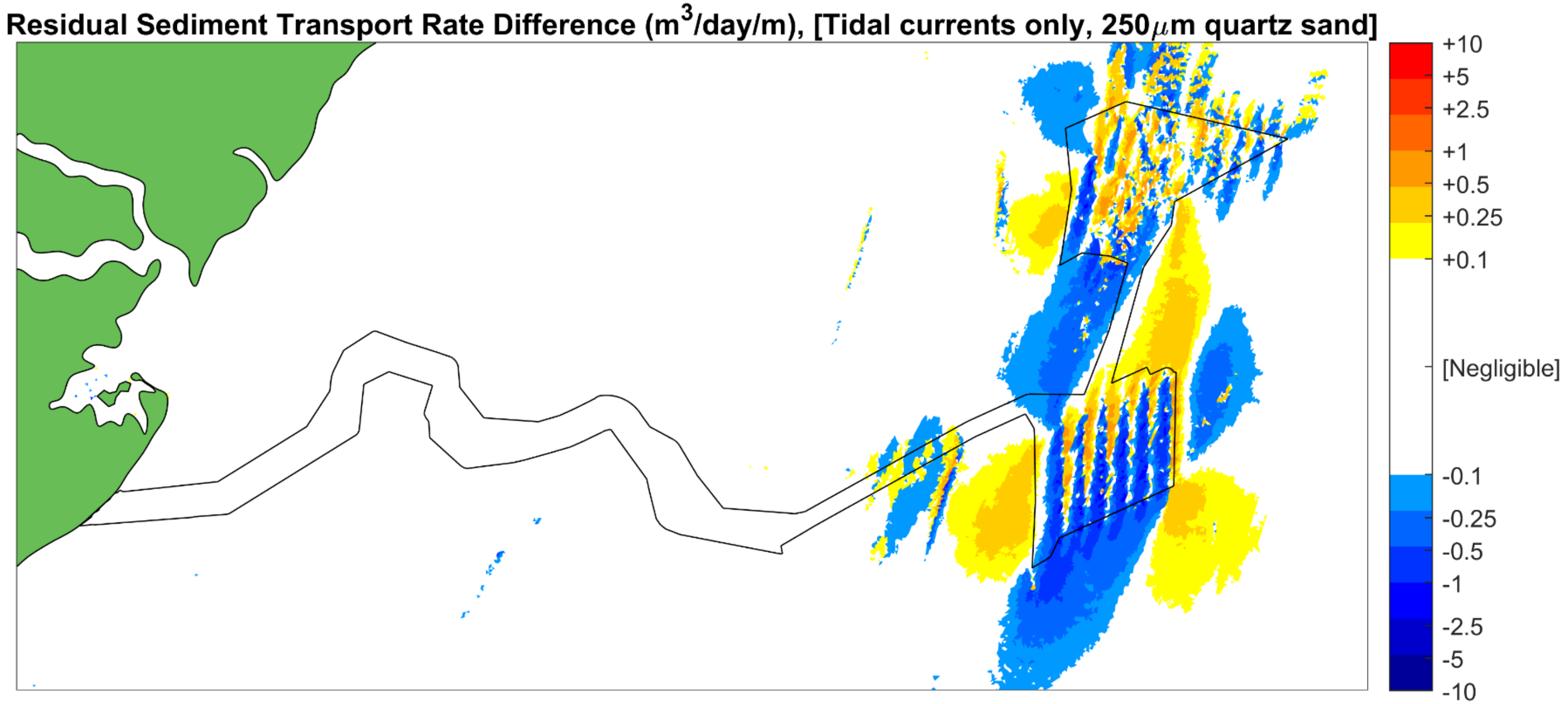


Figure C8. **Absolute difference in residual sediment transport rate and direction, for 250 µm quartz sand, measured over a representative spring-neap tidal period, detailed view. Negative and positive values are a relative reduction or increase in the residual sediment transport rate, respectively, as a result of the MDS infrastructure for Five Estuaries.**

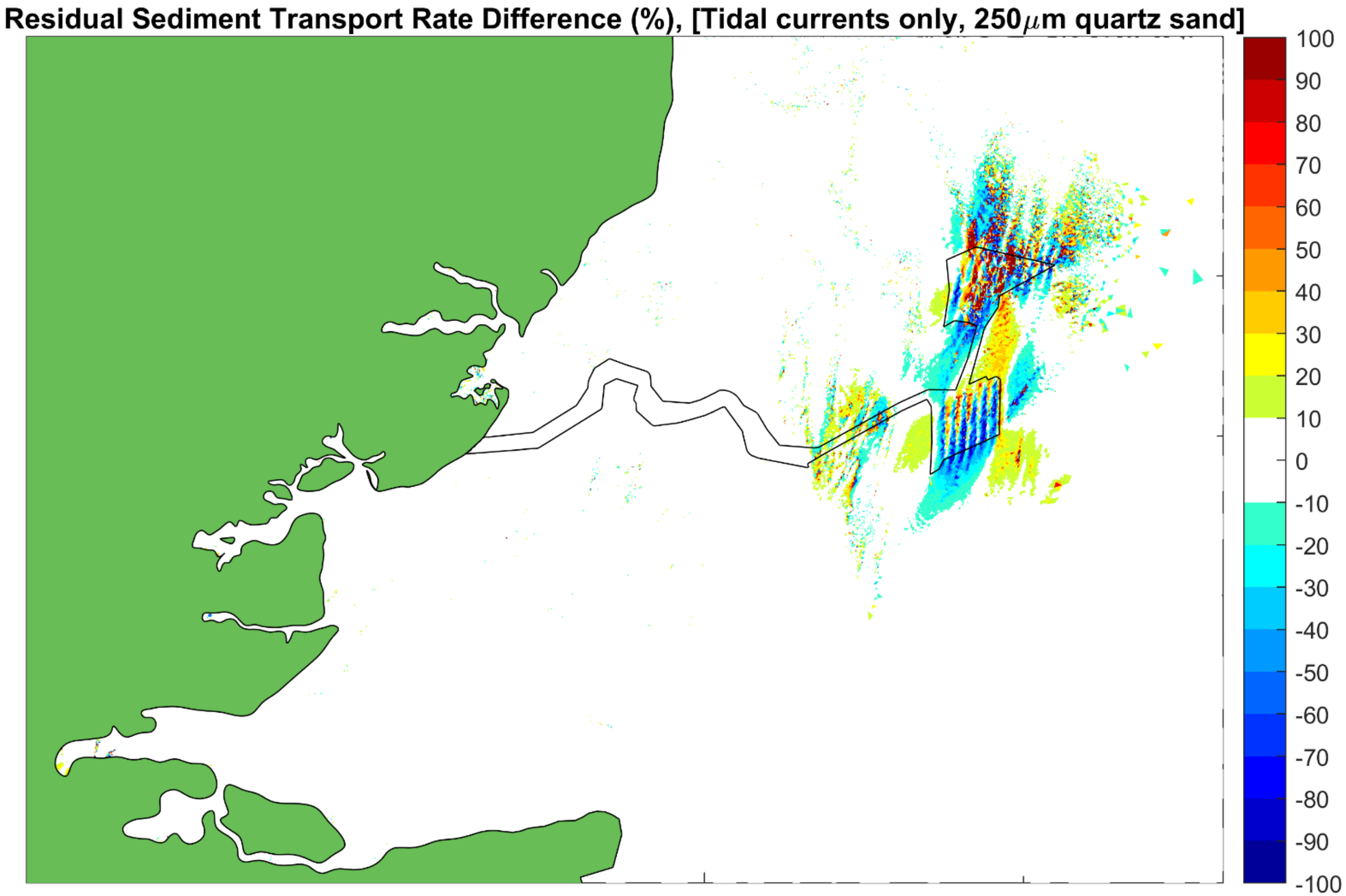


Figure C9. Relative difference in residual sediment transport rate and direction, for 250 μm quartz sand, measured over a representative spring-neap tidal period, regional view. Negative and positive values are a relative reduction or increase in the residual sediment transport rate, respectively, as a result of the MDS infrastructure for Five Estuaries.

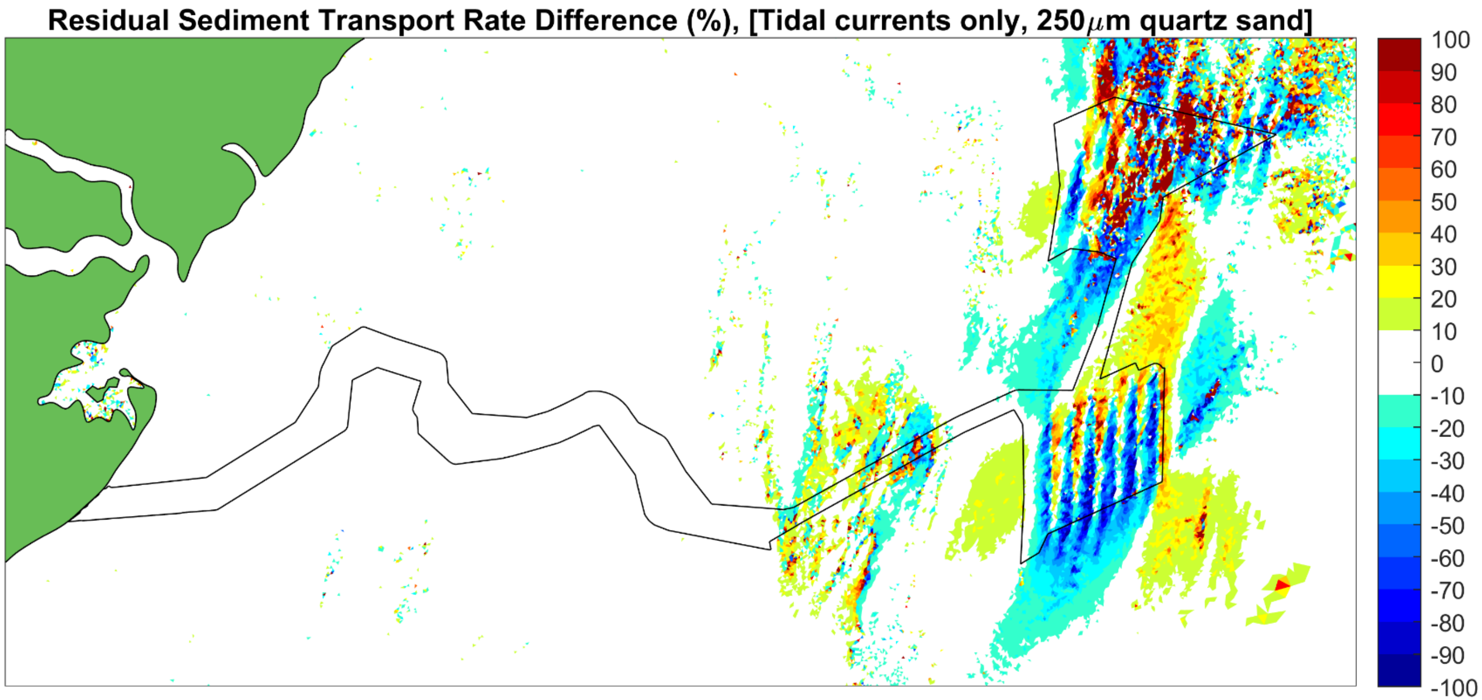


Figure C10. **Relative difference in residual sediment transport rate and direction, for 250 μm quartz sand, measured over a representative spring-neap tidal period, detailed view. Negative and positive values are a relative reduction or increase in the residual sediment transport rate, respectively, as a result of the MDS infrastructure for Five Estuaries.**

D Scour Calculations

D.1 Overview

In order to quantify the area of seabed that might be affected by scour (either the footprint of scour or scour protection), estimates of the theoretical maximum depth and extent of scour are provided below. Estimates are made of the primary scour, i.e. the scour pit directly associated with the presence of the main obstacle.

The equilibrium primary scour depth for each foundation type has been conservatively calculated assuming the absence of any scour protection, using empirical relationships described in Whitehouse (1998). This analysis considers scour resulting from the characteristic wave and current regime, both alone and in combination.

The project description (Volume 2, Chapter 1: Offshore Project Description) provides Maximum Adverse Scenario extents of scour protection for each foundation type. Scour protection might be applied around the base of some or all foundations depending upon the seabed conditions and other engineering requirements. By design, scour protection will largely prevent the development of primary scour, but may itself cause smaller scale secondary scour due to turbulence at the edges of the scour protection area.

D.2 Assumptions

The following scour assessment for VE reports the estimated equilibrium scour depth, which assumes that there are no limits to the depth or extent of scour development by time or the nature of the sedimentary or metocean environments. As such, the results of this study are considered to be conservative and provide an (over-) estimation of the maximum potential scour depth, footprint and volume. Several factors may naturally reduce or restrict the equilibrium scour depth locally, with a corresponding reduction in the area and volume of change.

This study makes the basic assumption that the seabed comprises an unlimited thickness of uniform non-cohesive and easily eroded sediment. In practice, the thickness of unconsolidated (and more easily erodible) surficial Holocene sediment is spatially variable across the VE study area, with the greatest thicknesses (typically 5 to 14 m, up to 15 to 17 m locally) found on and around the slope between the shallower western part and deeper eastern part of each array area (Fugro, 2022a). In other large parts of the site, the thickness of erodible Holocene sediments is relatively thin or absent (typically limited to up to 0.5 to 2 m) and underlain by relatively erosion resistant pre-Holocene (London Clay) material, which will limit the depth and extent to which scour can occur.

The foundation types, dimensions and numbers used in the assessment are consistent with the project design information provided in Volume 2; Chapter 1: Offshore Project Description.

Reported observations of scour under steady current conditions (e.g. in rivers) generally show that the upstream slope of the depression is typically equal to the angle of internal friction for the exposed sediment (typically 32° in loose medium sand; Hoffmans and Verheil, 1997) but the downstream slope is typically less steep.

In reversing (tidal) current conditions, both slopes will develop under alternating upstream and downstream forcing and so will tend towards the less steep or an intermediate condition. For the purposes of the present study a representative angle of internal friction (32°) will be used as the characteristic slope angle for scour development.

D.3 Equilibrium scour depth

The maximum equilibrium scour depth (Se) is defined as the depth of the scour pit adjacent to the structure, below the mean ambient or original seabed level. The value of Se is typically proportional to the diameter of the structure and so is commonly expressed in units of structure diameter (D).

Scour depth decreases with distance from the edge of the foundation. The scour extent (S_{extent}) is defined as the radial distance from the edge of the structure (and the point of maximum scour depth) to the edge of the scour pit (where the bed level is again equal to the mean ambient or original seabed level). This is calculated on the basis of a linear slope at the angle of internal friction for the sediment, i.e.:

$$S_{extent} = \frac{S_e}{\tan 32^\circ} \approx S_e \times 1.6$$

(Eq. 1)

The scour footprint ($S_{footprint}$) is defined as the seabed area affected by scour, excluding the foundation's footprint, i.e.:

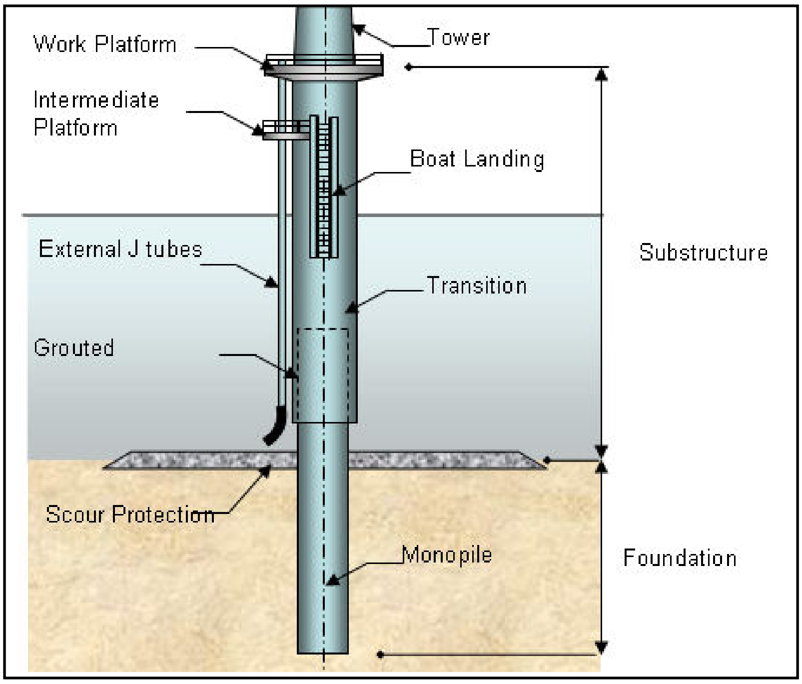
$$S_{footprint} = \pi \left(S_{extent} + \left(\frac{D}{2} \right) \right)^2 - \pi \left(\frac{D}{2} \right)^2$$

(Eq. 2)

The scour pit volume is calculated as the volume of an inverted truncated cone described by Equations 1 and 2 above, accounting for the presence of the foundation but excluding its volume.

D.4 Scour assessment method: monopiles

The outline design of the proposed monopile structure is shown in Figure D1.



Source: Garrad Hassan and Partners Ltd)

Figure D1. Outline design of a typical steel monopile foundation (with scour protection)

Compared to other more complex foundation types, scour around upright slender monopile structures in steady currents is relatively well-understood in the literature and is supported by a relatively large empirical evidence base from the laboratory and from the field. The maximum equilibrium scour depth, adjacent to the structure, below the mean seabed level (Sc), is typically proportional to the diameter of the monopile and is therefore expressed in units of monopile diameter (D).

D.4.1 Under steady currents

Breusers *et al.* (1977) presented a simple expression for scour depth under live-bed scour (i.e., scour occurring in a dynamic sediment environment) which was extended by Sumer *et al.* (1992) who assessed the statistics of the original data to show that:

$$\frac{S_c}{D} = 1.3 \pm \sigma_{Sc/D}$$

(Eq. 3)

Where $\sigma_{Sc/D}$ is the standard deviation of observed ratio Sc/D. Based on the experimental data, $\sigma_{Sc/D}$ is approximately 0.7, hence, 95 % of observed scour falls within two standard deviations, i.e., in the range 0 < Sc/D < 2.7. Based on the central value Sc = 1.3 D (as also recommended in DNV, 2016), the maximum equilibrium depth of scour for the maximum design scenario monopile (15 m diameter) is estimated to be 19.5 m.

D.4.2 Under waves and combined wave-current forcing

The mechanisms of scour associated with wave action are limited when the oscillatory displacement of water at the seabed is less than the length or size of the structure around which it is flowing. This ratio is typically parameterised using the Keulegan-Carpenter (KC) number:

$$KC = \frac{U_{0m} T}{D}$$

(Eq. 4)

Where U_{0m} is the peak orbital velocity at the seabed (e.g., using methods presented in Soulsby, 1997) and T is the corresponding wave period. Sumer and Fredsøe (2001) found that for KC < 6, wave action is insufficient to cause significant scour in both wave alone and combined wave-current scenarios.

Values of KC are < 6 for monopiles in the VE array area, for a range of extreme wave conditions (see Table E1) and for the full expected range of tidally affected water depths across the site (approximately 35 to 60 m). Therefore, it is predicted that waves do not have the potential to contribute to scour development around monopiles in the VE area.

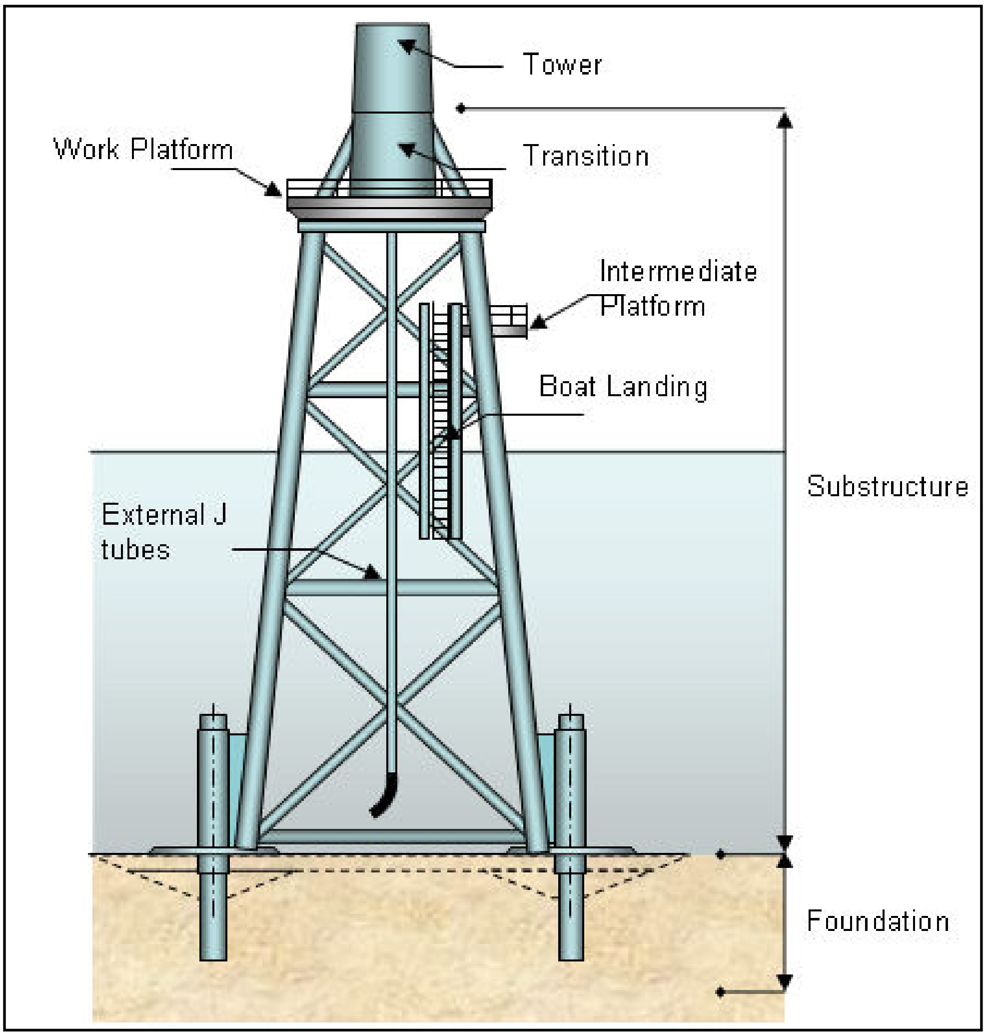
Table E1. Extreme omni-directional wave conditions considered

Return Period (years)	Significant Wave Height, Hs (m)	Zero Crossing Period, Tz (s)
1:1	5.2	6.2
1:10	6.7	7.0
1:50	7.7	7.5

The value of U_{0m} for given (offshore or deep water) wave conditions depend upon the local water depth, which varies between approximately 35 to 60 m within the array due to variations in absolute bathymetry and relative water level; the influence of shoaling and wave breaking have been ignored in the present study (a conservative assumption).

D.5 Scour assessment method: jacket foundations

The outline design of the proposed four-legged jacket foundation for turbines is shown in Figure D2. Above the seabed jacket foundations comprise a lattice of vertical primary members and diagonal cross-member bracing, up to 3.5 m in diameter; it is assumed that either no near-bed horizontal cross-member bracing is required, or that it is sufficiently high above the bed to not induce significant local scour. The four-legged jacket foundation will have a nominally square plan view cross-section with base edge dimensions of 40 to 45 m (Volume 2; Chapter 1: Offshore Project Description).



Source: Garrad Hassan and Partners Ltd

Figure D2. Outline design of a typical jacket foundation

The jacket foundation is anchored to the seabed at each corner by a pile driven into the seabed, 3.5 m in diameter. A jacket foundation structure may result in the occurrence of both local and group or global scour. The local scour is the local response to individual structure members.

D.5.1 Under steady currents

Under steady currents alone, the equilibrium scour depth around the vertical members of the structure base can be assessed using the same methods as for monopiles, unless significant interaction between individual members occurs. The potential for such interaction is discussed below.

The main scour development will be in proportion to the size of the largest exposed member near to the seabed. In this case, the largest exposed member will be the jacket leg which will have a diameter of up to 3.5 m. Using Equation 3, the scour depth for the largest jacket foundation is therefore estimated as 4.6 m.

In the case of currents, inter-member interaction has been shown to be a factor when the gap to pile diameter ratio (G/D) is less than 3. In this case limited experiments by Gormsen and Larson (1984) have shown that the scour depth might increase by between 5 % and 15%. However, in the case of the present study the gap ratio for members at the base of the jacket foundation structure is much greater than 3, and so no significant in-combination change is expected.

Empirical relationships also presented in Sumer and Fredsøe (2002) indicate that the depth of group scour (measured from the initial sediment surface to the new sediment surface surrounding local scour holes) for an array of piles similar to a jacket foundation (2x2) can be approximated as 0.4 D (i.e. approximately 1.4 m based on 3.5 m diameter jacket leg). On the basis of visual descriptions of group scour pits, their extent from the edge of the structure is estimated as half the width of the structure and following a broadly similar plan shape to that of the jacket foundation (i.e. square).

Together, the predicted maximum scour depth at the corner piles (4.6 m) and the group scour (1.4 m) is conservatively consistent with evidence from the field reported in Whitehouse (1998), summarising another report that scour depths of between 0.6 m and 3.6 m were observed below jacket structures in the Gulf of Mexico (although these could potentially be constrained from the maximum possible equilibrium scour depth by environmental factors and could also be subject to uncertainties in the seabed reference datum against which to measure the scour).

On the basis of the proposed jacket design, the diagonal bracing members are not predicted to induce seabed scouring due to the distance of separation from the seabed.

D.5.2 Under waves and combined wave-current forcing

Values of the KC parameter (Eq. 4) were calculated for a 3.5 m diameter jacket leg from the extreme wave conditions found at the site (Table E1)). Values of KC are less than 6 over the full expected range of tidally affected water depths across the site (approximately 35 to 60 m) and so it is predicted that waves do not have the potential to contribute to scour development around the base of the jacket foundations.

The diagonal bracing members will have a smaller diameter and so a larger KC value. However, they are again not predicted to induce seabed scouring due to the likely distance of separation from the seabed. For moderate KC numbers a sufficient distance to avoid scour is approximately one diameter for a horizontal member, increasing to approximately three diameters under increasing KC numbers.

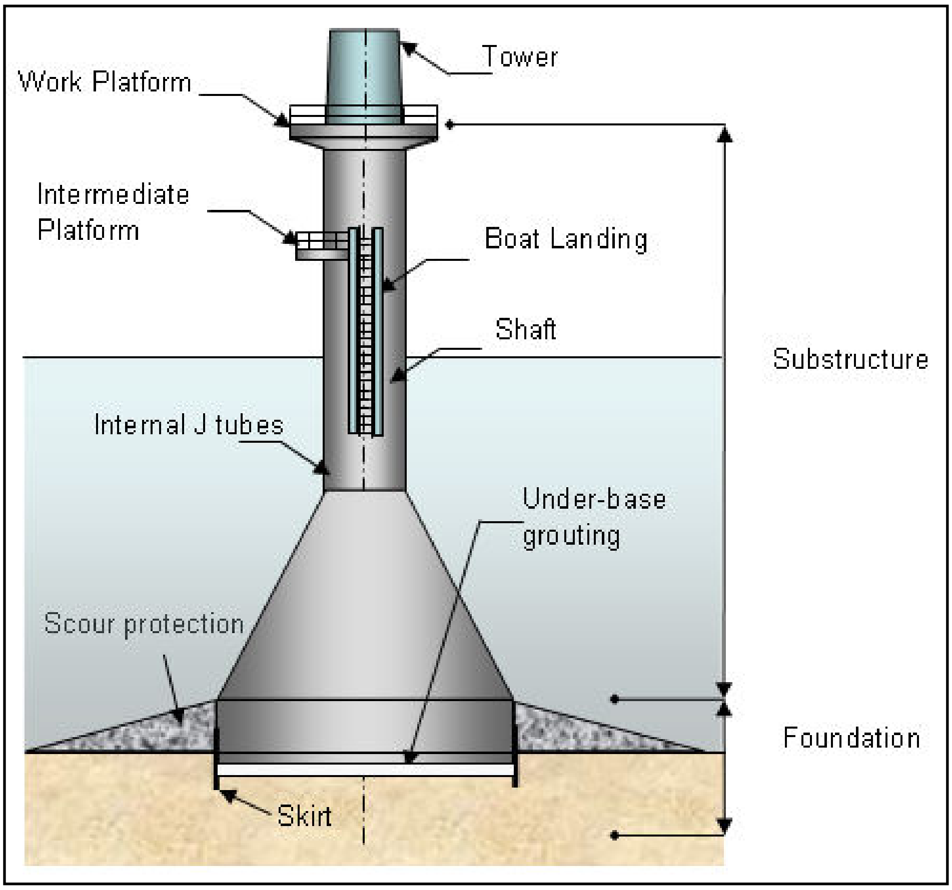
As such, little or no significant additional scour is predicted to result from waves, either alone or in combination with currents.

D.6 Scour assessment method: gravity base foundations

The outline design of the proposed gravity base foundation is shown in Figure D3. The foundation is characterised as a round base plate upon which sits a circular cross-section cone with a base diameter of up to 55 m (for all WTG sizes).

The evidence base for scour associated with gravity base foundation installations is relatively limited in comparison to that for monopiles and typically refers to oil and gas platforms which have a wide range of shapes and designs. Attempts to produce empirical relationships are complicated by this diversity of gravity base foundation structures.

The pattern and extent of scouring and the location of the point of maximum scouring may also vary depending upon the gravity base foundations relative size and shape. For the purposes of the present assessment, scour is assumed to be equally present at the predicted depth around the whole perimeter of the gravity base foundation, decreasing in depth with distance from the base edge to the ambient bed level at the angle of internal friction for the sediment (32°).



Source: Garrad Hassan and Partners Ltd)

Figure D3. Outline design of a gravity base foundation

D.6.1 Under steady currents

Hoffmans and Verheij (1997) presented the Khalfin (1983) current-only scour predictor for a gravity base foundation with the following modified features:

- The pile diameter was replaced by a characteristic length, Dc, taken as the average of the length and breadth of the gravity base foundation;
- The flow depth, h, in the water depth to diameter ratio h/Dc was replaced by the gravity base foundation height, hc; and
- The undisturbed depth-averaged flow velocity was multiplied by αc/2 with αc = 2 for a circular structure, and αc = 2.3 for a rectangular gravity base foundation expressing the additional turbulence generated at the corners of the structure. The coefficient αc is an influence factor that represents the flow enhancement near the structure caused by the structure.

The equilibrium scour depth, S, is then given by:

$$\frac{S}{D_c} = 8.96\left(2\frac{0.5a_cU}{U_{cr}} - 1\right)\left(\frac{h_c}{D_c}\right)^{1.43}\left(\frac{(0.5a_cU)^2}{gh}\right)^{N}$$

With U/Ucr = 1 for U>Ucr

And

$$N = 0.83\left(\frac{h_c}{D_c}\right)^{0.34}$$

Where:

Ucr is the value of depth-averaged flow velocity for initiation of sediment motion (m/s); and
g is the gravitational acceleration constant (9.81 m/s2)

(Eq. 5)

Assuming hc = h = 33 m and U>Ucr, the maximum equilibrium depth of scour for maximum design scenario gravity base foundation (Dc = 55 m) is estimated to be 3.1 m.

D.6.2 Under waves and combined wave-current forcing

The large scale of the gravity base foundation structures in relation to both water depth and wave orbital excursion length mean that the processes governing structure-flow interaction and scour are different from that described in relation to monopile and jacket structures. As such, relationships for scour associated with a shallow conical top gravity base foundation for waves alone are also not readily available from the literature. However, Whitehouse (2004) provides a relationship for a 'girder top' gravity base foundations, predicting equilibrium scour depth in response to waves alone of:

Se = 0.04D

(Eq. 6)

Yielding a value of 2.2 m for a 55 m diameter gravity base foundation. Empirical results from physical model testing by Whitehouse (2004) suggest that the maximum scour depth around a conical top gravity base foundation (broadly similar to that proposed here) under combined wave-current conditions will be:

Se = 0.064D

(Eq. 7)

Yielding a value of 3.5 m for a 55 m diameter gravity base foundation.

D.7 References

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