The Sizewell C Project

6.3 Volume 2 Main Development Site

Chapter 20 Coastal Geomorphology and Hydrodynamics

Appendix 20A Coastal Geomorphology and Hydrodynamics: Synthesis for Environmental Impact Assessment

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Sizewell Coastal Geomorphology and Hydrodynamics:

Synthesis for Environmental Impact Assessment (MSR1 – Edition 4)

BEEMS Technical Report TR311

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Please note that the red line boundary was amended after this document was finalised, therefore figures in this document do not reflect the boundaries in respect of which development consent has been sought in this application. However, amendments to the red line boundary do not have any impact on the findings set out in this document and all other information remains correct.

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Executive summary

This report is a synthesis of the coastal geomorphology and hydrodynamic evidence base relevant to the Sizewell C Environmental Impact Assessment (EIA). This synthesis follows the iteration of feeder reports during the course of studies undertaken to provide the evidence base for EIAs - regular updates which were prompted by changes in engineering design and comments/feedback from regulatory stakeholders during four meetings of the Sizewell Marine Technical Forum (comprising, with EDF Energy and Cefas, Natural England, Marine Management Organisation, Environment Agency and East Suffolk Council – plus the RSPB at the fourth meeting).

This Edition is based on Edition 3, but as it forms part of the Sizewell Environmental Statement (ES) certain sections that are detailed elsewhere in the ES have been removed. Where this occurs, the relevant ES section is cited. A more detailed evidence base can be found in a range of specific baseline and impact prediction feeder reports, which are cited throughout this synthesis. In addition to adjustments following written comments from regulators on Edition 3, new or substantially updated sections in Edition 4 are:

- Section 2.4.1: Future sea level (updated for UKCP18 alignment).
- Section 2.5: Resistance and Resilience of Coastal Geomorphology receptors (new).
- The temporary rock jetty has been removed as a method for the BLF deck construction.
- Sections 7.5 - 7.7: Future mitigation (beach maintenance) for potential HCDF impacts (substantial update).
- Appendix A: Natural England's condition assessment for Unit 113 on the Minsmere frontage.

This synthesis of evidence for EIAs discusses:

- the coastal geomorphology and hydrodynamics coastal geomorphology of the Greater Sizewell Bay;
- the marine components of the proposed Sizewell C development;
- the potential effects of the development on the coastal geomorphology receptor elements (shingle beach, sandy longshore bars, Sizewell – Dunwich Bank, and the subtidal Coralline Crag outcrops NE of Thorpeness);
- the potential effects of spatially and temporally overlapping Sizewell C development activities (inter-relationships) and those overlapping with other projects (cumulative impacts),
- high-level content regarding monitoring and mitigation, and
- the consideration of a likely future shoreline baseline that could result in future effects not apparent against the present baseline associated future mitigation, the trigger for mitigation action and cessation, and potential post-mitigation impacts.

The potential effects of the Sizewell C development on coastal geomorphology and hydrodynamics are dependent upon the engineering designs of specific coastal infrastructure. EDF Energy have not finalised the design of this infrastructure but the impact predictions undertaken are considered to follow the Rochdale envelope and capture the worst-case.

1 Introduction

This synthesis is the coastal geomorphology and hydrodynamics evidence base for the Environmental Impact Assessment (EIA and Environmental Statement (ES) of the proposed Sizewell C New Nuclear Build (NNB) power station (Volume 2, Chapter 20). It describes the baseline coastal geomorphology, the marine components of the Sizewell C power station (Sizewell C) and the potential effects of the proposed construction and operational activities on coastal geomorphology. It also provides evidence relevant to the EIAs and ES Chapters 21 and 22 (Volume 2) on Marine Water and Sediment Quality, and Marine Ecology, respectively. This synthesis does not include nuclear decommissioning of Sizewell C, as decommissioning will be the subject of separate environmental impact assessments at that time (considered in further detail in Chapter 5 of Volume 6 of this ES). However, the full life of the current and future marine structures is considered.

Although some feeder reports have been used in the evolving engineering design, this synthesis for the ES does not comment on engineering designs and how these were set. That content is primarily for the Nuclear Safety Case, which is assessed by the Office for Nuclear Regulation and is not duplicated in the Environmental Statement.

Previous editions of this report were designed to engage key marine regulators (Environment Agency, Natural England, Marine Management Organisation and East Suffolk Council – collectively, with EDF Energy and Cefas, the Marine Technical Forum (MTF)), in order to discuss baseline data and impact predictions, in order to avoid, or minimise possible impacts on coastal geomorphology. This report has developed over time in response to comments from these regulators as well as changes to the design of Sizewell C's marine components. This edition (Edition 4) is designed as an appendix of the ES, and hence the text in several sections has been removed and replaced by a cross-reference to the relevant section of the ES. In all it has been informed by four MTF meetings and comments on numerous technical reports, including this and previous editions of this report.

The elements of Sizewell C's Main Development Site that could have impacts on the coastal geomorphology and hydrodynamics of the Greater Sizewell Bay (GSB[1]) are:

- construction and operation of cooling water infrastructure (offshore cooling water intake and outfall headworks) and the nearshore outfalls for the Fish Recovery and Return systems (FRRs) and Combined Drainage Outfall (CDO);
- construction and operation of a Beach Landing Facility (BLF) to receive deliveries of Abnormal Indivisible Loads (AILs), rock armour and other marine freight by sea throughout the power station's operational life; and
- the construction and operation of a soft-Coastal Defence Feature (SCDF) and, some decades into the future, the operation a hard-Coastal Defence Feature (HCDF).

The receptor elements for Coastal Geomorphology and Hydrodynamics (Chapter 20) of the Sizewell C ES are (see Figure 1):

- the shingle beach;
- two sandy, shore-parallel longshore bars;
- the Sizewell–Dunwich Sandbank; and
- the erosion-resistant Coralline Crag that extends subtidally to the north-east from the Thorpeness headland.

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[1] The Greater Sizewell Bay extends from Walberswick to Thorpeness – see Figure 1.

1.1 Relevant legislation policy and guidance documents

Legislation, policy and guidance of relevance to the assessment of the potential main development site impacts associated with the Sizewell C Project, including the coastal geomorphology assessment are identified and described in Volume 1, Chapter 6 of the ES (Appendix 6P).

The key legislation, policy and guidance documents relevant to the EIA for coastal processes and hydrodynamics are:

- Marine and Coastal Access Act (2009)
- Overarching National Policy Statement for Energy (EN-1)
- Suffolk Shoreline Management Plan (SMP7, Zone 4: Dunwich Cliffs to Thorpeness)

See Section 20.3 of this chapter for a full list.

The Main Development Site is situated in an ecologically diverse area and, as a result, is subject to a range of nature conservation designations. The statutory designated sites relevant to Coastal Geomorphology that may be affected by the Sizewell C development are shown on Figure 1 and include:

- Minsmere to Walberswick Heaths and Marshes SAC,
- Minsmere to Walberswick SPA,
- Minsmere to Walberswick Heaths and Marshes SSSI, and
- Leiston to Aldeburgh SSSI.

Supra-tidal shingle can support the annual vegetation of drift lines (Annex I, habitat type 1210) and potential for nesting little tern. Natural England condition surveys show that this habitat was in decline on the Minsmere frontage (Unit 113) in 2004 and was destroyed between 2010 and 2011 (Appendix A). The non-statutory Suffolk Shingle Beaches County Wildlife Site also features supra-tidal shingle adjacent to Sizewell B, which is above MHWS and is assessed in the Terrestrial Ecology and Ornithology Chapter of the ES (Volume 2, Chapter 14). The Orfordness – Shingle Street SAC and the Orford Inshore MCZ have been scoped out as they are outside of the Zone of Influence (ZoI) and have no pathway to impact for geomorphology.

1.2 Outline of what is covered in this synthesis

This synthesis of evidence for the Coastal Geomorphology and Hydrodynamics of the Sizewell C ES (Volume 2, Chapter 20) primarily draws upon technical feeder reports, as well as other scientific literature. Only information relevant to the baseline environment and the potential effects of the development are included.

Overall, this synthesis presents:

- a description of the coastal geomorphology and hydrodynamics coastal geomorphology of the GSB;
- a description of the marine components of the proposed Sizewell C development;
- the potential effects of the development on the coastal geomorphology receptor elements (shingle beach, sandy longshore bars, Sizewell – Dunwich Bank, and the subtidal Coralline Crag outcrops NE of the Thorpeness headland);
- the potential combinations of spatially and temporally overlapping development activities (inter-relationships) and those overlapping with other projects (for cumulative impacts),
- high-level content regarding monitoring and mitigation, and
- consideration of a likely future shoreline baseline that could result in future effects not apparent against the present baseline, future mitigation, cessation of mitigation and post-mitigation impacts.

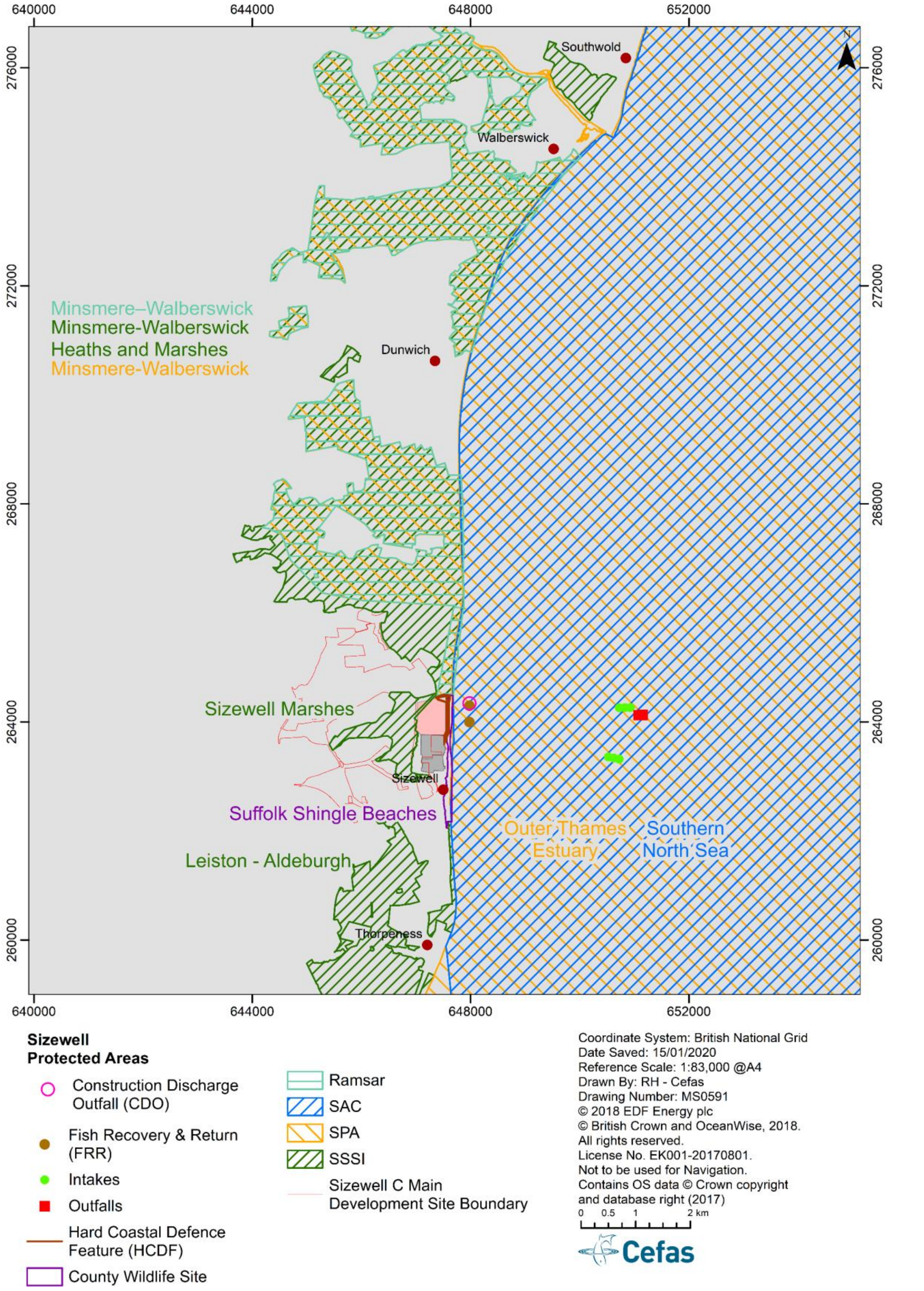

Figure 1: Statutory and non-statutory designated sites within the GSB whose geomorphic receptors could be affected by Sizewell C.

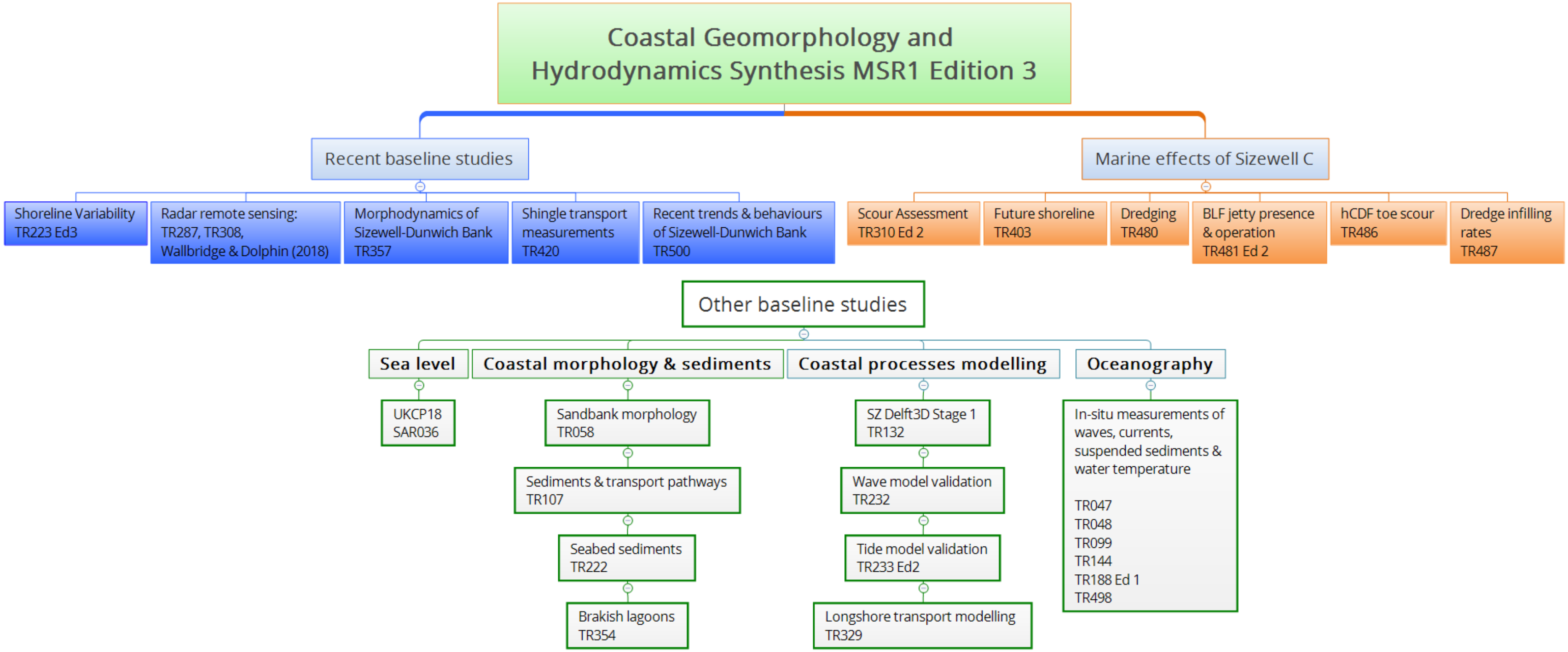

Figure 2: Key feeder reports (blue and orange background) developed to support baseline studies and prediction of effects of Sizewell C.

2 Coastal geomorphology and hydrodynamics of the Greater Sizewell Bay

2.1 Aim

This section summarises the aspects of the Greater Sizewell Bay's coastal geomorphology and hydrodynamics that are relevant to the proposed Sizewell C marine infrastructure and EIA.

2.2 Evolution of the Greater Sizewell Bay

The GSB (Figure 3) and the Suffolk coastline developed following a phase of rapid sea level rise (a post-glacial marine transgression) between 8000 and 5000 BP (before present day) (Eisma *et al*., 1981; Shennan *et al*., 2000). By 6000 BP, the general morphology, bathymetry and tidal regime observed today had been established, however, the region's weak cliff structure (sandy sediment of the pre-glacial Norwich Crag Formation) was easily eroded leading to subsequent long-term shoreline retreat and the release of large volumes of sediment into the nearshore coastal system. After 5000 BP, the rate of (relative) sea level rise slowed dramatically, and presently is around 2.5 – 4.3 mm/yr ± 0.83 mm (Woodworth (2009) and BEEMS Technical Report TR252, Table 4) as a result of slow net crustal subsidence and globally rising sea levels.

A shingle-barrier separates the centre of the GSB from the low lying Minsmere Levels. It can be divided into three sections based on its volume and elevation:

- The **Northern Barrier** is north of the Minsmere Outfall and has a relatively low volume (270–800 m^3 per metre of beach length (m^3/m)) and is occasionally overtopped, particularly north of the Coney Hill cross-bank (also known as the North Wall). Sedimentary deposits on the top and landward flank of the barrier are evidence temporary breaches and overtopping during high energy storms and elevated (storm-surges during NE storms) water levels.
- The **Central Barrier** extends from just north of the outfall for around 1500 m to the south, has a large volume (800–1650 m^3/m), is not overtopped and erodes by scarping.
- The **Southern Barrier** extends south to Sizewell C, has intermediate volumes (940–1200 m^3/m), presently erodes by scarping and is not morphologically evolving as a result of overtopping. However, small shingle deposits (< 10 cm thick, 3 m wide and 10 m long) have been found in two locations, indicating that overwash has occurred but not with any measurable changes to morphology. Additionally, during the 1:250 year December 2013 storm surge, some minor overwash deposits were found behind a high berm on the Sizewell frontage. Overwash on the Southern Barrier is infrequent and is not presently causing roll-back, degradation, building or breaching of the barrier.

Although protected from the sea by the barrier and drained via the Minsmere Sluice (built in 1830), the Minsmere Levels have experienced marine incursion on a number of occasions since 3390 (± 60 cal. years BP) and were deliberately flooded during World War II as an anti-invasion measure. Limited natural saline incursion occurs today via groundwater (e.g., saline lagoons described in BEEMS Technical Report TR354). Larger volumes are intentionally introduced to some of the lagoons at the Minsmere Reserve via the Minsmere Sluice when the tidal stage is near high water.

The general shape of the GSB derives from substantive coastal erosion and accretion events of the 19th century, which are considered to have been driven by several decades of stormy conditions and the release of large volumes from erosion of the Minsmere – Dunwich cliffs. From the 1830s to the 1880s, the cliffs and beaches north of Minsmere Outfall retreated rapidly (averaging -2.3 m/yr for half a century), although there was little net change around the outfall. During the same period, the coast between the outfall and Thorpeness accreted significantly (+1.7 m/yr), resulting in a wide beach/dune system fronting the former cliffs at Sizewell. The resulting broad anticlockwise re-orientation of the shoreline about the Minsmere Outfall is considered to have been driven by a prolonged period of north – north-easterly storms (as deduced by Pye and Blott (2006) from Lamb's (1995) interpretation of patterns in climate data). This phase of erosion in the north and accretion in the south concluded with the onset of the present 70+ year phase, which is typified by a fluctuating patchwork of erosion and accretion with relatively low overall rates of change.

The present regime is considered to be the result of:

- a change from the energetic NE unidirectional wave climate of the 19th century to the present more balanced NE – SE bidirectional one;
- an overall reduction in inshore wave energy due to growth of the Sizewell – Dunwich Bank (elevation, width and extent), which is thought to have been a sink for some of the material eroded from Minsmere – Dunwich Cliffs during its 19th Century erosive phase (BEEMS Technical Report TR223 Edition 3); and
- the presence of headlands at natural and man-made hard points – Thorpeness' Coralline Crag, Minsmere Outfall and the Blyth river mouth jetties – within an otherwise soft and erodible coast.

2.3 Contemporary coastal geomorphology and hydrodynamics

2.3.1 Geomorphic elements of the Greater Sizewell Bay

The GSB is anchored in the north by the Blyth river jetties and in the south by the Thorpeness headland and its underlying erosion-resistant Coralline Crag, which outcrops sub-tidally (Figure 3). The main morphological features/receptors of the GSB are:

- the shingle beach/barrier;
- two sandy, shore-parallel longshore bars;
- the Sizewell–Dunwich Bank; and
- the Coralline Crag ridges that outcrop sub-tidally, extending to the north-east from Thorpeness.

The intertidal beach is primarily comprised of shingle (i.e., gravel-sized material) with a smaller sand-fraction that is either mixed with shingle or exists as surface, or sub-surface, veneers (BEEMS Technical Reports TR420 and TR480). The seaward limit of the shingle beach is an abrupt beach-step that meets a sub-tidal, low sloping, sandy bed. This boundary demarcates the seaward limit of the shingle beach and indicates that cross-shore exchange of shingle occurs almost exclusively landward of the low-tide beach step.

The low net rates of longshore transport on the Sizewell power stations frontage, which are due the balanced bi-directional wave climate (see Section 2.3.2.2), give rise to very low rates of shoreline change. Net shoreline change rates are also low around the Minsmere Outfall, which acts like a long groyne, partially blocking longshore transport during storms. In contrast, there is persistent shoreline erosion c. 1 – 2 km either side of the outfall (see Section 2.3.6).

Landward of the continuous shingle beach are cliffs (Minsmere – Dunwich and Sizewell – Thorpeness) or low-lying hinterlands (Walberswick Marshes and the Minsmere Levels). A shingle barrier has crest elevations ranging 2.4 – 7.2 m above Ordnance Datum Newlyn (ODN[2])) and separates the Minsmere Levels (c. 0.3 m ODN[3]) from the sea along that frontage.

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[2] All topographic and bathymetric elevations in this report are with respect to ODN. Positive values are above ODN and negative values are below ODN.

[3] Average elevation immediately landward of the barrier between the Minsmere Sluice and Sizewell C

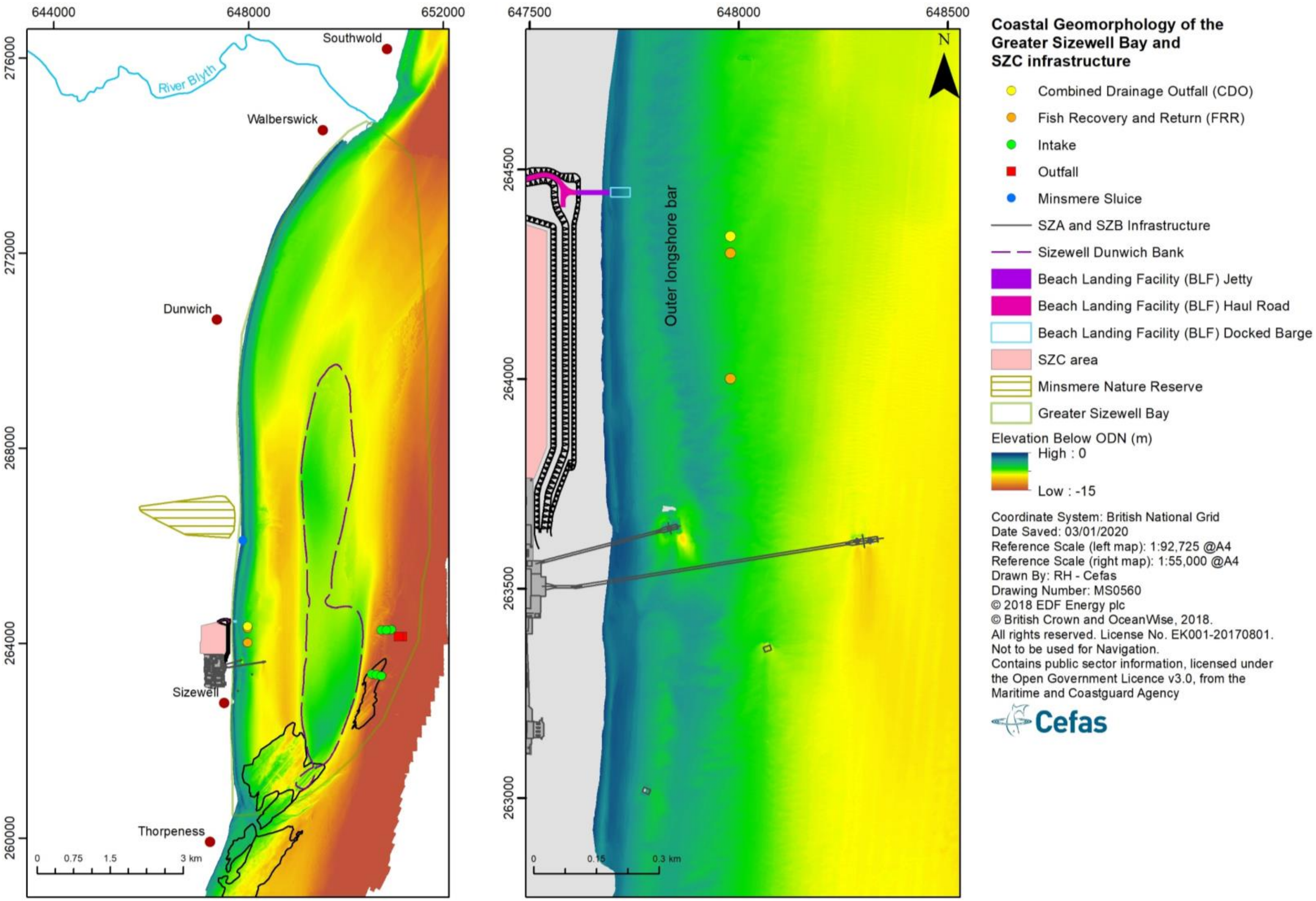

Figure 3: Location map of the Sizewell C frontage, coastal bathymetry and coastal geomorphology.

The subtidal beach is sandy and features an inner longshore bar 50 – 150 m from shore of -1.0 to -3.0 m (ODN)[4] elevation, as well as a larger outer bar 150 – 400 m from shore of -2.5 to -4.5 m (ODN) elevation. The bars are approximately shore-parallel and play an important role in dissipating wave energy (through wave breaking) and reducing wave angle at the shore/bar line (which controls longshore transport – see Section 2.3.4.2). During larger storms, when both bars are part of the surf zone, high suspended sand concentrations will drive sand transport along the bar crests and troughs, which accounts for most of the low annual (net average) ca. 10,000 m^3 of southerly sediment transport (BEEMS Technical Report TR329). That is, the bars are the primary sand transport corridor during storms.

Seaward of the bars, a 1200-m-wide channel (up to 9 m deep) separates the coast from the Sizewell – Dunwich Bank. Whilst primarily sandy, muds (white – blue in Figure 4) are found in a narrow stretch just landward of the bank. Muddy sediments also dominate the area to the north of the Dunwich end of the bank, whilst the bank itself is comprised of well-sorted fine-sands.

The Sizewell – Dunwich Bank is a single sedimentary feature, 8 km in length and with a landward flank located 1.2 – 1.7 km from shore. Its higher north and south ends, often referred to as Dunwich Bank (-4 to -5 m elevation) and Sizewell Bank (-3 to -5 m elevation) respectively, are joined by a lower elevation saddle (-7 m elevation). Due to its large size (633 ha above the -8 m contour; BEEMS Technical Report TR500) the bank is not regularly surveyed, however it is apparent in recent soundings and radar data that it can remain stationary for several years or longer. Historical records indicate that the bank tends to migrate landward at an average rate of 6 – 7 m/yr in its central and northern sections (BEEMS Technical Report TR058, Section 3.5). Records over the last decade show that Sizewell Bank has remained static in its position. However, the development of a 300 m wide, 600 m long, northward extending spur along its seaward flank increased bank height locally by 0.4 – 1.0 m.

In contrast, Dunwich Bank exhibited greater variability in both its morphology and position with:

- erosion north of 267000N, resulting in bank lowering of -0.5 – -1.5 m,
- a decrease in its northern extent of approximately 250 m,
- landward movement (200 – 475 m) of the northernmost 2.75 km of its seaward flank,
- accretion/migration on its landward flank adjacent to its peak and most landward position (between approximately 267000N – 267600N), and
- ongoing migration of the landward flank for the -6 to -10 m (ODN) contours (approximately -6 m/yr) (BEEMS Technical Report TR500).

Growth in Sizewell Bank is considered to be sustained by sand supply from the coast. There are several strands of evidence supporting the coast to bank sand transport pathway:

- trends in sediment size and colour;
- bedform orientation;
- patterns of erosion and accretion observed over successive bathymetric surveys;
- sediment build up (accumulations) and release episodes seen in radar data;
- the size and north-east orientation of Coralline Crag ridges; and
- modelled hydrodynamics and sediment transport.

These factors support net southward movement of sand along the longshore bars, which accumulates near the apex of Thorpeness where it is funnelled seaward to the bank by the north-east oriented Coralline Crag ridges (BEEMS Technical Reports TR107, TR308 and TR357, and Aldridge *et al*. (in prep)).

[4] Although not apparent in BEEMS or EA bathymetric surveys, the inner bar was emergent in the summers of 2018 and 2019, suggesting beach building summer conditions and/or an abundance of sand.

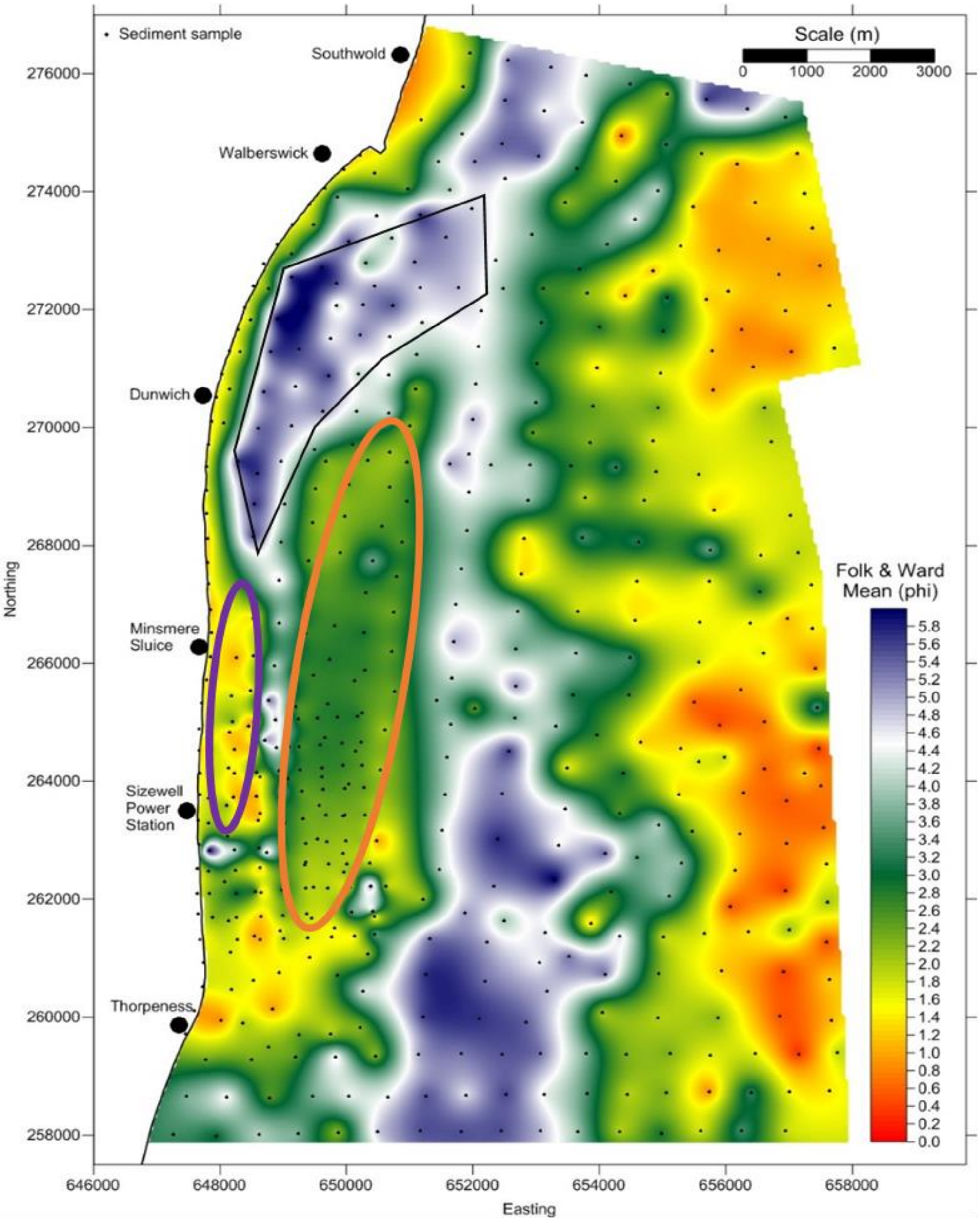

Figure 4: Mean sediment diameter (phi or φ units[5]) map. Sample locations are shown as black dots. Medium sands (0 – 1.5 φ / 355 – 1000 μm), fine sands (1.5 – 4 φ / 63 – 355 μm) and mud (> 4 φ / 63 μm) shown (adapted from BEEMS Technical Report TR107). Features shown are: sandy nearshore zone (purple circle), Sizewell-Dunwich Bank (orange circle) and mud patch north of the sand bank (black polygon).

The erosion resistant Coralline Crag outcrops at Thorpeness form a shallow platform and a series of descending shallow ridges that extend seaward (north-east) to Sizewell Bank (Figure 3). Sediment grabbing is difficult in this area where the ridges are exposed or only thinly covered in sediment. The presence of the crag at Thorpeness fixes the location of the headland, which subsequently controls the local tidal streams

[5] The Krumbein phi (φ) scale, is a logarithmic scale estimate as follows: $\varphi = -\log_2 D/D_0$ with φ the Krumbein phi scale, D the diameter of the particle or grain in millimetres and D_0 a reference diameter, equal to 1 mm.

(e.g., offshore diversion of the ebb stream) that maintain the bank's stable form[6]. The Coralline Crag outcrops between Thorpeness and the bank, and on the seaward side of the bank; its presence underneath the bank may have influenced its initial formation and added to its positional stability.

2.3.2 Hydrodynamics setting

2.3.2.1 *Tides*

The tidal currents in the GSB are semi-diurnal (M2 and S2 dominant). The tidal range increases from North to South across the region with spring tides of 1.9 m at Lowestoft, 2.2 m at Sizewell and at 3.5 m at Felixstowe. Water movement is dominated by tidal currents that flow south for most of the rising (flood) tide (1.14 m/s (peak) seaward of Sizewell Bank) and flow north for most of the falling (ebb) tide (1.08 m/s). The water column is thermally well-mixed throughout the year due to the strong tides and shallow bathymetry. The only exception to this is in the vicinity of the Sizewell B discharge plume, but this is of insufficient spatial extent to affect the flow regime. As expected, tidal currents reduce close to shore and peak at about 0.2 m/s 50 m from the shoreline (BEEMS Technical Report TR481).

The TELEMAC2D tidal flow model was used to simulate the tidal regime of the GSB (see BEEMS Technical Report TR233 Edition 2 for details). The model was run for the validation period (7/11/2013 to 6/12/2013) to enable a direct comparison between model and observed data. As the tidal stream is strongly rectilinear and north – south aligned, the V axis captures most of the tidal current (Figure 5), which flows toward 6° - 10° on the ebb tide and toward 186° - 190° on the flood tide.

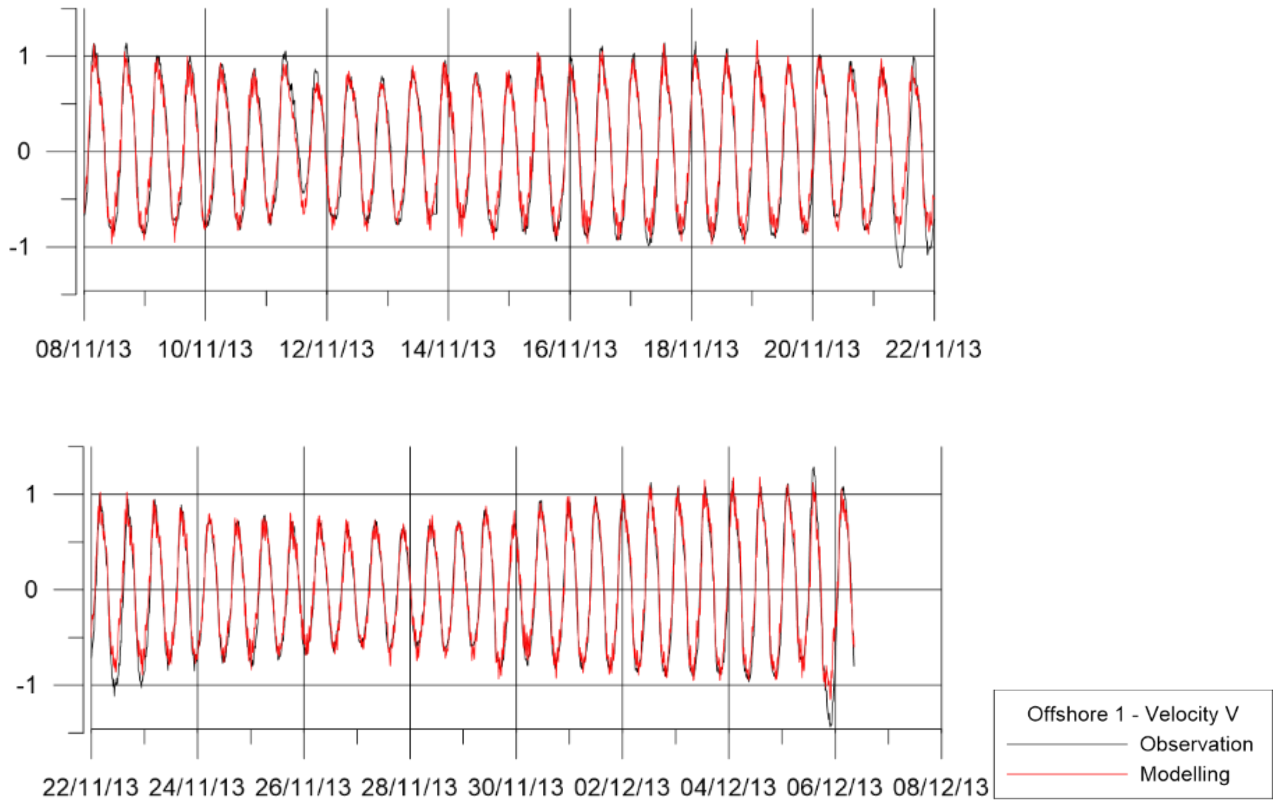

Figure 5: Comparison of observed and modelled offshore V component of velocity, which is approximately aligned with the tidal axis (BEEMS Technical Report TR233 Edition 2).

[6] The historic stability of the Sizewell end of the bank can be linked to the fixed position of the crag ridges; in comparison the northern Dunwich end is more mobile and has no anchoring feature.

2.3.2.2 *Waves*

2.3.2.2.1 Regional nearshore wave climate

The EA deployed several nearshore Acoustic Wave and Current Profilers (AWACs) in c. 5 m water depth around the East Anglian coast for three years (2006 – 2009), five of which were on the Lowestoft – Felixstowe coast. In Figure 6, the AWACs' locations are presented along with the wave roses that highlight the typical bidirectional wave climate of the Suffolk coast, the balance of which varies with location although most sites are fairly evenly balanced. Covehithe shows an almost symmetrical bimodality, whereas just 6 km to the south the bimodality is asymmetric with a slight dominance from the northeast / east north-east. Further south at Slaughden, the waves approach almost equally from the two directions but for significant wave heights (Hs) > 1.5 m, the wave climate is unidirectional with almost all waves approaching from the east north-east sector. The wave energy levels south of Orford Ness are significantly less than those experienced along the central Suffolk coast. Shoreline movement, cliff erosion and longshore transport rates along this section of the Suffolk coast exhibit a high degree of spatial variability (see respectively Figure 67 and Section 3.2.3 of Environment Agency 2011), which is partly driven by spatial variability in local wave climate.

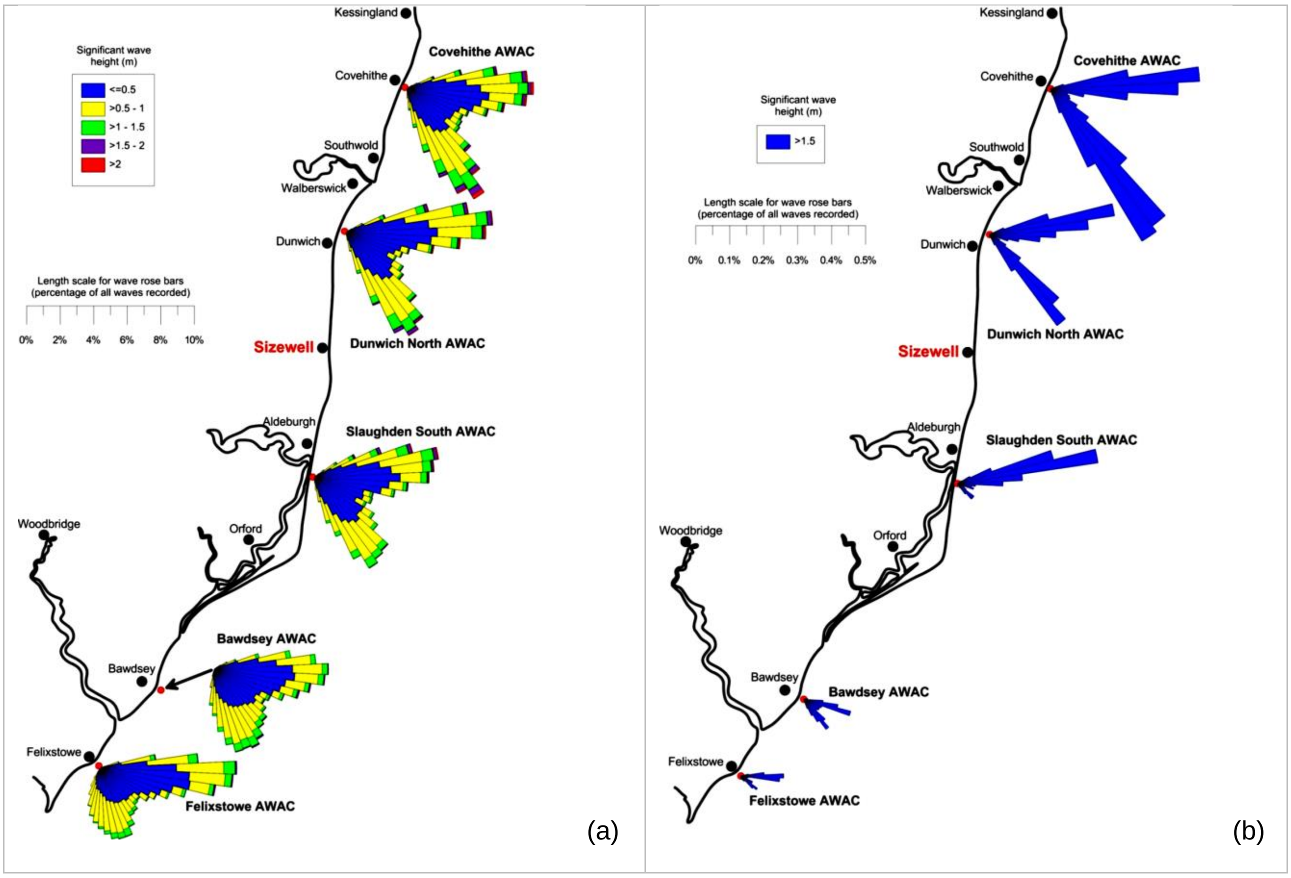

Figure 6: Regional inshore wave roses from EA deployments. Regional wave roses for (a) all data and (b) Hs > 1.5 m. Data were collected by AWAC gauges on the Suffolk Coast for the period October 2006 and September 2009 (BEEMS Technical Report TR139, original data source: Environment Agency, operated by Gardline, disseminated via Cefas Wavenet).

2.3.2.2.2 Sizewell offshore wave climate

The offshore wave climate at Sizewell is monitored with a Datawell Directional Wave Recorder buoy (DWR), 4 km from shore, just seaward of Sizewell Bank and in 18 m depth of water. The main features of the wave climate there, based on the ten-year record, are:

- The bidirectional wave climate (Figure 7), with the most frequent waves arriving from NE (23.16%), S (20.25%) and SE (15.13%). Most waves (93%) have periods less than 8 seconds. For the decade 2008-2018, wave heights greater than 1.5 m occurred 7.87% of the time (directions from east-north-east and south).
- The largest fetch is towards the north (up to 3,000 km) and correspondingly the largest and longest waves arrive from the N-NE sector. Waves with periods greater than 8 seconds approach exclusively from the NE – ENE sector.
- Waves from the south through south-east sector are generated over a much shorter fetch (up to 150 km) and are therefore typically smaller than waves from the north, even though 4 m waves have been recorded, propagating from the south – south east.
- Although the long-term wave climate is almost directionally balanced, individual winters tend to be dominated by either NE or SE events, with the longest persistent phase lasting for four years (BEEMS Scientific Position Paper SPP094).
- Although easterly waves are less common (15%) and fetch limited, large waves can also be generated under strong air flows arising when depressions tracking south of the British Isles combine with a weakening polar vortex, such as the Storm Emma and Beast from the East event in March 2018. That event produced a peak Hs of 4.31 m and had a long storm duration of 65 hours (Hs > 2.1 m); see BEEMS Scientific Position Paper SPP094.
- The wave energy density plots in Figure 8 show the energy balance between the two directions, with a slight dominance from the north-easterly sector.

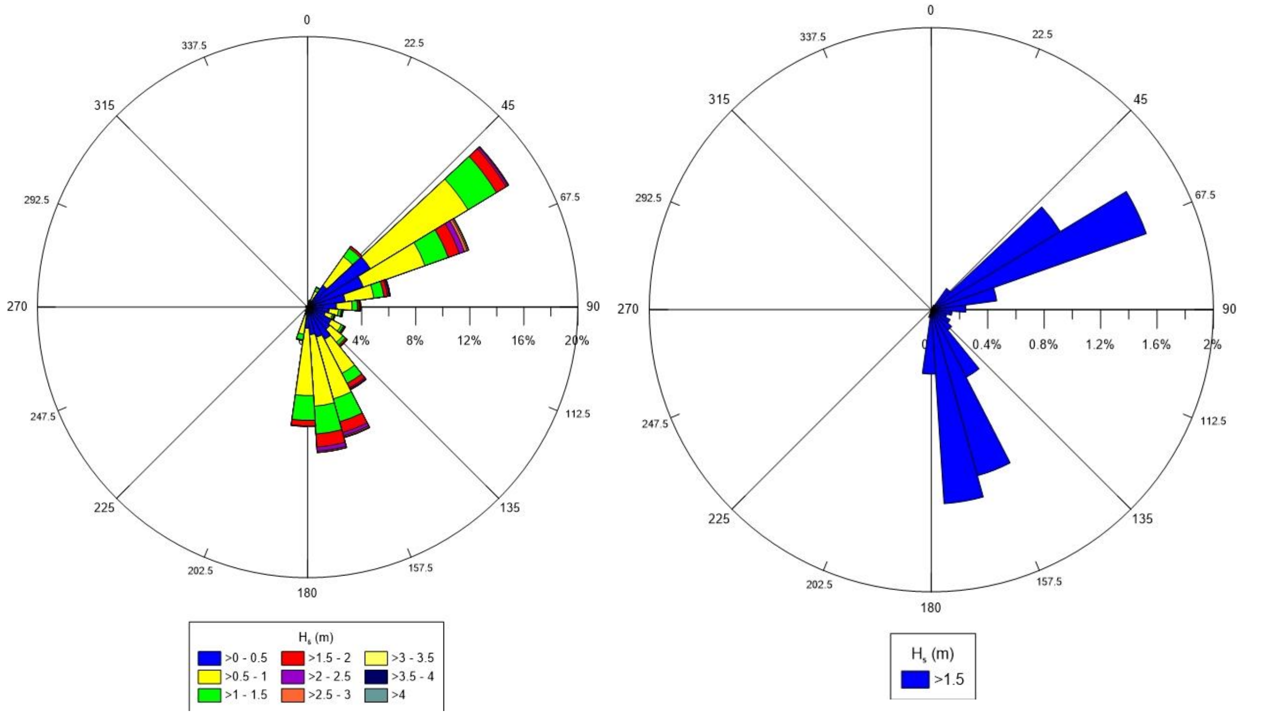

Figure 7: Wave roses (Hs) for the Sizewell DWR (left panel) and for Hs > 1.5 m (left panel). Record length is 11/02/08 – 17/09/2018.

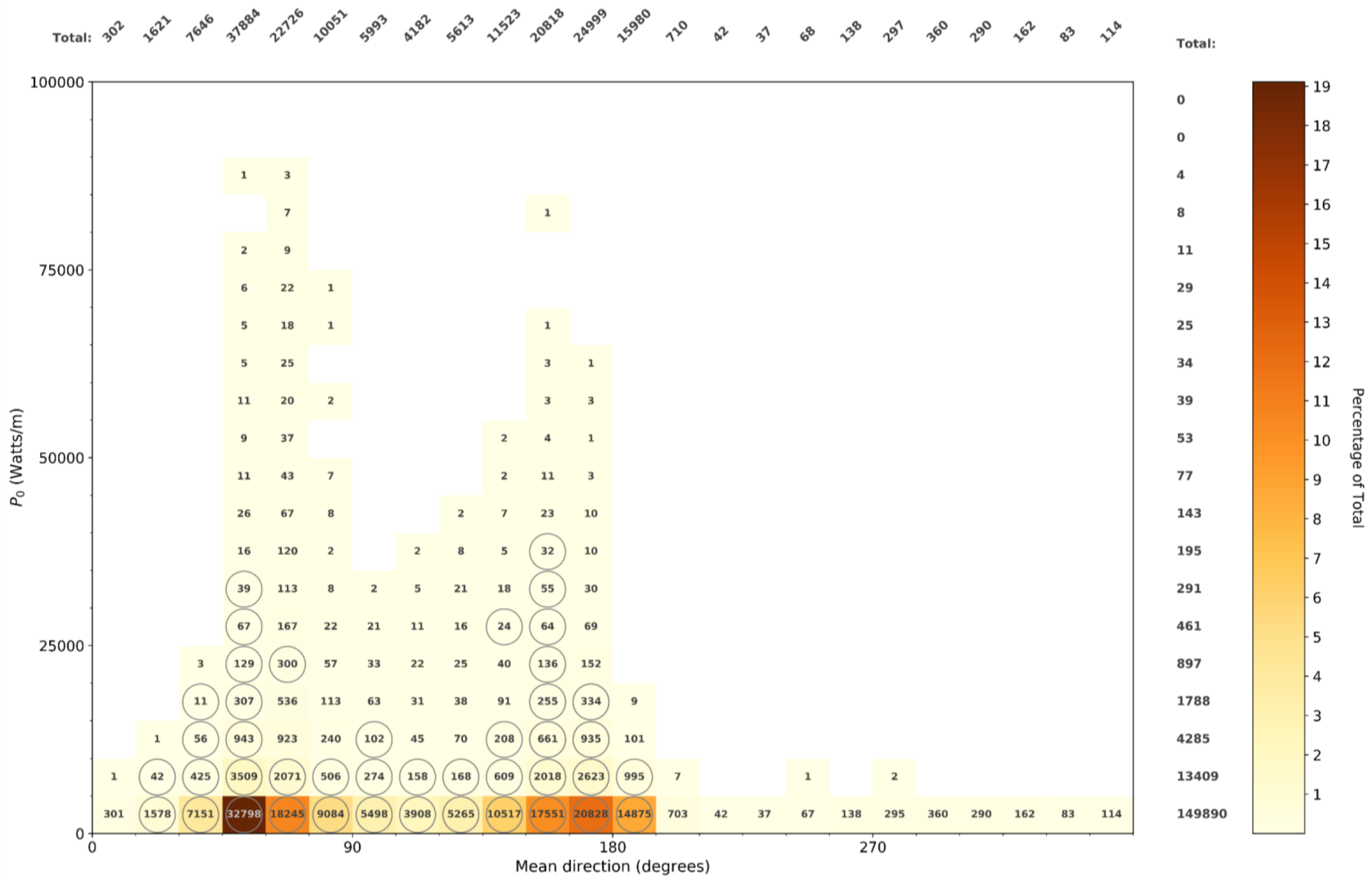

Figure 8: Wave energy density plotted against wave peak direction (30° bins) from the Sizewell DWR (11-02-2008 to 01-06-2018). Colours represent counts and circles refer to measurements coinciding with gravel transport study (see Section 2.3.4.2).

2.3.2.2.3 Sizewell nearshore waves

The waves acting on the shingle beach face and, during elevated water levels on its landward barrier, are substantially lowered before arriving at the shore due to dissipation across the GSB's three positive relief features; the Sizewell – Dunwich Bank and the two longshore bars. Coastal sandbanks and longshore bars dissipate wave energy through bottom friction in shallow water (e.g., over the 1-km-wide sand bank) and wave breaking, when the water depth is less than about 1.3 times the wave height (i.e., waves larger than about 3-4 m approaching the GSB). As waves shoal across these morphologies, they are also refracted toward a more shore-normal direction of travel, which reduce longshore transport potential.

The inshore wave climate (between the bank and the bars) has been recorded by several BEEMS AWAC deployments (2006, 2008, 2013, 2016 and 2019) and reported in various BEEMS Technical Reports (BEEMS TR139 (2006/2008), BEEMS TR317 (2013/2016)). AWAC locations per deployment are shown in Figure 9. In summary, characteristics of the inshore wave climate revealed by these deployments include:

- For waves just inshore of the bank (around 10 m depth), the 2013 deployment showed a lesser directional change (relative to waves offshore) at a gauge in the lee of the deeper saddle compared with a second gauge in the lee of Dunwich Bank. This is due to refraction over Dunwich Bank bending to waves by 12-15° toward a more shore normal direction of travel. Even though the deployment was dominated by NE waves, the same gauges also provide some evidence of similarly increased refraction of SE waves over the higher part of the Sizewell Bank compared to the deeper saddle.
- Three AWACs deployed within 600 m of the shoreline (water depths 6-8 m) in 2016 (300 m alongshore spacing) showed more subtle differences in the wave direction. At this nearshore location (directly in front of Sizewell B) it was the SE waves which were refracted 12-15° toward shore normal relative to the two gauges 300-600 m further north (while NE waves were similar at each location).
- Overall, from all nearshore deployments mean wave height did not exceed 0.7 m, whilst the maximum wave height recorded from AWACs was approximately 3 m in the 2016 record. The mean of the peak spectral wave period parameter (Tp) was 6 seconds.

Earlier studies (Carr, 1981; Tucker *et al*. 1983), the BEEMS AWAC deployments and numerical modelling show that very large offshore waves (typically around 4 m) tend to break on the Sizewell-Dunwich Bank (Figure 10). Tucker *et al*. (1983) showed that small waves experience negligible attenuation over the bank, but that wave heights during storms were reduced inshore of the bank (Figure 10). Effectively, the bank imposes a cap on inshore wave height (BEEMS Technical Report TR232, Carr (1981) and Tucker *et al*. (1983)), as a result of wave breaking during larger storms - inshore waves just seaward of the outer bar tend to be capped at about 2.5 m (BEEMS Technical Report TR319).

Wave modelling has demonstrated both refraction and wave height limiting due to the bank (BEEMS Technical Report TR319[7]). Large waves (greater than 4 m in height offshore; return intervals > 5 years) break extensively on the bank (as marked by the sharp change in colour from red to yellow-green in Figure 11) capping the wave heights there and inshore. The wave condition modelled in Figure 11 shows wave heights around 0.5 m higher than in the lee of the saddle compared to the lee of the higher elevation Sizewell and Dunwich Banks. The streamlines shown on Figure 11 also illustrate wave refraction over the bank – as well as the over the nearshore topography (the colour transition to blue shows energy losses on the double bars before the beach-face which, due to their shallower depth, induce more wave breaking than the bank).

As the bank elevation and width are not uniform, larger waves can penetrate to the outer bar through bank sections where the water is deeper, and vice-versa. However, despite larger waves penetrating through the deeper bank saddle, wave diffraction and the variation in storm direction means that there is no persistent alongshore pattern in wave height near the coast, nor in shoreline behaviour. Net shoreline stability at the Minsmere Outfall occurs because of sediment trapping (see Section 2.3.6.3).

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[7] BEEMS Technical Report TR319 used TOMAWAC coastal model forced by a 30 year hindcast model.

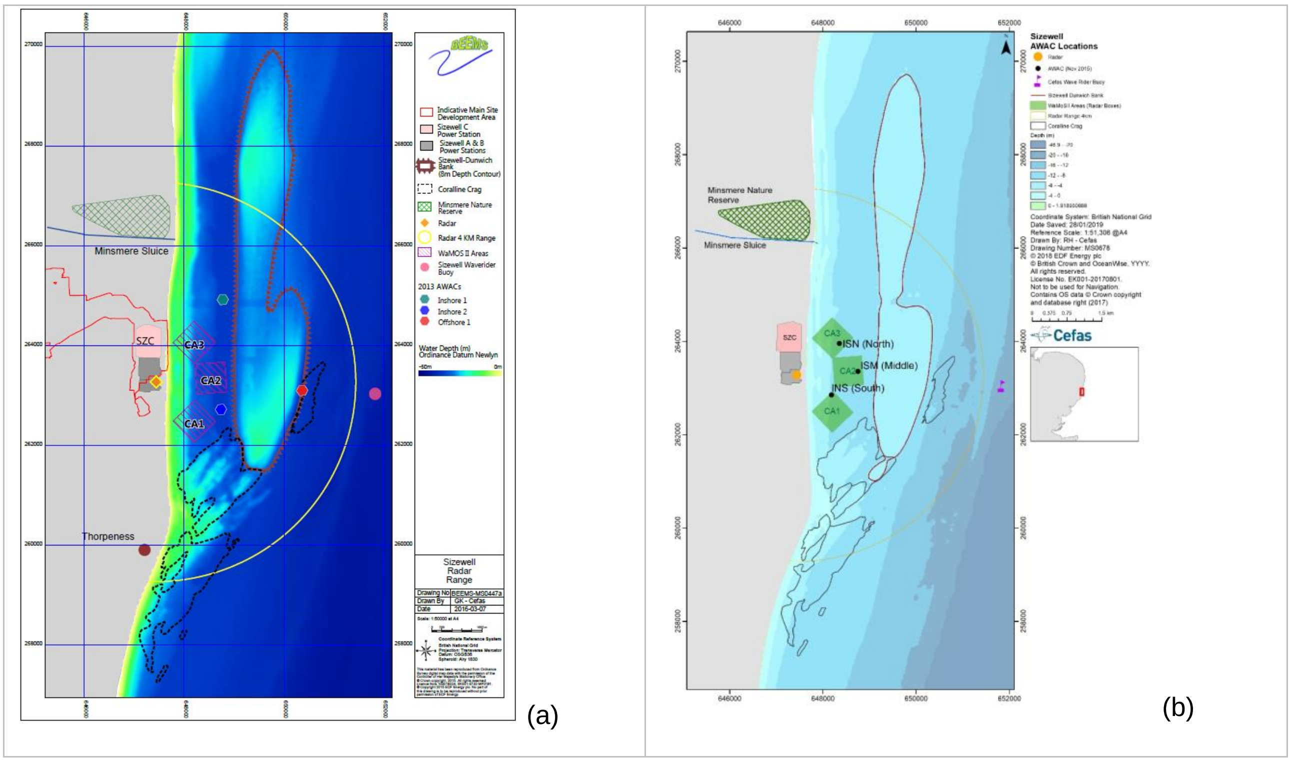

Figure 9: AWAC locations for (a) July – October 2013 and (b) January – April 2016 and February – May 2019.

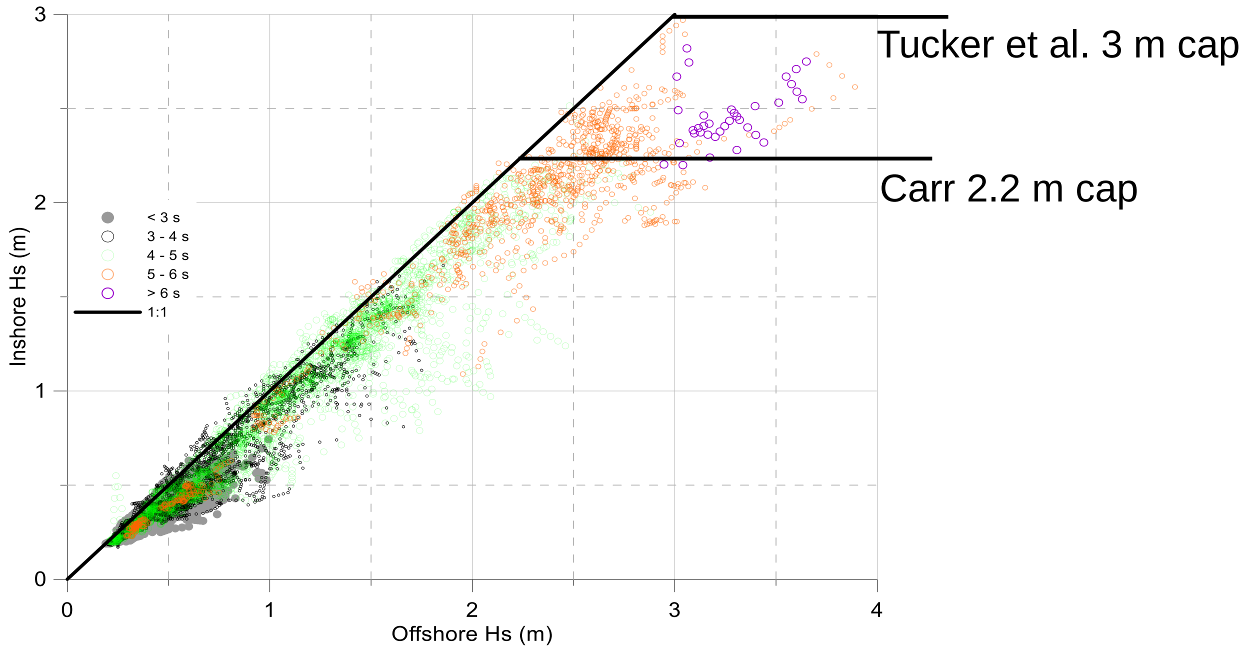

Figure 10: Comparison of significant wave height offshore and inshore of Sizewell – Dunwich Bank showing the height reductions that result from energy dissipation due to bottom friction and wave breaking. Also marked are the 'caps' (maxima) to inshore wave height proposed by Carr (1981) and Tucker *et al*. (1983) as a result of wave breaking. Symbols are coloured by wave period.

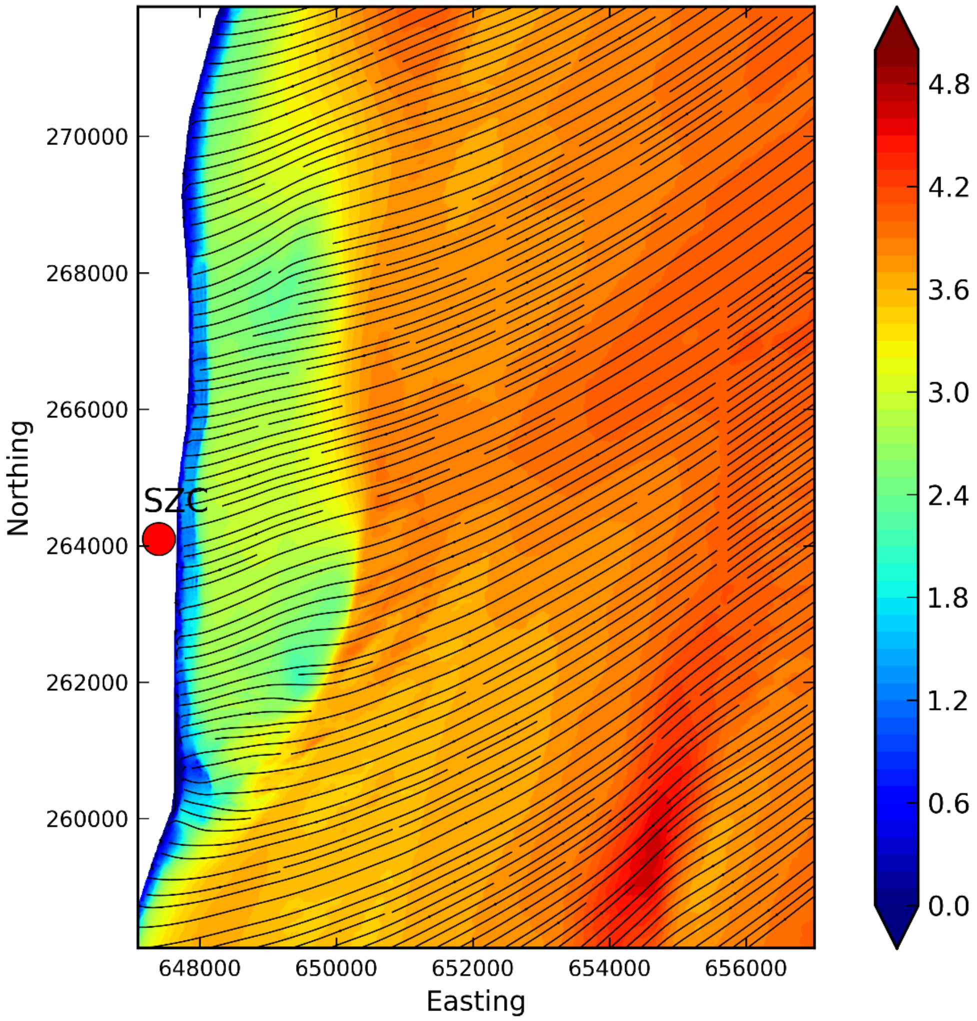

Figure 11: Modelled wave height (m) for a 1:5-year NE incoming storm for the current bathymetric conditions; the colour bar indicates significant wave height (m) whilst the streamlines show wave directionality.

2.3.3 Sediment supply

The primary potential sources of new sediment entering the GSB are the Minsmere – Dunwich Cliffs (within the embayment) and the Easton – Covehithe Cliffs (2.5 – 10.5 km north of the embayment). These cliffs comprise unconsolidated pre-glacial (Pliocene to early/mid Pleistocene) marine sediments (Norwich and Red Crag) that are weakly bound and are predominately sandy, with some gravel (shingle) (up to 60%) and mud (up to 15%) deposits (Pye and Blott, 2006). The older and underlying Coralline Crag (early/middle Pliocene) is well-cemented and more erosion resistant; its presence at Thorpeness fixes the headland there and is considered to aid sandbank stability due to its persistent deflection of the tidal streams. Observations and numerical models indicate that the ness is the site of convergence between net southward transport along the Sizewell frontage and net northward sediment transport between Aldeburgh (to the south) and Thorpeness (e.g., BEEMS Technical Reports TR223, TR357 and TR420; also Burningham and French (2016) and Atkinson and Esteves (2019)), and that little sand/shingle sediment bypasses the ness. It is therefore, the southern boundary of the sediment cell and SMP7 Zone 4.

Although severely eroding in the 19th and early 20th centuries (up until 1926; based on an assumed change in wave climate), Sizewell – Dunwich Bank grew and the Minsmere – Dunwich Cliffs erosion rate more than halved in 1926 – 1970 and has been near zero since with almost no new Minsmere – Dunwich Cliff sediment being added to the coastal system.

In comparison, the Easton and Covehithe Cliffs are actively eroding and releasing sand into the coastal system. These are the fastest eroding cliffs in Britain – during the period 1992–2008 the mean rate of retreat of the Benacre Cliffs was 7.02 m/yr, whilst the long-term rate (1883–2010) at Covehithe is 3-4 m/yr (BEEMS Technical Report TR107). Brooks and Spencer (2012) modelled shoreline retreat and cliff erosion using the SCAPE model for a range of future sea level scenarios until 2050 and 2095 and showed that up to 460 ha of land could be lost. Utilising data on cliff composition and topographic elevation, the sediment volumes released under different sea level scenarios was shown to rise from the measured value of:

- 178,500 m^3/yr in 1992-2008 to
- 270,100 m^3/yr under a 4.4 mm/yr[8] sea level rise in 2008–2050 and rising again to
- 299,500 m^3/yr[9] for 6.7 mm/yr sea level rise in 2050–2095.

In regional terms these results indicate a rise in sediment supply, from the north, to the GSB.

2.3.4 Sediment transport

The following description of sediment transport patterns is drawn from literature, sediment and bedform patterns in BEEMS bathymetry data, four longshore shingle tracer experiments (in 2017 and 2018) and numerical modelling (BEEMS Technical Reports TR329 and TR357). The GSB is characterised by four main sedimentary features (Figure 4) – the shingle dominated intertidal beach and backing supra-tidal barrier, the sandy nearshore zone (with two longshore bars), Sizewell – Dunwich Bank, and the mud patch north of the Dunwich end of the sand bank.

2.3.4.1 *Subtidal sand transport*

Modelling shows a general net southward transport within the GSB (Figure 12). Landward of the Sizewell-Dunwich Bank, there is a net southward sediment transport between the Minsmere-Dunwich Cliffs and Sizewell (BEEMS Technical Report TR357). On the seaward side of the bank, the net transport is also southward, except along the south-eastern flank where sediment patterns and modelling results show localised northward transport (Figure 12). Additionally, seaward vectors highlight an area of very slow potential sediment loss from the bank, although it is not sufficient to cause morphological change. Persistent tidal bedload and suspended load converge at Sizewell Bank, as well as a weaker convergence around Dunwich Bank, reflects the likely mechanism for bank maintenance.

Although the path of sandy sediment within the longshore bars has not been directly measured, several strands of evidence provided in Section 2.3.1 indicate funnelling offshore toward Sizewell – Dunwich Bank by the rocky Coralline Crag ridges at Thorpeness.

Successive marine surveys of the Sizewell – Dunwich Bank, indicate that there is no present sediment transport mechanism that could give rise to seaward migration; trends over the last 70+ years to date have shown stability (Sizewell) or landward migration of the bank flanks (particularly of the landward flank on the saddle and Dunwich Bank). Hydrodynamic modelling (BEEMS Technical Report TR357) also suggests that the flows patterns that maintain the bank in its present position would still occur even if there were no bank at this location – implying that sediment accumulation at the bank position is a natural condition and that the bank position is therefore likely to be enduring.

[8] Based on the 50th percentile of the UKCP09 medium emissions scenario

[9] Based on the 95th percentile of the UKCP09 medium emissions scenario

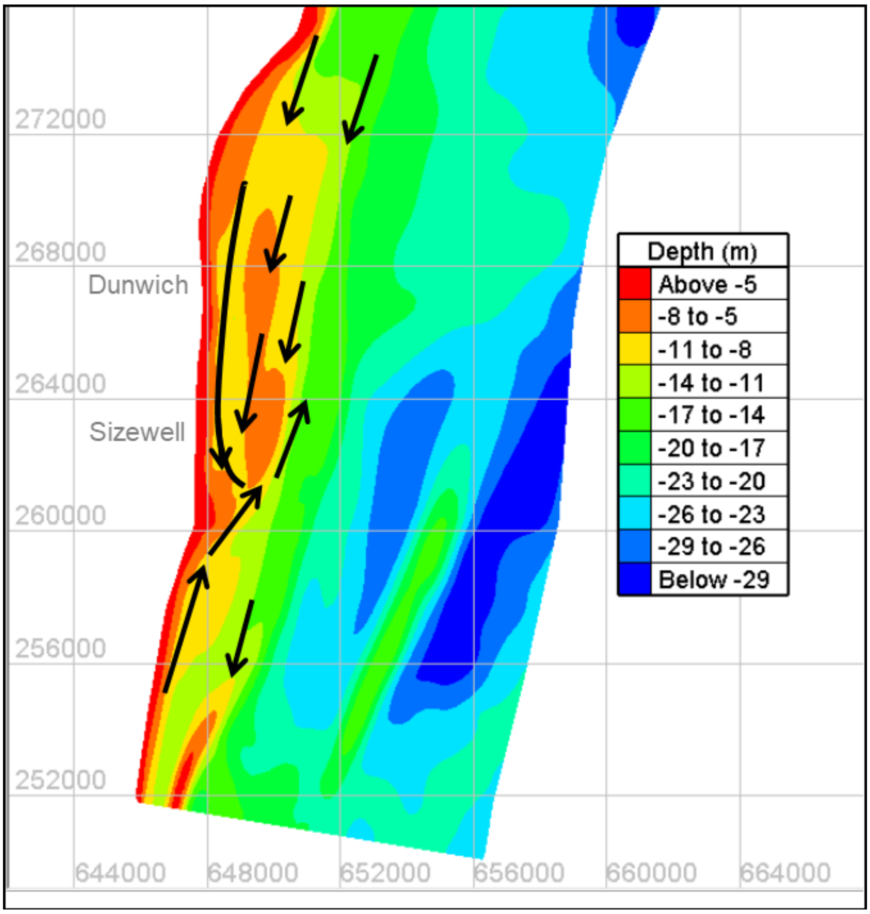

Figure 12: Schematic subtidal sand transport pathways deduced from numerical modelling (BEEMS Technical Report TR357).

2.3.4.2 *Longshore shingle transport*

Shingle is the term widely used in the UK to describe gravel-sized sediments (2 – 63 mm ϕ). Shingle moves by sliding, rolling and saltation (under high energy conditions) during competent wave activity (breaking and swash) (Carter and Orford, 1984). At Sizewell, shingle is confined to a narrow corridor between the beach toe and the dune/barrier line. Sub-tidal grab samples taken immediately adjacent to beach were sandy (BEEMS Technical Report TR238), indicating that there is little or no exchange of beachface shingle with the subtidal. Although models of longshore transport show a net southerly drift, geomorphic evidence (e.g., no route for loss yet no new inputs) suggests that the shingle is more or less static, in net terms.

A shingle transport tracer study was conducted along the beach at Sizewell (between Minsmere and Thorpeness) to understand the net transport rate and residence times of shingle in the Sizewell Bay area (i.e. how closed or open the sediment system is). For full details of the experiment see BEEMS Technical Report TR420.

Two-thousand and eighty-six shingle tracers (native gravel-sized particles with embedded RFID tracking tags) were deployed in four experiments corresponding to discrete storm events representative of the primary wave directions (two northerlies and two southerlies). The transport of individual particles was related to the local wave conditions at different locations along the coast (every 100 m), determined using the Sizewell DWR and the calibrated Sizewell spectral wave model (TOMAWAC). The collective transport of traced shingle particles was related to the wave power and wave angle (relative to the shoreline), which were expressed together as the alongshore component of wave power. The first part of the analysis confirmed that shingle transport was indeed driven by waves (and no other process), and, using these findings, the second part considered net shingle transport over a multiannual period.

As expected, the transport of tracer particles was generally south-directed during storms from the north and north-directed during storms from the south. No relationship was found between the intrinsic properties of the tracer particles (e.g. size and shape) and transport. However, there was a strong relationship between the most mobile particles (i.e. the 99th percentile; considered to best represent the mobile layer of the beachface under wave action) and the magnitude and direction of the longshore component of wave power. Figure 13 shows very high correlation ($r^2 = 0.82$) between the 99th percentile of daily observed longshore displacements and longshore component of wave power.

The ten-year record of waves from the Sizewell DWR and TOMAWAC were then used to determine the alongshore component of wave power at the coast and examine the energy balance that drives the transport of beach shingle. This was an important step for the analysis, as shingle transport under individual events can be large (i.e., gross transports of up to a few hundred metres for a small percentage of particles), but overall the more-or-less balanced climatology suggests low net transport.

The wave climate measured between 1 June 2008 and 31 May 2018 was thus used to examine shingle transport in the longer term. While individual storm events might transport some shingle particles tens to a few hundred metres north and south along the shore, the modelling analysis indicates that the net transport rate over a multiannual period was relatively small, which is in agreement with numerous modelling studies conducted in the area (Halcrow, 2001; Black and Veatch, 2005; BEEMS Technical Report TR329). That is, the sum of the north- and south-directed components of wave power approximately cancel each other out. Furthermore, the wave force to the south of Minsmere Outfall is southward, and the wave force north of Thorpeness is northward. This promotes the retention of gravel within this section (including the power stations) suggesting it is a relatively closed system that does not leak much shingle. Burningham and French's (2016) and Atkinson and Esteves' (2019) calculations of sediment transport around the ness also support this conclusion, providing evidence that transport from the north and south converge at the ness and there is little transmission around it. That is, the ness marks the southern boundary of the sediment cell.

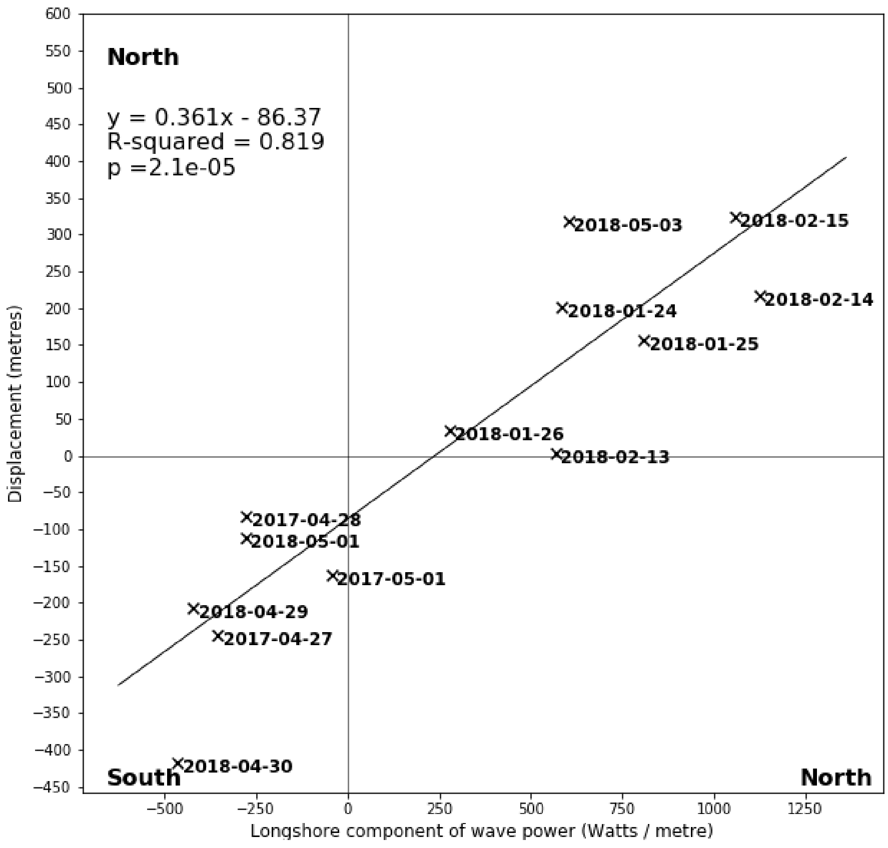

Figure 13: 99th percentile of longshore particle displacement versus the long component of wave power.

Additionally, the vector arrows diverge at Minsmere Outfall and Thorpeness, indicating the boundary of the sediment cell and a lower tendency for sediment to pass into the intervening cell (

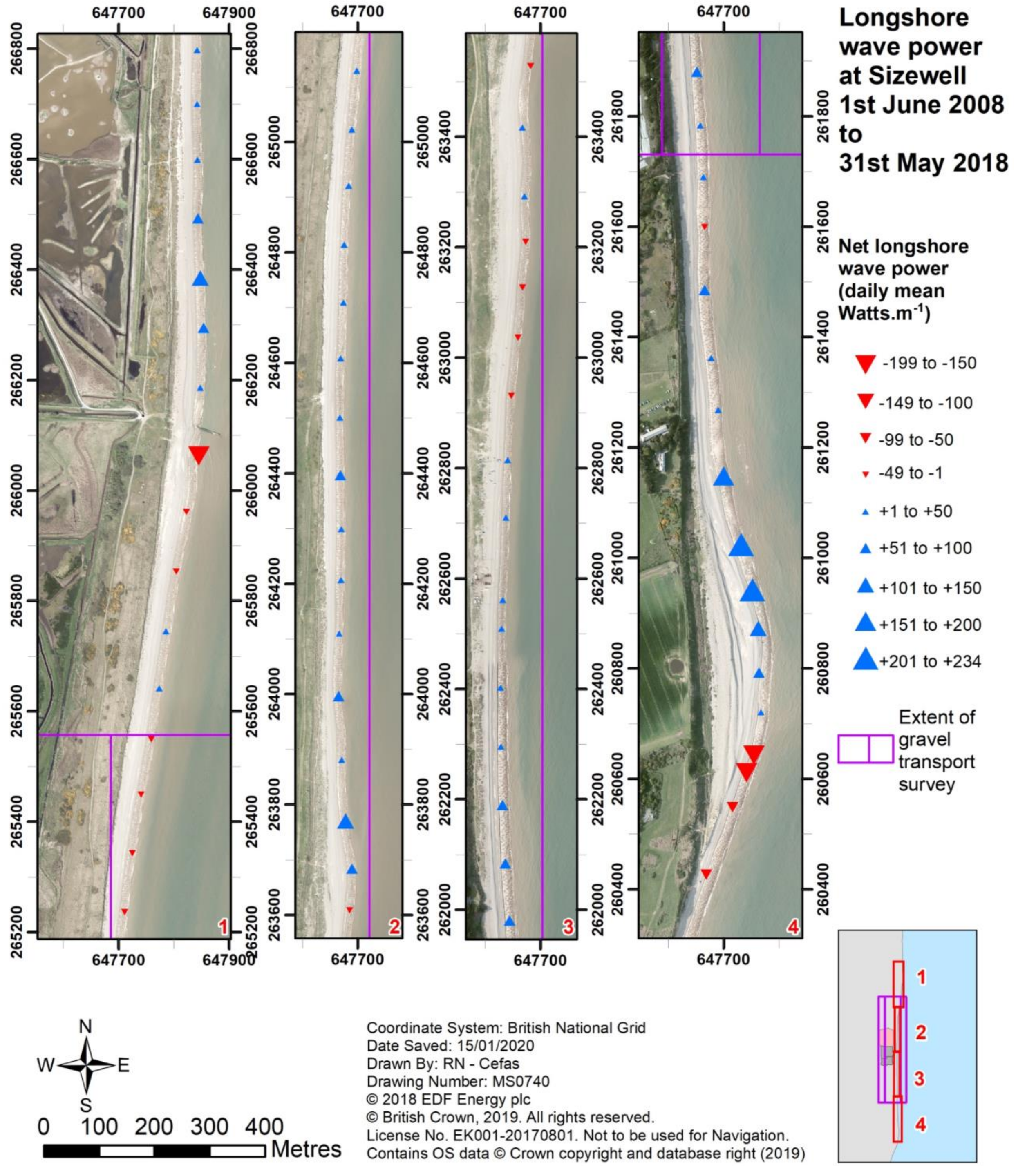

Figure 14). These measured shingle transport results, extrapolated using the wave data and model, confirm the geomorphic evidence and longshore transport model results, that shingle is, in net terms, effectively static. It is not lost to the subtidal nearshore and moves very slowly within the longshore transport system.

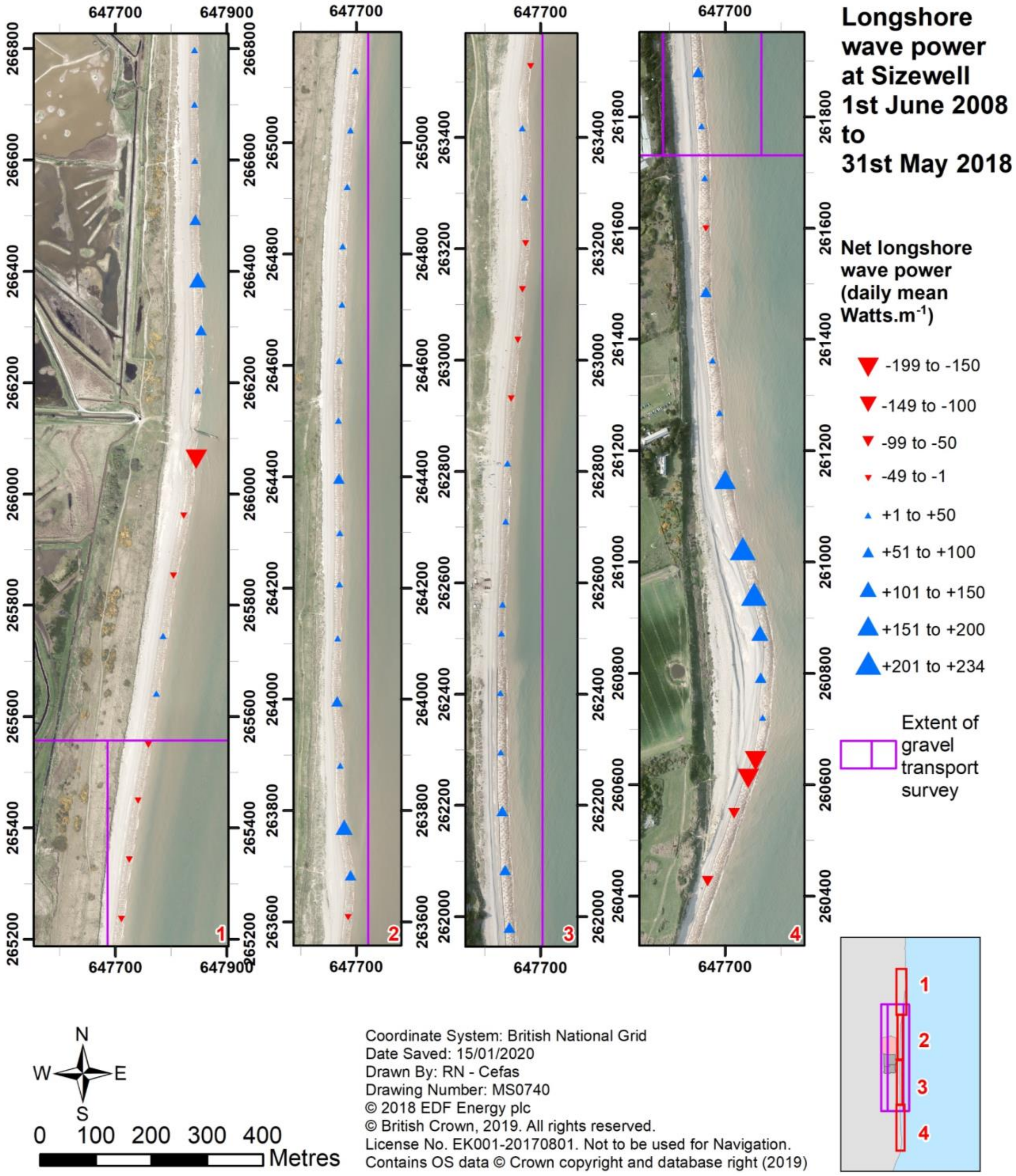

Figure 14: Residual longshore component of wave power along the Sizewell frontage.

2.3.5 Suspended sediment concentrations

Various records of suspended sediment concentrations (SSC) in the Sizewell area have been made. Calibrated optical backscatter sensors deployed on a minilander deployed approximately 0.5 km east of Sizewell C (52° 12.93' N 001° 38.28' E; BEEMS Technical Report TR098) and near the Sizewell B outfall (52° 12.73' N, 001° 37.77' E; BEEMS Technical Report TR189) were used to investigate seasonal variation in SSC. The minimum, mean and maximum SSC are shown in Table 1.

High levels of SSC from the sampling conducted in 2008 - 2009, shown as peaks in Figure 15, were driven by both storms and peak spring tidal currents. Minimum values were observed during neap tides when there is low wave energy.

Suspended Particulate matter (SPM) data, which is analogous to SSC at the surface, was also examined from MODIS satellite data (Dolphin *et al*., 2011 and Eggleton *et al*., 2011) for almost an 8-year period from 01/07/2002 to 31/05/2010. A single image/data-file is provided each day from satellite scans made at 1100 or 1300 hours (or an average of the two). Each daily image was cropped to the North Sea – English Channel Domain (48.5° to 60.5°N and from 5.75° to 16.36°E) and has a high resolution of 0.01° x 0.01° (c. 0.7 x 1.1 km). Mean and maximum values of surface SPM from the satellite data are summarised in Table 2.

Two calibrated optical backscatter sensors were also deployed seaward of the Sizewell-Dunwich Bank, near the cooling water intakes between 13 November 2018 and 13 February 2019 (BEEMS Technical Report TR498). Summary statistics of observed SSC are presented in Table 3. The particle size distribution of suspended sediments collected in booner tubes mounted on the minilander for the duration of the deployment, were comprised of approximately 40% muds and 60% sands. Estimates of the mean and peak SSC were greater seaward of Sizewell-Dunwich Bank compared with those observed landward of the bank (BEEMS Technical Report TR098 and TR189), with variations in SSC driven by tidal currents and wave events. In general, the sheltering effect of Sizewell-Dunwich Bank and slower tidal currents explains the reduced SSC there. In contrast, a relatively elevated baseline SSC is maintained seaward of Sizewell Bank as a result of stronger currents and high turbulence near the Thorpeness coralline crag ridges. This results in tidally driven resuspension and an active sand transport pathway that acts to maintain Sizewell-Dunwich Bank (BEEMS Technical Report TR357, TR498 and TR500).

Peak SSC was observed during high wave energy conditions and periods of low water slack tide. This was attributed to the settling of muddy plumes (probably during and following wave events) advected by the sensors (in a similar fashion to Green *et al*., 2000)). SSC was low during calm conditions (low/no waves) and during neap tide periods (BEEMS Technical Report TR498).

Table 1: Inshore suspended sediment concentrations (mg/l).

Report	Date	Height above bed (m)	SSC (mg/l)		
			Min	Mean	Max
BEEMS Technical Report TR098	28 November 2008 to 2 March 2009	0.3	26	103 – 161	609
	28 November 2008 to 2 March 2009	1.0	17	72 – 105	459
BEEMS Technical Report TR189 (Station 5)	April 2010 to August 2011	1.0	9	-	426
	July 2016	1.0	8.65	-	68.35
	August 2016	1.0	7.21	-	38.38
	September 2016	1.0	5.2	-	16.98

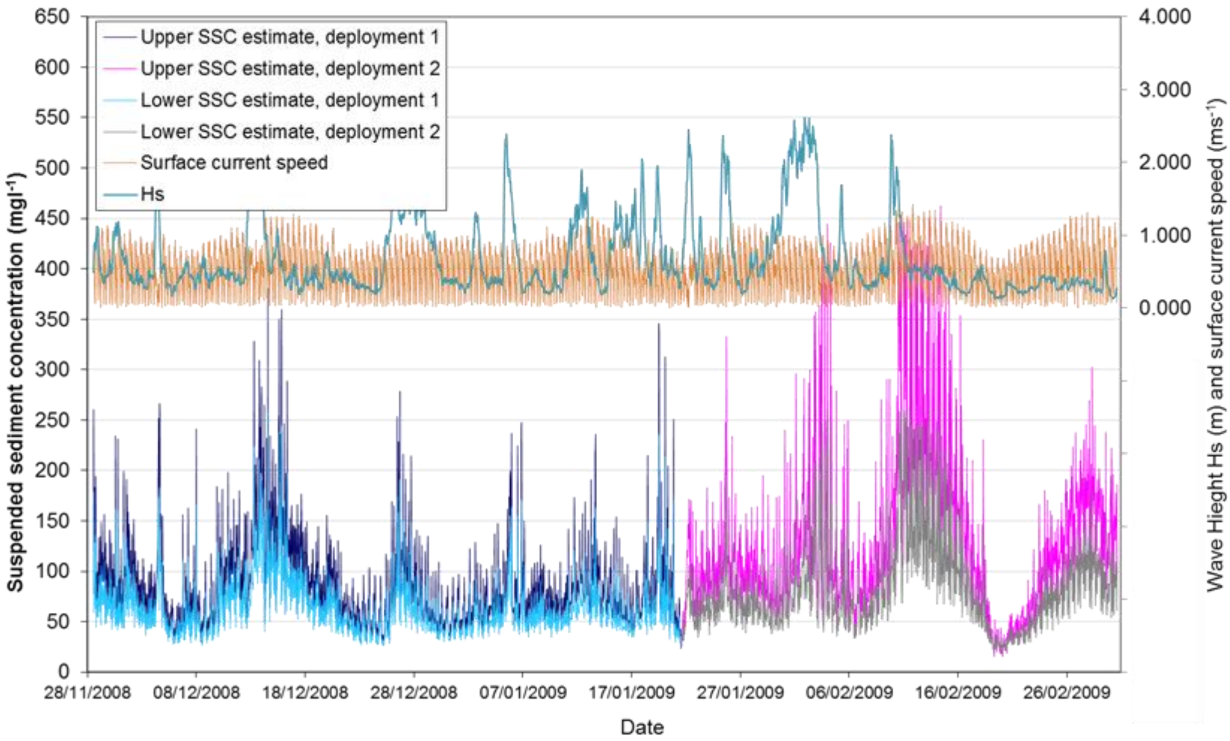

Figure 15: Background suspended sediment concentration (mg/l) at the minilander site (0.5 km off Sizewell C) measured 1 m above the bed. The plot shows two deployments (colour change) showing SSC estimated by optical backscatter sensors calibrated using sediment traps (upper) and water samples (lower). It also shows the wave height as measured at the lander and the near surface current speed (right axis) (BEEMS Technical Report TR098).

Table 2: Surface mean and maximum estimates suspended particulate matter from MODIS satellite database (Eggleton *et al*, 2011).

Period	**Mean SPM (mg/l)**	**Maximum SPM (mg/l)**
April – August	31	80
September – March	73	180

Table 3. Offshore suspended sediment concentrations (mg/l) at 1.4 m above the bed at two sensors locations labelled SZ1 and SZ2 (BEEMS Technical Report TR498).

Statistic	SZ1	SZ2
Start	13 November 2018 16:40	13 November 2018 16:00
End	13 February 2019 14:40	13 February 2019 16:30
Minimum	105.26	99.98
Maximum	2,245.75	2,131.27
Mean	452.09	513.25
Standard Deviation (+/-)	221.45	277.61

2.3.6 Shoreline and nearshore bar behaviour

The position of the Sizewell shoreline has been influenced by the local wave climate (storm frequency, direction and magnitude) at the coast as well as:

- changes in the morphology of the Sizewell – Dunwich Bank and longshore bars;
- natural and man-made erosion resistant features (e.g., Thorpeness, Minsmere sluice outfall);
- the supply of sediments along the coast from beach and cliff erosion north of the GSB;
- elevated water levels, which can occur as a result of storm surge during some NE events and high shore-normal run-up during severe easterly events (e.g., BEEMS Scientific Position Paper SPP096); and
- the unquantifiable antecedent conditions of each beach section (e.g., Masselink and van Heteren, 2014; Dolphin *et al.*, 2020), which could result in a different response to the same hydrodynamics.

The result of these factors combined on the Sizewell – Minsmere frontage is a fluctuating patchwork of erosion and accretion that creates subtle differences in persistent trends of shoreline behaviour when observed over varying temporal and spatial scales. A detailed assessment of shoreline variability and patterns of erosion and accretion the GSB has been undertaken and reported in BEEMS Technical Report TR223 Edition 3. The findings of this report were also used to inform BEEMS Technical Report TR403 and the future shoreline baseline assessment in Section 7.

2.3.6.1 Pre-1925 vs Post-1925

The Sizewell shoreline has experienced two distinct phases in the past 180 years. Almost 100 years ago, prior to 1925, long-term persistent and spatially coherent erosion and accretion occurred to the north and south of Minsmere Outfall, respectively. Following a brief reversal of this trend, the shoreline change rates since 1940 lowered and became highly variable, both temporally and spatially.

The cause of this long-term change was investigated by Pye and Blott (2006). They consider that prior to 1925, a high energy north-easterly dominant wave climate with a low Dunwich Bank (2 – 4 m lower than present day) led to rapid shoreline and cliff erosion at Dunwich. Consequently, large volumes of sediment were released from the Dunwich-Minsmere Cliffs and advected south under the prevailing wave climate, leaving the cliffs prone to ongoing erosion.

The eroded sediments subsequently accumulated to the south of Minsmere Outfall, likely due to the lower energy environment in the lee of the Sizewell – Dunwich Bank and lower wave obliquity at the coast (due to refraction around the bank and a change in shoreline orientation). Given the large shoreline and volumetric changes compared to the present day, this is likely to have been sandy material.

A significant proportion of the eroded sandy sediment is also believed to have been channelled offshore by the hard rock ridges of Coralline Crag at Thorpeness, resulting in deposition and associated enlargement of the Sizewell – Dunwich Bank. This likely transport pathway was discussed in Section 2.3.4.1.

Since 1925, the shoreline behaviour has been marked by the shift to a bimodal wave climate and a reduction in nearshore wave energy, with the latter being partially attributed to the shallower and more extensive Sizewell – Dunwich Bank morphology. The result has been high spatial and temporal variability in shoreline behaviour, characterised by significantly lower rates of net southerly longshore sediment transport and shoreline change (Pye and Blott, 2006; BEEMS Technical Report TR223 Edition 3).

2.3.6.2 Medium term (1940-present)

This period is marked by higher quality subaerial topographic and photographic data collected since 1940 (BEEMS Technical Report TR223 Edition 3). It broadly shows a pattern of spatially and temporally varying shorelines over several decades. Bands of retreat at Dunwich, south of Minsmere, and Sizewell Hall, were interspersed by sections of relative stability, or slight seaward advance, north of Dunwich, adjacent to Minsmere Outfall, at Sizewell and to the north of Thorpeness.

Shoreline analysis methods (Thieler *et al*., 2009) were used to determine change statistics every 50 m along the coast:

- Shoreline Change Envelope (SCE) (m) – the distance between the shoreline farthest from the baseline and the closest (regardless of when they occurred);
- Linear Regression Rate of Change (LRR) (m/yr) – the rate of change of the shoreline relative to the baseline, determined by linear regression using all available shoreline data (negative values indicate landward movement (erosion), positive values indicate seaward movement (accretion)); and
- Trend Strength (r^2), or the coefficient of determination, r^2 – the percentage of variance in the data that is explained by the linear regression, an estimate of how well the regression fits the data.

There were three coastal sections with high r^2 values, indicating areas of persistent shoreline erosion or accretion over the last 60-70 years (Figure 16) (BEEMS Technical Report TR223 Edition 3):

- At Dunwich and on the Northern Barrier (Minsmere Reserve), high r^2 values of 0.7 to 0.9 reflect a slow but persistent shoreline retreat rate of -0.2 to -0.6 m/yr interspersed with occasional, short phases of seaward advance;
- From 500-700 m south of the Minsmere Outfall to the proposed Sizewell C site (Southern Barrier), persistent erosion is marked by very high r^2 values of 0.8 to 1.0 and a shoreline retreat rate of -0.6 to -0.8 m/yr, increasing to -1.7 m/yr after 1992. This section also has a relatively high SCE of 52 to 64 m;
- A 1500 m section adjacent to Sizewell Hall (extending approximately 900 m north and 600 m south) had very high r^2 values of 0.8 to 1.0 and a shoreline retreat rate of up to -1.2 m/yr. This section also has a relatively high SCE of 57 to 72 m. Recent stability and very low rates of change observed along this section since 1992, are not reflected in the statistics, due to masking by high rates of retreat of up to -2.1 m/yr during the period of 1952 to 1983. Alternating phases of stability and rapid change in shoreline behaviour at Sizewell Hall, are considered an important feature of the wider Sizewell shoreline and highlight the need for temporal and spatial context when assessing shoreline changes.

Note that persistent stability produces low r^2 values.

The areas of persistent change described above are separated by sections exhibiting variable shoreline behaviour with lower r^2 values (BEEMS Technical Report TR223 Edition 3):

- A 1000-m-long section of relative shoreline stability (the Central Barrier), including Minsmere Outfall, separates the two areas of persistent erosion. Since its construction in 1830, Minsmere Outfall has been an important feature, acting as a groyne and a stabilising 'hard point' within the Sizewell coastline. It traps sediments moving alongshore and thus creates an area of net shoreline stability, but erosion has continued on either side creating two shallow sub-embayments. This section is characterised by very low r^2 values up to 0.3, low rates of change ranging between -0.2 to 0.2 m/yr and a SCE of up to 24 m. These statistics reflect the fluctuations in shoreline position regularly observed around the outfall.
- The shoreline fronting the Sizewell power stations has historically experienced very low net rates of change. This section showed a consistent but slow trend in seaward advance of 0.2 to 0.4 m/yr, variable r^2 values of 0.2 to 0.8 and a relatively large SCE of up to 43 m, indicating that this shoreline is in dynamic equilibrium. Computer modelling of waves moving over the Sizewell – Dunwich Bank indicates that this section of shoreline is currently exposed to lower wave energy, for both primary wave directions, and consequently experiences very low net longshore sediment transport (Figure 16). These factors, which are influenced by the size and shape of the bank as well as the longshore bars, are considered to provide the shoreline stability observed.
- To the south of Sizewell Hall, the bimodal wave climate combined with relatively low rates of shoreline change of up to 0.4 m/yr, indicates that the supply of sediment toward Thorpeness is very low. This is in agreement with the Shoreline Management Plan (SMP) that covers the Sizewell area and states that "there is a very weak net drift to the south past Thorpeness" (SMP7, Royal Haskoning, 2010) as well as recent modelled and measured results (see Section 2.3.4.2). It is for this reason that the southern boundary of sub-cell 3c is located at Thorpeness.

2.3.6.3 Recent period (1992-present)

The 1992 – present data is considered separately because it represents a reasonably long period (almost 30 years) of high-quality shoreline position data derived from topographic, orthorectified aerial imagery and LiDAR survey datasets. These data were collected by the Environment Agency and Sizewell Shoreline Management Steering Group (comprised of Magnox and EDF Energy) and allow quantification of the seasonal, inter- and intra-annual variability that contributes to the patterns of erosion and accretion observed over greater temporal and spatial scales (BEEMS Technical Report TR223 Edition 3).

The shoreline analysis derived from aerial imagery shows a coastline that exhibits a high degree of spatial variability, as is common for beaches in the lee of sand banks (Environment Agency, 2008; Environment Agency, 2011; Dolphin *et al*., 2012), with zones of common shoreline response typically constrained to less than several hundred metres (Figure 17). That spatial variability is also likely to result from complexities in the behaviour of mixed gravel and sand beaches, including a reliance on variable antecedent conditions and volumetric changes due to winnowing and return of sandy materials – beaches of this type are poorly understood (Van Wellen *et al*., 2000; Masselink and Van Heteren, 2014 and Payo *et al*., 2019).

The Sizewell shoreline can be broadly divided into nine zones of common behaviour (see the time-series of shoreline positions in Figure 18):

- **Dunwich Cliffs.** Low rates of change, advancing in the north and south (0.0 to 0.7 m/yr, r^2 = 0.5 to 0.8) and retreating in the centre (approximately, -0.1 to -0.6 m/yr, r^2 = 0.4 to 0.8), with a SCE of up to 18 m;
- **Northern Barrier** (Minsmere north). Highest rates of retreat in the GSB, with a generally persistent trend (-0.6 to -2.2 m/yr, r^2 = 0.6 to 0.9) and a relatively large SCE of up to a 57 m. The shingle barrier backing the beach has a relatively low volume (270–800m^3 per metre of beach length (m^3/m)) and is occasionally overtopped;
- **Central Barrier** (around Minsmere Outfall). Highly variable shoreline position, with near-zero or slowly advancing net shoreline change (0.2 to 1.1 m/yr, r^2 = 0.1 to 0.9) and an SCE of up to 31.5 m. The Central Barrier extends from the outfall for around 1500m to the south, has a large volume (800–1650m^3/m), is not over topped and erodes by scarping;
- **Southern Barrier** (Minsmere south). High rates of retreat, with a generally persistent trend (-0.5 to -1.4 m/yr, r^2 = 0.6 to 0.9) and an SCE of up to 33 m. This section has intermediate volumes (940–1200m^3/m) and also presently erodes by scarping and not overtopping (although thin deposits at two locations indicate that waves have splashed onto the top of the barrier);

- **Sizewell C frontage**. Low rates of shoreline change, retreating to the north and stable and/or advancing to the south (-0.4 to 0.5 m/yr, r^2 = 0.0 to 0.2) and a SCE of up to 25.5 m;
- **Sizewell B frontage**. Variable shoreline position, with high rates of net shoreline advance, due largely to post-construction foreshore recovery and development of a shoreline protrusion (called a salient) in the lee of the Sizewell B outfall (-0.1 to 1.6 m/yr, r^2 = 0.2 to 0.8). The salient has been more or less volumetrically stable since 2011, indicating it is no longer accumulating longshore sediments. It has a relatively large SCE of up to 58 m;
- **Sizewell Gap and south**. Variable shoreline position, with low rates of shoreline change (-0.4 to 0.3 m/yr, r^2 = 0.0 to 0.6) and a relatively small SCE of up to 10 m;
- **Thorpeness north**. Variable shoreline position, generally stable or slowly retreating, with isolated areas of greater rates of retreat (-0.8 to 0.2 m/yr, r^2 = 0.0 to 0.9) and a SCE of up to 17.1 m; and
- **Thorpeness**. Highly variable shoreline position, with high rates of change near the tip of the ness (-1.4 to 0.7 m/yr, r^2 = 0.1 to 0.7) and a SCE of up to 52 m.

The high rates of net shoreline advance observed adjacent to the Sizewell B frontage were largely due to post-construction foreshore recovery. The shoreline was indented during 1989 to 1992 as a result of construction of a coffer dam recessed into the beach face and dredging (cut and fill) for the installation of intake and outfall tunnels; and dredging and construction for the Sizewell B's Beach Landing Facility. The coffer dam and dredging most likely created the initial indentation (Figure 25), with the BLF, sited north of the indentation, acting like a groyne, interrupting sediment supply and exacerbating or maintaining the indentation by down-drift erosion. By 1997, following removal of the coffer dam (summer 1992) and the BLF (summer 1993), the shoreline had recovered.

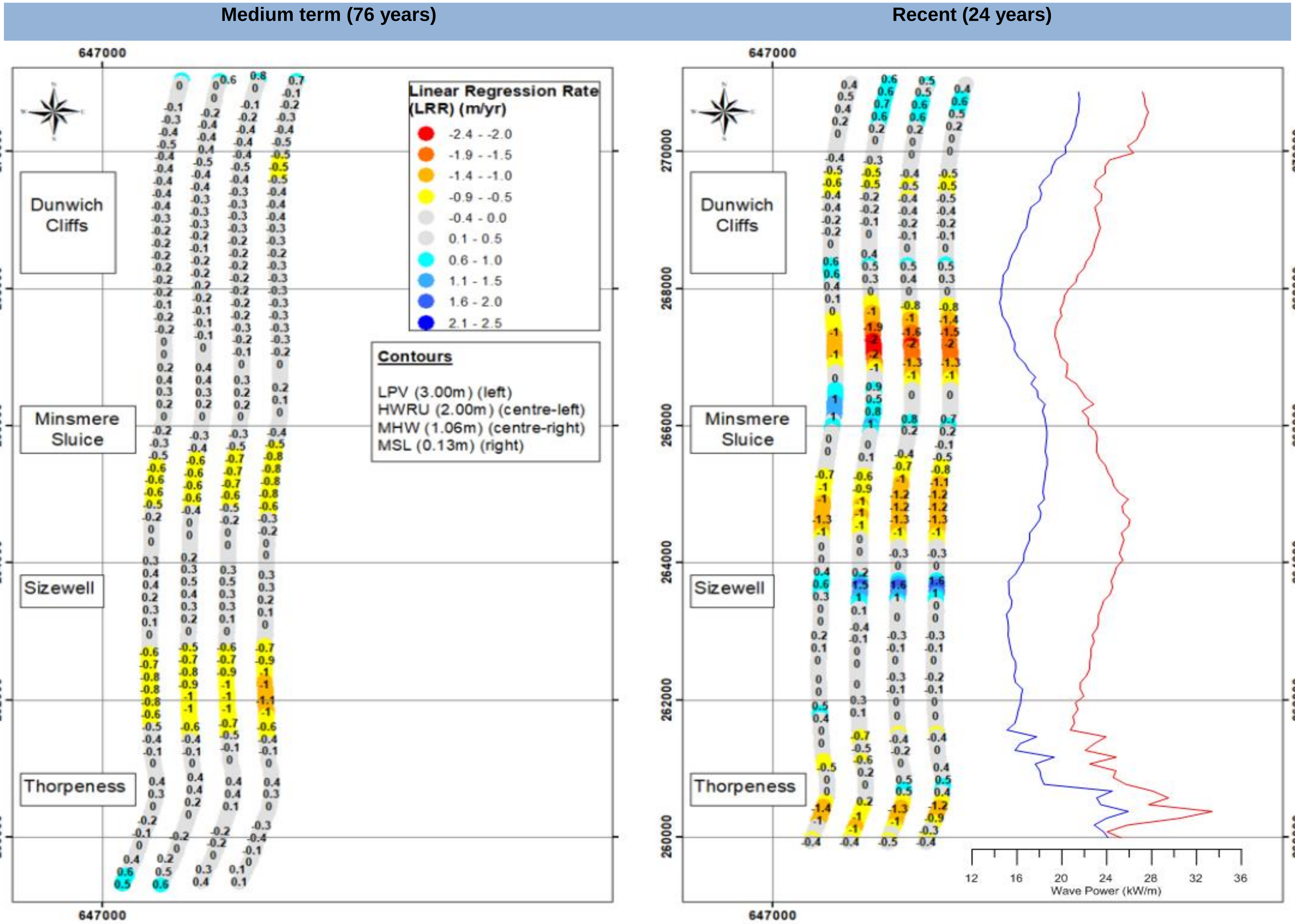

Figure 16: Comparison of alongshore patterns in nearshore wave power for a south of south-easterly (case F; blue line) and north easterly storm (case H; red line), with the medium term (76 years (1940 - 2016); left panel) and short term (24 years (1992 - 2016); right panel) shoreline change rates derived from aerial photos. LPV = Limit of Permanent Vegetation; HWRU = High Wave Run Up; MHW = Mean High Water; MSL = Mean Sea Level. Easting scale true for the LPV (3.00 m ODN) contour, with all other contours manually offset to facilitate comparison. The nearshore wave results were extracted outside of the breaker zone on the -7 m (ODN) contour (BEEMS Technical Report TR223 Edition 3).

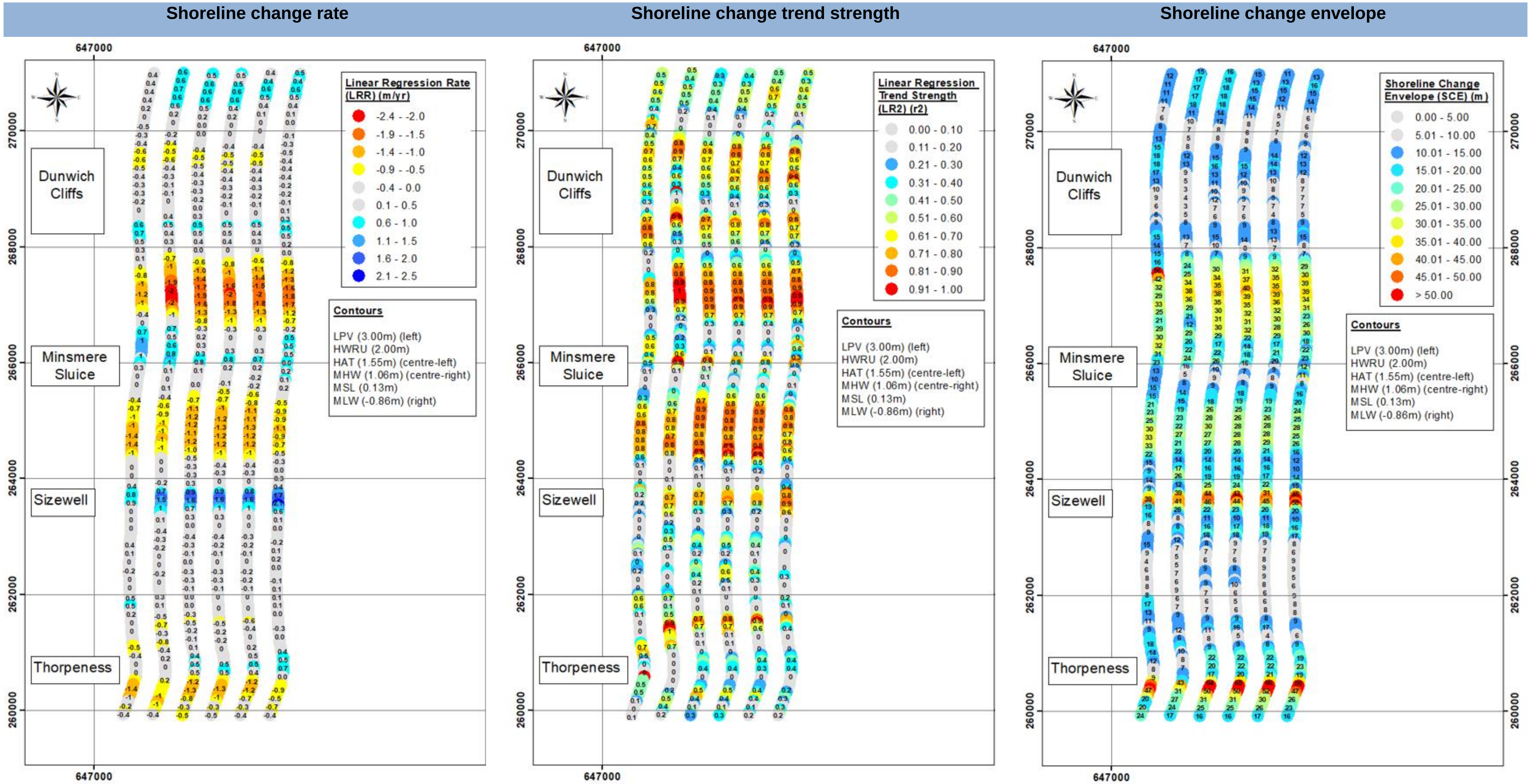

Figure 17: Longshore variability for six shoreline contour elevations (1992 – 2016). LPV = Limit of Permanent Vegetation; HWRU = High Wave Run Up; HAT = Highest Astronomical Tide; MHW = Mean High Water; MSL = Mean Sea Level; and MLW = Mean Low Water. Easting scale true for the LPV (3.00 m ODN) contour, with all other contours manually offset to facilitate comparison (BEEMS Technical Report TR223 Edition 3).

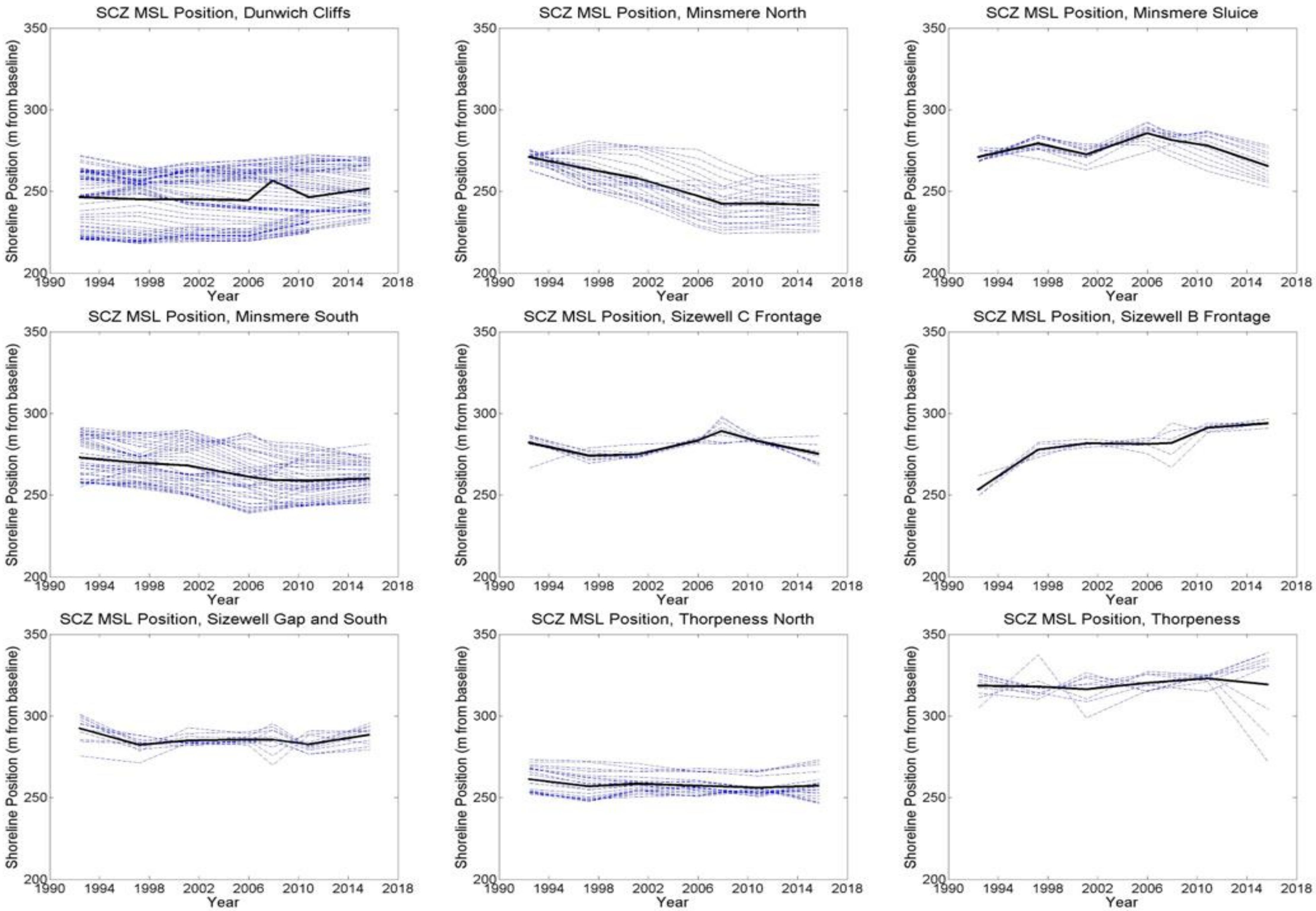

Figure 18: Shoreline position time-series for the nine zones identified and described above, where a relatively consistent shoreline response is observed based on shoreline statistics calculated from aerial imagery during the period of 1992 to 2016 (BEEMS Technical Report TR223 Edition 3).

Following the shoreline recovery at Sizewell B and a period of stability, a salient developed in 2005, which continued to grow until 2011, varying in position (northing) over time. It probably formed as a result of the reshaping of the outer longshore bar once it migrated into the area occupied by the 51.5 m^3/s Sizewell B outfall discharge during 1997 and 2003. As the bar cannot co-exist with the operational outfall (due to turbulence and scour from the structure and its discharge jet), it has curved seaward of the outfall, which may have altered wave refraction patterns inshore and led to development of the salient. A similar feature was observed adjacent to Sizewell A's outfall, which disappeared once the discharge was reduced when the station began decommissioning after 2006.

Several elevation contour lines were examined in the shoreline analysis and these show broadly similar patterns (Figure 17). The 3.00 m ODN contour differs most significantly from other contours as it is only affected during storm surges and severe erosive events. In contrast, the 0.13 m ODN contour (MSL) is inundated on every tide and therefore gains and loses sediment over very short time periods via sand exchange with the subtidal zone, including the longshore bars, and subaerial movements of shingle. Whilst these very short-term fluctuations in shoreline position have no direct bearing on the long-term patterns of shoreline change, where they are large they can introduce uncertainty into the measurement campaigns, which historically only sample at long intervals – this reiterates the need to assess trends in shoreline change over small temporal and spatial scales to understand natural variability (and to measure impacts).

Seasonal changes in beach elevation and sediment volume are observed: a reduction in volume during winter months and stability or growth during summer. As shingle does not exchange with the subtidal, this pattern is interpreted as the winter winnowing and summer return of sand. Although common, not all years display this pattern. Large storm events, especially easterly storms with high run-up due to their shore-normal approach, can have significant but typically spatially varying effects, reflecting the fact that sand is usually moved along as well as across the shore, during and following storm events (BEEMS Technical Report TR223 Edition 3 and Scientific Position Paper SPP096). Antecedent hydrodynamic and sediment conditions are also likely to contribute to variability (Masselink and van Heteren, 2014).

Whilst areas of persistent erosion exist on the Northern and Southern Barrier sections, the short-term patterns in shoreline position and beach volume suggest that sediment transport and the wider Sizewell shoreline are in a form of dynamic equilibrium along much of the coastline. Whilst erosion resistant coastal features and sediment supply are local and regional contributory factors, this equilibrium is considered to be primarily associated with fluctuations in wave climate and longshore drift conditions, which due to the closely balanced bimodal wave climate and interactions with the Sizewell – Dunwich Bank, results in short-term drivers of shoreline change that do not persist long enough to affect net geomorphic change. Additionally, the dominant beach shingle material is likely to have a stabilising influence because it is not exchanged with the subtidal and does not leave the shingle sediment cell in any substantive way (BEEMS Technical Report TR420).

In addition to subaerial variability, subtidal changes in the morphology of the nearshore longshore bars have been assessed using Environment Agency, BEEMS and SSMSG bathymetric surveys every 5 years (approximately) (Figure 19 and Figure 20). The nearshore bars are approximately shore-parallel, except for a 1000 m section centred on Minsmere Outfall, where the bars are deflected into a north-northeast to south-southwest orientation, and adjacent to Sizewell B, where the outer bar is deflected seaward of the Sizewell B outfall (see Figure 19).

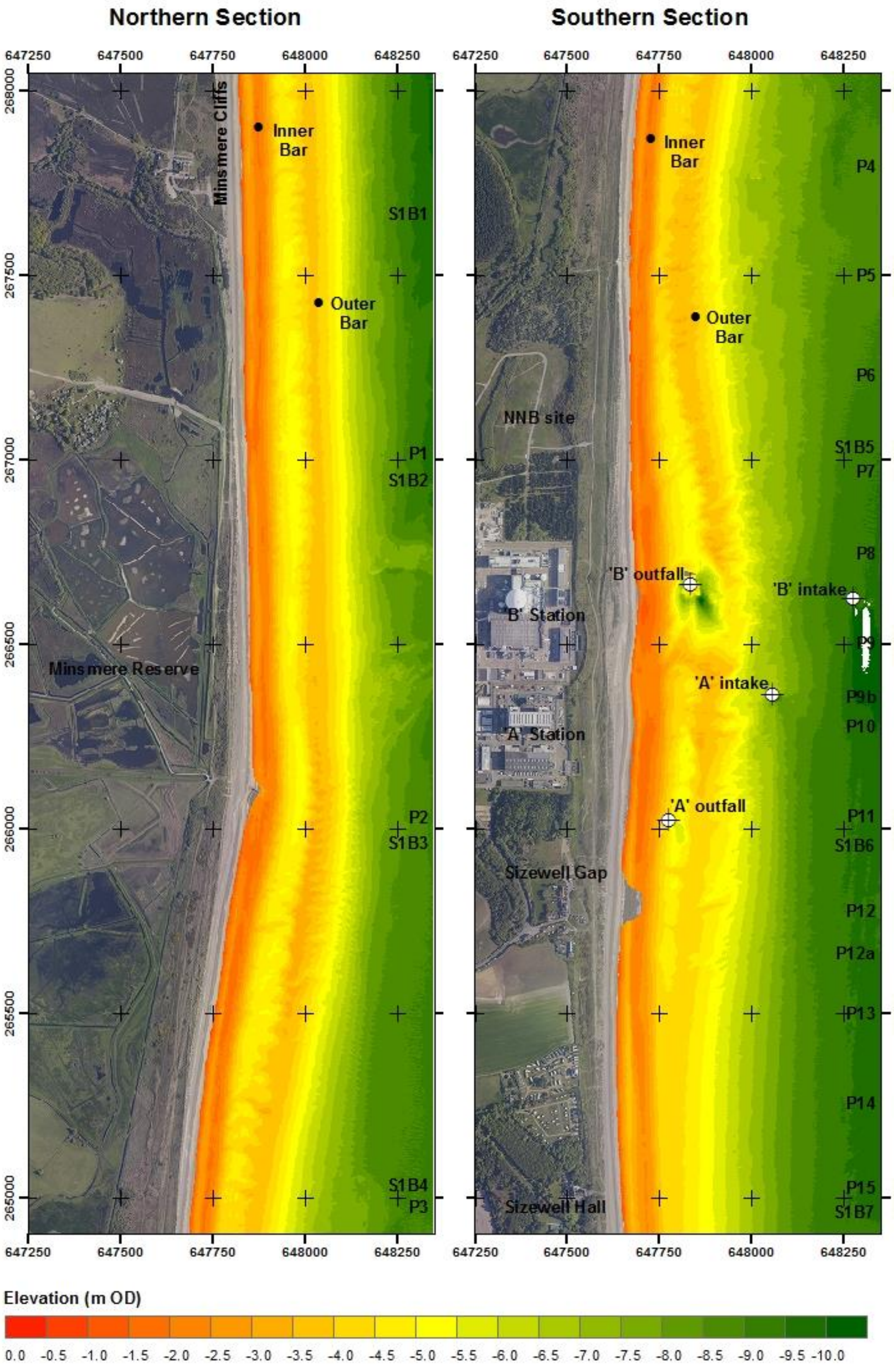

Figure 19: Comparison of subtidal morphological features between Dunwich – Minsmere Cliffs and Sizewell Hall, based on swath bathymetry (2017) (aerial imagery, 2012) (BEEMS Technical Report TR223 Edition 3).

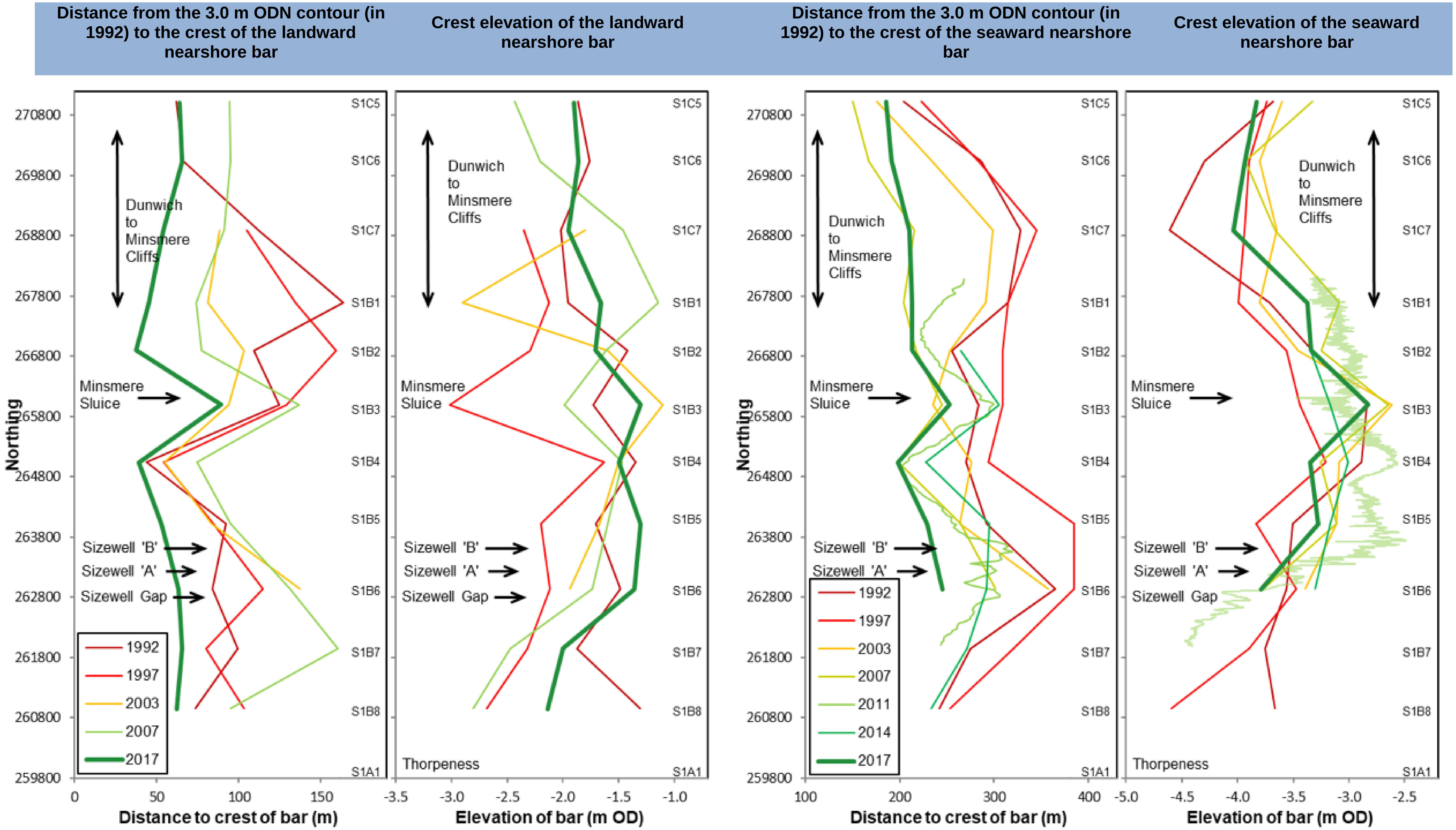

Figure 20: Size and position of the nearshore bars in Sizewell Bay, based on EA topographic and bathymetric surveys (1992, 1997, 2003, 2007, 2014 and 2017) and BEEMS multi-beam survey (2011), between Dunwich and Thorpeness (BEEMS Technical Report TR223 Edition 3).

The crest of the inner bar is typically located 50 to 150 m offshore of the 3 m ODN contour and has a crest elevation ranging from -1.0 to -3.0 m ODN (Figure 19 and Figure 20). Along the Dunwich-Minsmere Cliff frontage, the position and elevation of the inner bar shows significant variation and is sometimes absent. To the south of Minsmere Cliffs, it showed slight landward migration, with maximal landward positions located within the areas of persistent erosion either side of Minsmere Outfall and the maximal seaward position located close to Minsmere Outfall. Adjacent to the Sizewell power stations, the position of the inner bar is variable, with low net landward migration. Whilst it has maintained a relatively stable elevation of approximately -1.5 m ODN, gradually decreasing to the south, a net elevation gain of approximately 1.5 m has occurred in proximity to Minsmere Outfall.

The outer bar is larger than the inner bar in both height and width (Figure 19 and Figure 20), and is typically located 150 to 400 m offshore of the 3 m ODN contour with a crest elevation ranging from -2.5 to -4.5 m ODN. Movements mirror the inner bar – i.e., landward migration adjacent to eroding shorelines. The maximal seaward positions were located in proximity to Minsmere Outfall and adjacent to the Sizewell power stations (in the vicinity of the Sizewell B cooling water outfall). The crest elevations of the outer bar have remained relatively stable, with a maximal elevation of approximately -2.5 m ODN occurring in proximity to Minsmere Outfall, before reducing in elevation to the south of Sizewell B and moving landward to merge with the landward nearshore bar adjacent to Sizewell Hall. The merging may be linked to the presence of Thorpeness and the Coralline Crag, and/or to declining wave energy levels toward the south of the GSB.

At Thorpeness, the nearshore morphology becomes a combination of sedimentary bedforms and static erosion-resistant outcrops of Coralline Crag. A single, low-lying offshore bar with variable height and width was present in 1992 but has since become a less pronounced feature. Recent observations (Atkinson and Esteves, 2019) suggest that the bar is a fluctuating feature whose dimensions and extension affects the propagation of nearshore waves and the local-scale pattern of shoreline change immediately south of the ness.

2.4 Climate change

This section describes aspects of climate change relevant to coastal geomorphology EIAs for background information. Note that shoreline change (in part driven by climate change) that could result in new impacts several decades into the future is covered in Section 7.

The main factors influenced by climate change that could affect the geomorphology or hydrodynamics of the GSB are:

- increased relative sea level, which is likely to increase overtopping, beach/cliff erosion[10], breaching, and may increase rates of longshore transport;
- an altered sediment (in particular sand) supply regime into, or out of, the GSB, which could be caused by climate change or changes in regional coastal management practices (defined through shoreline management plans).

Any effects on coastal geomorphology or hydrodynamics would be expected to take decades to develop as changes in the above factors would be progressive and geomorphic response may lag those changes.

[10] Note the negative feedback loop that additional sediments from increased cliff erosion could reduce the likelihood or frequency of breaching.

UKCP18 projections were released late 2018 and new results have been taken into consideration for this report and for any analysis presented (see also BEEMS Scientific Advisory Report SAR036 and Environment Agency 2019).

2.4.1 Future sea level

Estimates of future mean sea level at Sizewell have to incorporate both global changes in mean sea level and local factors – collectively these are sea level changes relative to the land (or relative sea level). Global changes in sea level are primarily controlled by three factors:

- thermal expansion of the ocean;
- melting of glaciers; and
- changes in the volume of the ice caps of Antarctica and Greenland.

Local changes can take the form of either isostatic effects (changes in land elevations due to the redistribution of weight on the land surface, e.g. glacial rebound), tectonic effects (changes in land elevations due to tectonic adjustments) or from aquifer dewatering.

Furthermore, water levels can be affected by changes in storm surge magnitude or changed tidal amplitude. The overarching National Policy Statement for Energy (EN-1, sec 4.8.6) states:

> *The IPC[11] should be satisfied that applicants for new energy infrastructure have taken into account the potential impacts of climate change using the latest UK Climate Projections available at the time the ES was prepared, to ensure they have identified appropriate mitigation or adaptation measures. This should cover the estimated lifetime of the new infrastructure. Should a new set of UK Climate Projections become available after the preparation of the ES, the IPC should consider whether they need to request further information from the applicant.*

Coastal flood risk due to sea level rise is expected to increase over the 21st century and beyond under all emission scenarios due to a rise in both the frequency and magnitude of extreme water levels. This increased future flood risk is expected to be driven mainly by the effects of sea level rise, rather than by changes in atmospheric storminess. Storm surge modelling suggests no significant additional increase in the statistics of extreme water levels due to atmospheric storminess change. The largest trend in surge simulations from this additional component equates to a change of about 10 cm per century for the 1-year return level – much less than the time-averaged sea level change under the same emission scenario.

Projections of sea level rise (SLR) have been generated in UKCP09 and UKCP18. However, where UKCP09 projections were based on socio-economic emissions scenarios, UKCP18 projections are based on atmospheric CO_2 concentrations (RCP 2.5, RCP4.5 and RCP8.5), so it is difficult to infer direct equivalence across the reports.

Regardless of the projections adopted, there is currently no means to infer geomorphic change in the GSB directly from exact estimates of SLR i.e., the impact on geomorphology of the variations between each projection cannot be quantified. This is explored in detail in BEEMS Technical Report TR403 (summarised in Section 7 herein), where the likelihood of future shoreline change affecting the Sizewell C development was assessed based on SLR in 2070 of 0.54m (the 95th percentile under the UKCP18 mid-range scenario).

SLR projection scenarios with approximate equivalence to this over a 50-year window (i.e., to 2070) are highlighted on Figure 21. Royal Haskoning (2015) recommended the use of the 95th percentile of the median UKCP09 projection for flood risk assessment – over a 50-year window (to 2070), this is roughly equivalent to

[11] The IPC (infrastructure planning committee) has been replaced by National Planning Inspectorate.

the RCP4.5 95th percentile, or the 70th percentile of the RCP8.5 (high) projection. For comparison, on the RCP8.5 95th percentile projection, SLR of 0.54 m is reached around 8 years earlier, in c. 2062.

Future geomorphic change projections (Section 7) have been based on the presumption that the projections of UKCP18 are relatively similar to 2070 and can be reasonably inferred from present trends. Beyond 2070, the variation in SLR between mid- and high-range (probable worst case) RCP scenarios increases considerably, suggesting an increasing uncertainty in future environmental change prediction. Further, Lowe et al. (2018) assert that "the high end scenarios of UKCP09 (referred to as the H++ scenario), which allowed for a low probability future with sea level rise up to around 2m by 2100, can still be considered a useful plausible but unlikely high-end sea level pathway for decision-making". No amendment to the H++ high-end scenario was proposed in UKCP18.

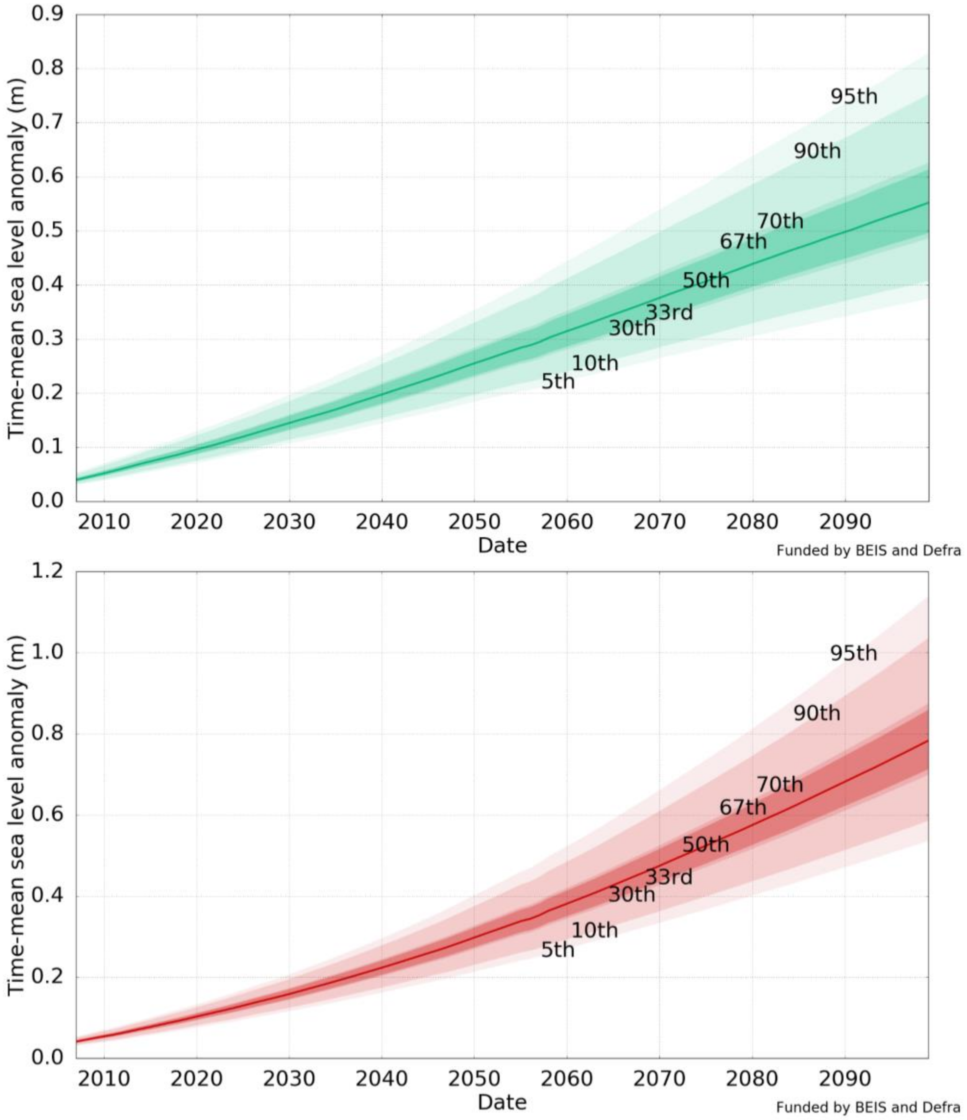

Figure 21: UKCP18 time-mean sea level anomaly (m) for years 2007 up to and including 2099, for grid square 52.17°, 1.75° (closest grid to Sizewell) using baseline 1981-2000, and scenarios RCP4.5 (top) and RCP8.5 (bottom), showing the 5th, 10th, 30th, 33rd, 50th, 67th, 70th, 90th and 95th percentiles.

2.4.2 Future wave climatology

Projections from an ensemble of seven CMIP5 (Coupled Model Inter-Comparison Project Phase 5) global wave simulations suggest an overall reduction in mean and annual maximum significant wave heights (SWHs) of up to 10-20% around the UK by the end of the 21st century under RCP8.5 (BEEMS Scientific Advisory Report SAR036, Environment Agency (2019)). The southern North Sea and Lowestoft (the closest point of UKCP18 data to Sizewell) show a reduction in the mean annual maximum significant wave height of around 5% under RCP8.5 (Figure 22), however there is some uncertainty for semi-enclosed seas due to local weather in important (Bricheno and Wolf, 2018), and therefore large inter-decadal variability is possible. Although there are predictions about future changes of the North Atlantic Oscillation (NAO), Burningham and French (2013) show that there is no correlation in the North Sea between the NAO and SE winds (which are important for Sizewell) and there is only a very weak correlation for NE winds. Lowe *et al*.'s (2018) regional analysis (Figure 23) gives small reductions in mean SWH at Sizewell (RCP4.5 = -1.7% and RCP8.5 = -3.3%) but larger reductions in the annual maximum SWHs, which are more representative of the storm wave climate relevant to coastal geomorphology (RCP4.5 = -2.6% and RCP8.5 = -12.3%).

2.4.3 Future regional sediment supply

A significant rise or fall in the supply of sediment to GSB could affect patterns and rates of shoreline change, and potentially the form and volume of the Sizewell – Dunwich Bank. There are four broad possibilities for future sediment supply, though the latter two are considered very unlikely:

- *Natural increase in sediment supply.* A natural increase could occur if the beaches fronting Minsmere-Dunwich cliffs were eroded and/or as a result of expected increased sediment volume supplied from the eroding Easton/Covehithe/Benacre cliffs (Brooks and Spencer, 2012). Note that present retreat rates of retreat only need to be maintained in order to gain an increase in supply from these cliffs because available cliff area increases as the shoreline retreats. That is, as shorelines erode, the cliffed frontage increases in length and height. The main driver is sea level rise.
- *Unnatural increase in sediment supply.* This could occur if man-made structures were removed or fell into serious disrepair. The candidates are the Minsmere sluice outfall (unlikely in the next 50 years according to the Environment Agency) and the Blyth river mouth jetties (no active plans or updates in the SMP)[12]. The removal of these partial blockages to sediment transport would increase sediment supply.
- *Natural reduction in sediment supply.* This is very unlikely and could only occur if there was a significant increase in SSE storms or a dominant SSE unidirectional wave climate, slowing supply from the Easton/Covehithe/Benacre cliffs. There is no forecast or historical case for this; it is especially influenced by the geometry of the North Sea with largest fetch and therefore waves being from the N – NE.
- *Unnatural decrease in sediment supply.* The very unlikely introduction of a coastal protection scheme for the Easton/Covehithe/Benacre cliffs would significantly impact supply to the Suffolk coasts further south, including Sizewell. This is considered very unlikely because the opposite – no active intervention – is specified in the SMP and because at present, and most likely in the future, cost benefit analysis would not justify coastal protection. If such protective measures were undertaken, there would be a considerable time lag before impact at Sizewell due to the distance and the likely storage function of the Blyth ebb tidal delta.

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[12] SMP2010 has a Hold the Line (HTL) status for the Blyth river mouth section. In the absence of any formal announcements, this indicates that the jetties are expected to be retained in one form or another to hold the line.

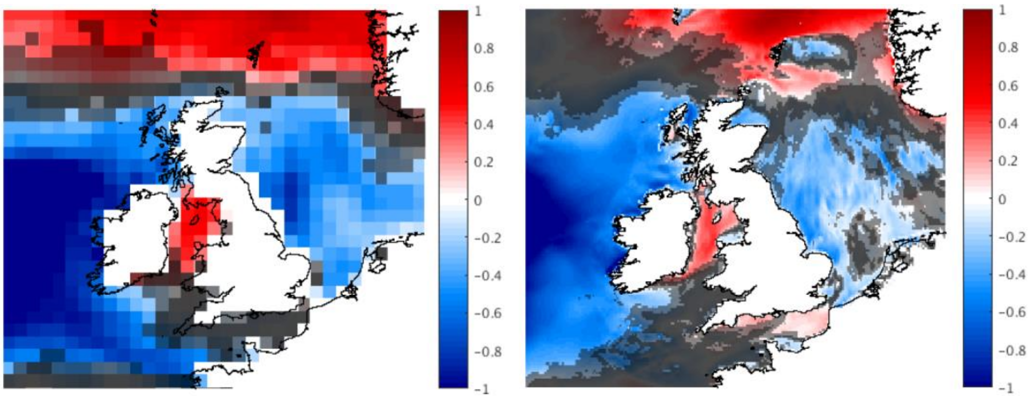

Figure 22: 21st century end change in mean annual maximum significant wave height (m) under RCP8.5. Global model (left) and regional (right). Grey masking indicates where natural variability is high. Where there is no masking, there is higher than a 75% chance that the future wave conditions are different to the historical conditions, rather than masked by natural variability. Source: Palmer *et al*. (2018).

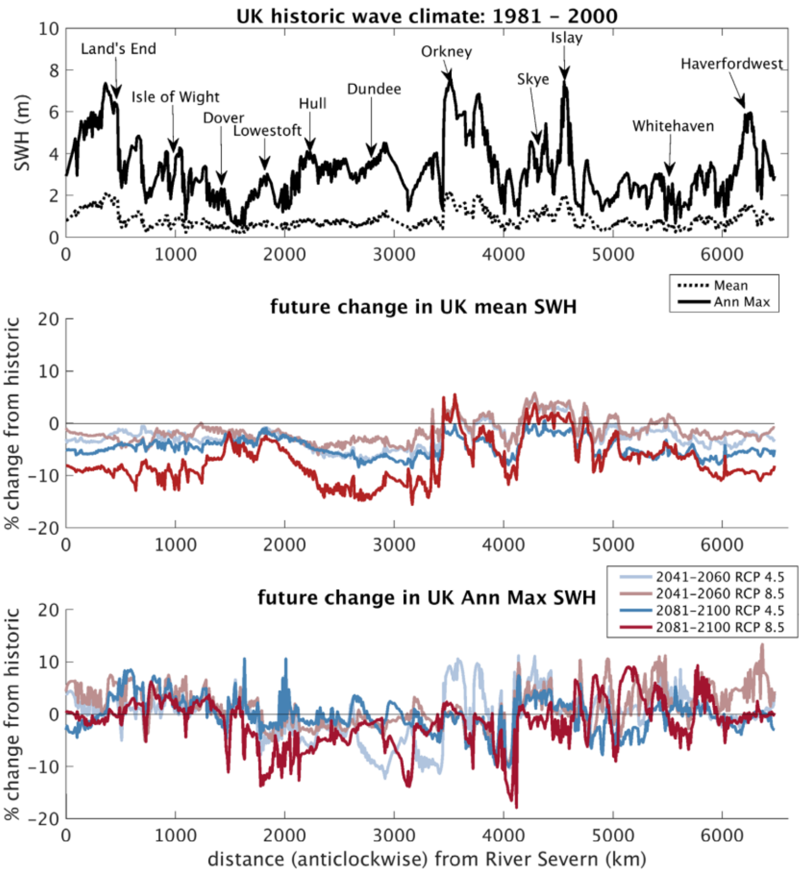

Figure 23: Historical wave climate and projected future changes for UK coastal sites. The top panel shows the mean Significant Wave Height (SWH, dotted line) and Annual Maximum SWH (AnnMax, solid line) from the historical simulation. The middle and lower panels show percentage changes in mean SWH and AnnMax respectively, relative to a 1981-2000 baseline period. The four coloured lines represent "mid-21st century" (2041-2060) and "end-21st century" (2081-2100) change signals under RCP4.5 and RCP8.5. Source: Lowe *et al*. (2018).

2.4.3.1 *Natural increase in sediment supply to the Greater Sizewell Bay*

A natural increase in supply is likely because the Easton – Benacre Cliffs are likely to remain unprotected (see Section 2.4.3.4) and as the cliff-line retreats in this area, the volume of sediment released per unit retreat will rise due to increases in cliff height and available cliff length (Brooks and Spencer, 2012). Cliff exposure will rise with rising sea levels. The likely consequence is a rise in, or maintenance of, sediment supply. Additional sediment will slow rates of shoreline retreat and potentially increase accretion rates where it occurs, and over a long period of time it could counter shoreline retreat (i.e., reduce erosion rates) and result in slow growth of the Sizewell – Dunwich Bank. A growing bank that keeps pace with sea level rise will deliver similar patterns of inshore waves and shoreline change to those presently experienced. Importantly, the bank, as well as the longshore bars, would continue to dissipate wave energy and refract waves toward a more shore-normal orientation, maintaining the low rates of longshore transport and shoreline change in the Sizewell area.

2.4.3.2 *Increases in sediment supply as a result of removal/loss of man-made hard points and coastal protection structures*

Removal or loss of key structures north of Sizewell could result in additional sediment supply into the nearshore system due to subsequent erosion of the sediment volumes they hold. The key structures are the Minsmere Sluice (considered to have a lifetime of at least 50 years), the Blyth river mouth jetties and the coastal protection around Southwold. Although included here as a method by which supply might increase, loss of positional stability at the mouth of the Blyth would be complex and rates of supply would be difficult to forecast; this would also require a change in the SMP with both jetties having a *hold the line* policy (see Table 4).

The effects of increases in supply are the same as those described in Section 2.4.3.1, although the volumes are likely to be significantly lower.

2.4.3.3 *Natural reduction in sediment supply to the Greater Sizewell Bay*

This is unlikely because the primary sources, the cliffs in the Easton – Benacre coastal section, will increase in length and height as the cliff-line retreats with sea level rise. Therefore, a natural decrease in sediment supply to the Greater Sizewell Bay is unlikely. The primary means by which this could occur would be a significant reduction in wave energy acting upon the shoreline.

2.4.3.4 *Decreases in sediment supply due to large-scale coastal protection resulting from a fundamental shift in SMPs*

The SMP that includes Sizewell (SMP7) divides the coast into Policy Development Zones (PDZs) of which PDZ4 and the southern part of PDZ3 cover the GSB. PDZ2 and 3 cover the area to the north of the GSB that is typified by high rates of cliff erosion and release of sediments into the nearshore environment.

Significant changes to SMPs leading to a decrease in sediment supply from the Easton – Benacre Cliffs would require significant changes to the present SMP and coastal/sediment management principles (i.e., allowing sediment to move naturally within the coastal system where possible). A reduction in supply due to a change in policy (re-classification from *no active intervention* or *managed realignment* (Table 4) to *hold the line*) would imply the construction of extensive hard engineering (sea walls, groynes etc.) to prevent cliff erosion in the BEN 6.1 – SWD 8.1 section. Such an action is considered to be highly unlikely as it would require a reversal in UK coastal management policy and a large and ongoing financial investment (e.g., Sea Palling coast in Norfolk cost £65M to protect and maintain a 4 km section; Dolphin *et al*., 2012); it would also starve down-drift beaches and initiate or exacerbate down-drift coastal erosion.

Table 4: Summary of specific shoreline management policies for Sizewell Bay in SMP7. The key coastal sections that do or could provide sediment via cliff erosion are in red. (Source: The Suffolk SMP, 2010, with revisions in MIN13.3 (April 2015))

Policy Unit		**Policy plan (epoch)**		
		2025 (1)	**2055 (2)**	**2105 (3)**
BEN 6.1	Kessingland South	HTL	MR	MR
BEN 6.2	Kessingland Levels	HTL	MR	MR
BEN 6.3	**Benacre Farm**	**MR**	**MR**	**MR**
COV 7.1	**Benacre Broad to Easton Broad**	**NAI**	**NAI**	**NAI**
COV 7.2	**Easton Broad**	**NAI**	**NAI**	**NAI**
SWD 8.1	**Easton Bavents**	**MR**	**MR**	**MR**
SWD 8.2	Easton Marsh	HTL	MR	HTL
SWD 8.3	Southwold Town	HTL	HTL	HTL
BLY 9.1	The Denes	HTL	HTL	HTL
BLY 9.2	Harbour Entrance (north and south)	HTL	HTL	HTL
DUN 11.1	Walberswick	HTL	HTL	HTL
DUN 11.2	Walberswick Marshes	MR	MR	MR
MIN 12.1	**Dunwich & Minsmere Cliffs**	**NAI**	**NAI**	**NAI**
MIN 12.2	Minsmere North	MR	MR	NAI
MIN 12.3	Minsmere Central	MR	MR	MR
MIN 12.4	Minsmere South	MR	MR	MR
MIN 13.1	Power Stations & village	HTL	HTL	HTL
MIN 13.2	Sizewell Cliffs	NAI	NAI	NAI
MIN 13.3	Thorpeness	MR	MR	MR

If the supply from the Easton – Benacre Cliffs were lost due to changes in the SMP, this would probably re-initiate erosion of the Minsmere – Dunwich Cliffs. Unless these were also protected, they would release sediment into the system substituting the loss from BEN 6.1 – SWD 8.1 and in this way the downdrift beaches would still receive a longshore sediment supply. Were the Dunwich Cliffs also protected, rapid erosion around Minsmere Sluice would lead to outflanking (if this had not already occurred), release of some sediment from the eroding shingle barrier and the Minsmere Levels, and regular or permanent flooding of the northern Levels.

Following the logic presented, a loss in supply of longshore sediments to the Sizewell frontage is considered to be not plausible (or very unlikely) and therefore is not considered.

2.5 Resistance and resilience of coastal geomorphology receptors to construction – Sizewell B example

Potential impacts arising during the construction and operation of Sizewell C can be foreseen by examining the shoreline response to the construction and operation of the adjacent Sizewell A and Sizewell B power stations. Sizewell A construction started in 1959 and was completed in 1965, however there is little documentary evidence of effects of construction on coastal geomorphology. At some stage during its operation a salient formed in the lee of its outfall structure and disappeared soon after the onset of decommissioning – that is the slight bulge in the coast disappeared and the shoreline straightened.

A similar feature was observed in aerial photographs around ten years after Sizewell B began to operate. It probably formed as a result of the reshaping of the outer longshore bar once it migrated into the area occupied by the Sizewell B outfall discharge (at a rate of 51.5 m^3/s) during 1997 and 2003. As the bar cannot co-exist with the operational outfall (due to turbulence and scour from the structure and its discharge jet), it has curved seaward of the outfall, which may have altered wave refraction patterns inshore and led to development of the salient. Impacts due to Sizewell C's nearshore outfalls is unlikely to produce a salient because tunnels are subterranean not through the sea-bed sediment, and the outfalls and discharges are substantially smaller (> 100 times less).

In Figure 24, the timeline of marine activities for Sizewell B is presented. A BLF was constructed in 1989, consisting of a solid concrete platform extending across the intertidal zone and a 56 m long mooring jetty. A navigable jetty access channel was dredged to a depth of -5.5 m OD across the nearshore zone, including both the outer and inner nearshore bars. To facilitate the installation of the cooling water outfalls, a 150 m wide sheet pile coffer dam was built along the landward side of these culverts, the face of which was exposed to the sea (i.e., the beach was removed or eroded as waves reflected off the vertical coffer dam).

Longshore sediment transport was disrupted by the presence of the BLF and the cofferdam structures in the subtidal and intertidal environment, specifically with the blockage of the intertidal shingle transport corridor. In comparison, Sizewell C's BLF would be piled with solid components recessed away from the beach, making it transmissive.

Furthermore, during the construction of Sizewell B, a shallow 400–500 m-long bay developed between the BLF in the north and a groyne south of the coffer dam (for the intake and outfall culverts). It is likely that the bay was formed by dredging of the intertidal beach and the nearshore, with very limited potential to recover due to the BLF in the north, groyne in the south and low net longshore transport rates. Additionally, turbulence caused by waves reflecting off the coffer dam wall would have inhibited deposition.

The resistance of the beach to the direct action of culvert and coffer dam dredging was low – the beach was quickly lost due to direct dredging – meaning that beach resistance to the BLF cannot easily be assessed, due to this direct removal of the beach to the south of the BLF. However, a slight build-up of material on the northern side of the BLF indicates that, as a solid feature blocking longshore beach shingle transport, it would have inhibited any natural recovery during construction.

The following examples show a lower resistance, lower resilience (i.e. longer recovery) and larger impact extents than would be expected for Sizewell C. However, they are useful in indicating scales worse than the proposed development.

- Substantial capital dredging of the nearshore was required for Sizewell B: the largest impacts were caused by dredging for culverts (640,000 m^3) and the approach channel to the BLF (83,000 m^3). Maintenance dredging of the beach (13,471 m^3) was required over 150 m of frontage alongside the sheet pile coffer dam associated with the culverts (giving a total capital dredge in the nearshore zone of 723,000 m^3). This is substantially greater than the 51,535 m^3 nearshore capital dredging for Sizewell C construction, which consists of 4,600 m^3 per year for ten years the BLF approach and 1,845 m^3 for each of the three nearshore outfalls.
- Approximately 82,985 m^3 of sediment was removed from the nearshore system (net loss), whereas Sizewell C would remove none.
- Maintenance dredging ceased in November 1991 and the coffer dams and BLF were removed by summer 1992 and the southern groyne by August 1993.
- The recovery (infilling of the bay to restore the naturally straight coastline) included a small 5,000 m^3 recharge (1993), which would have aided recovery but is less than half of the directly dredged beach volume (13,471 m^3). Therefore, much of the beach recovery would have been through natural infilling.
- Following removal of the coffer dam and BLF, the bay infilled and eventually disappeared between 1995 and 1997, restoring the naturally straight coast (Figure 26 and Figure 27). The natural restoration of the shoreline took 2–4 years, which indicates resilience time scales from a substantially more severe impact that predicted for Sizewell C. As the dredging would be subtidal (no direct beach dredging), less than 10% of Sizewell B's, and the sand and shingle transport corridors would not be blocked during construction or operation (current baseline), the impacts would be substantially smaller and recovery times (resilience) much faster.
- The Sizewell C intakes and outfalls are offshore of the Sizewell – Dunwich Bank and the associated dredging (93,100 m^3 in total; 17,400 m^3 per intake head and 11,750 m^3 per outfall head) would have no impact at the shoreline.

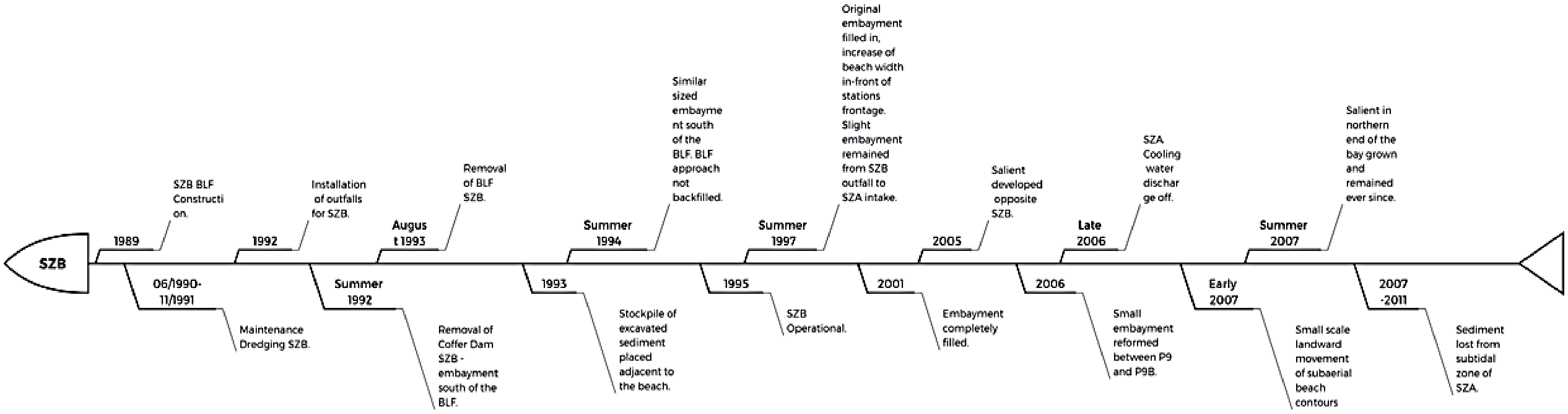

Figure 24: Timeline of marine activities at Sizewell B.

Figure 25: Aerial photographs of the coast during Sizewell B construction. Oblique aerial photograph looking west, showing details of the beach landing facility (BLF) on the right and the intake and outfall culverts on the left (May 1990). Note the removal of the beach and replacement with coffer dam at a position landward of the adjacent shoreline. The resulting bay is likely to be the result of dredging for the cooling water culverts. Bottom: North looking aerial photography (November 1992) following removal of the culvert coffer dam showing the partly constructed sea defences and the embayment formed at the position of the coffer dam and in the lee of the BLF.

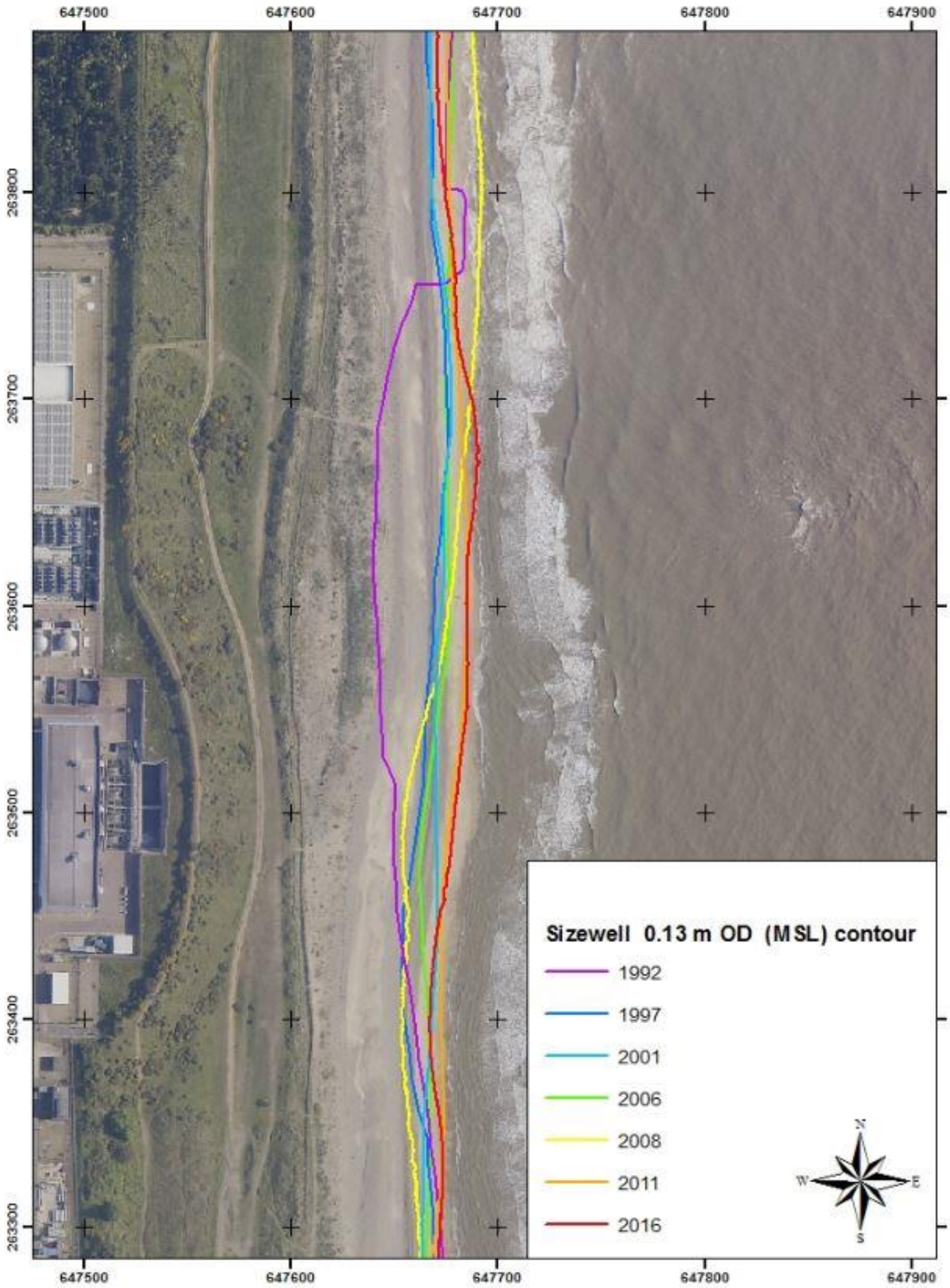

Figure 26: Aerial photograph of the Sizewell B frontage with superimposed 0.13 m OD summer shorelines derived from other aerial photographs, illustrating foreshore recovery following the removal of the coffer dam (1992) and BLF (1993) (taken from BEEMS Technical Report TR223 Edition 3).

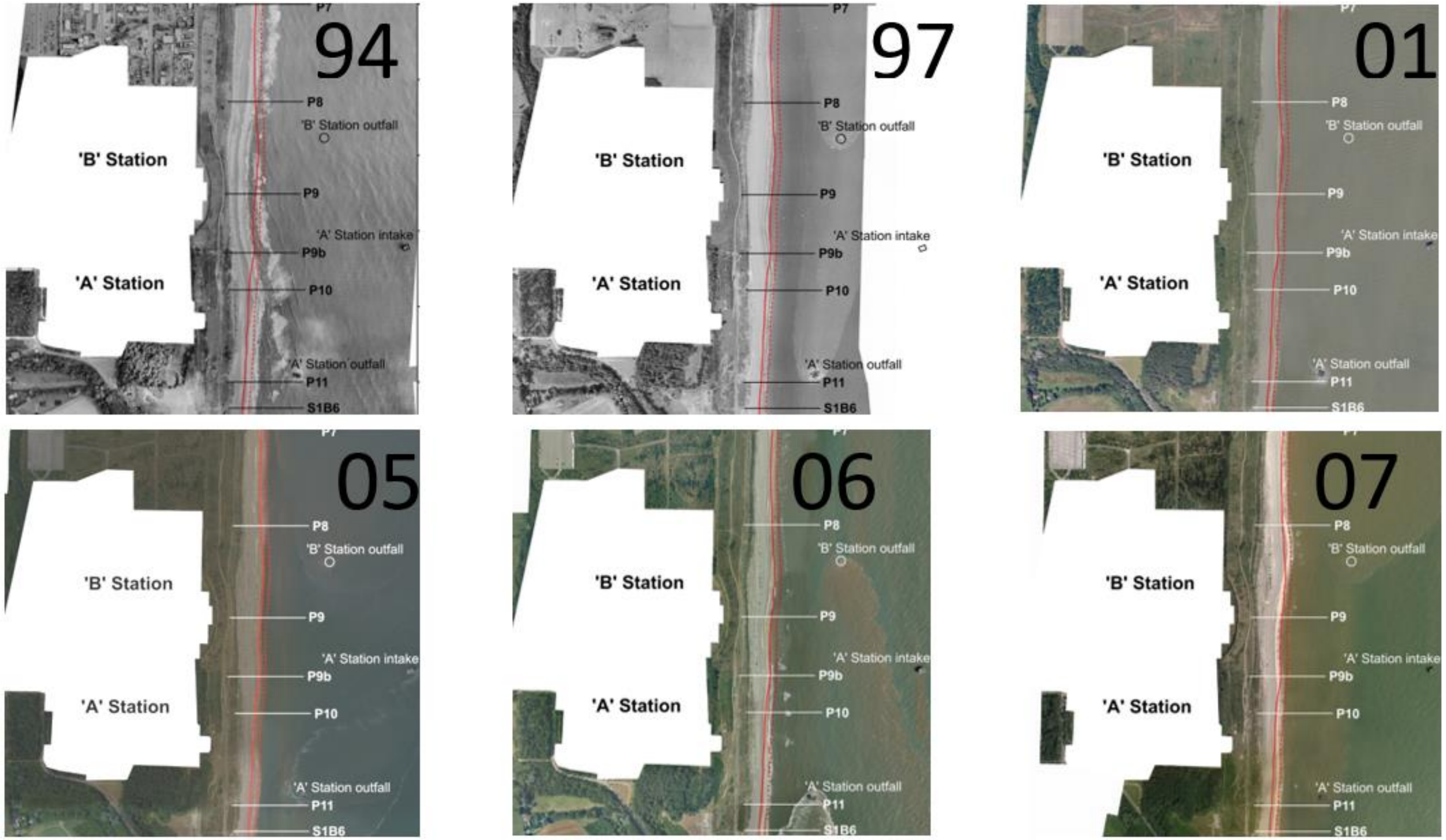

Figure 27: Aerial images showing recovery of the Sizewell power stations frontage following Sizewell B construction. The solid and dashed red lines are the 2011 MHW and MLW shorelines, respectively (taken from BEEMS Technical Report TR223).

3 The marine components of Sizewell C

The marine components of the development site include:

- Hard and Soft Coastal Defence Features (HCDF, SCDF);
- Beach Landing Facility (BLF);
- Offshore Cooling Water Intakes (4) and Outfall heads (2);
- Nearshore Fish Recovery and Return (FRR) system outfall heads (2); and
- Nearshore Combined Drainage Outfall (CDO).

The position of the marine development components is shown in Figure 28.

3.1 Study area and zone of influence

The GSB is the reference area or Zone of Influence (ZoI) for coastal geomorphology and extends from Walberswick in the north to the Coralline Crag formation at the apex of the Thorpeness headland in the south. The seaward boundary extends to the eastern flank of the Sizewell-Dunwich Bank and includes the cooling water infrastructure on the east side on the bank (Figure 28). The landward limit of the marine study area is delineated by MHWS (although migration of the MHWS due to coastal erosion and any new impacts arising is considered in Section 7). Activities that could be affected by storm surge or future sea level rise are also considered. Although some effects will be felt outside the GSB (for example dredge plumes), they have no significant effect on the coastal geomorphology receptors. Nonetheless, they are still included in this synthesis extending beyond the GSB as they are relevant for some of the Ecology EIAs (see Book 6, Volume 2, Chapter 22 of the Environmental Statement).

3.2 Hard and soft coastal defence

Sizewell C would have a hybrid coastal defence comprised of a hard engineered feature that would protect the eastern and northern flanks of the main site, and a fronting sacrificial soft feature made of beach grade sediments and vegetated soil. Similar material would be used to dress the hard feature in keeping with the surrounding landscape where possible. The hard feature is designed to be exposed, although for much of Sizewell C’s operation it would have a natural or maintained beach frontage.

Hybrid solutions that combine hard and soft features fulfil the requirements of high levels of protection, adaptability to future challenges related to climate change, sustainability, and pleasing natural aesthetics (Almarshed et al., 2019). This intentional alignment of natural and engineering processes to efficiently and sustainably deliver economic, environmental, and social benefits is also known as ‘Engineering with Nature’. As well as maintaining local aesthetic values, the soft feature is dynamic, can evolve or be replenished and provides an additional source of sediment to the coast.

3.2.1 Hard Coastal Defence Feature (HCDF)

The HCDF would be constructed from the north, beginning with the development of a 5.2 m ODN platform and haul road, which would connect to the Beach Landing Facility (BLF) deck (Figure 28). The materials and rock armour to build this first part of the HCDF and the BLF will be supplied from land. Once the BLF has been constructed and secured, it would be used to bring in the rock armour component of the HCDF, which would be placed along the seaward margin of the HCDF and its northern flank. Storage of unplaced rock armour would be on the Sizewell C estate landward of the 5-m ODN barrier. The HCDF would be developed in a series of phases (to be described in the LDA landscaping scheme) with additional rock armour being brought in through the BLF to address each stage. Abnormal Indivisable Loads (AILs) and other marine freight would be delivered via the BLF at a later stage within the construction programme (see Section 5.1.4).

These works would be constructed and maintained landward of the current 5-m ODN barrier/dune, and therefore are not in the marine environment as it presently stands. Sheet piling with rock armour would most likely be used around the BLF deck abutment and the haul road. Under very extreme conditions (overtopping of the 5 m barrier), the sheet piling could be exposed to seawater. Coffer dams may be required to place the initial toe of the HCDF, but they would be set further back and are unlikely to be exposed to the sea during construction, so long as the present beach width and elevation are maintained.

3.2.2 Soft Coastal Defence Feature (SCDF)

The Soft Coastal Defence Feature (SCDF) would be sacrificial and made of landscaped beach grade sediments (primarily shingle, to aid longevity) at 5.2 m ODN elevation between the HCDF and the MHWS (a distance of around 35 m)(Figure 29). It would cover any parts of the HCDF below this elevation. Based on the current beach and barrier topography, approximately 120,000 m^3 of sediment would be needed to create the SCDF. Initial investigations suggest the shingle won from the excavation of the footings for the HCDF will be of suitable size and quality to be used as source material for construction of the SCDF. This is subject to a further suitability assessment once excavations begin; otherwise sediments for the SCDF would be delivered to the site (from a licenced aggregate extraction site and/or using excavated material from the MDS) rather than reprofiling the beach. The SCDF would result in a substantial volumetric increase of the back-beach area. The SCDF would be progressively eroded when elevated water levels were high enough to reach it and wave run-up fast enough to entrain its sediments. The sedimentary material presently in this location will be impacted during construction, as required for landscaping and some aspects of the HCDF or BLF (e.g., the installation of the BLF decking at c. 5.2 m ODN across the 5 m bund and/or SCDF which has the same elevation). As the SCDF is sacrificial, it would be managed once largely depleted as required by the beach monitoring and mitigation plan (MMP) – see Sections 6.5 and 7.5 for a high-level overview; thresholds would be set in the MMP. The MMP would be produced post-DCO/Marine Licence (ML) and would require approval from the Marine Management Organisation (MMO) (with MTF consultation). Additional Mitigation (also called Secondary Mitigation) is planned should the HCDF be at risk of exposure (see Section 7.5 – 7.7). Although maintenance of the SCDF is not required for site integrity (the HCDF serves that function), EDF may elect to maintain the SCDF over and above the requirements of the beach MMP.

Source of material would be from a licenced aggregate extraction site or from earth works on the MDS, which would qualify as a form of beneficial re-use. Placement of shingle has been done successfully at sites in the UK such as Dungeness and Horsey Island. At Horsey Island ABPmer (2016) report that “benefits can persist over at least two or three decades (including a period which has seen major storm events) and provide a cost-effective flood defence mechanism”.

3.3 Beach Landing Facility (BLF)

A Beach Landing Facility (BLF) will be used to import rock armour, AILs and marine freight during the construction phase, and occasional AILs over the operational life of the site (Figure 30). During Sizewell C’s operational life, AIL maintenance deliveries will be required for 2–4 weeks once every 5-10 years (approximately). During the maintenance phases, the BLF would be in operation for less than four weeks (notwithstanding unexpected poor weather).

The BLF consists of an 85-m-long piled deck that abuts to the haul road on the 5.2 m ODN platform of the HCDF. This section of the HCDF protrudes further seaward (see Figure 30) to accommodate the haul road and the turning radius required for AIL vehicles.

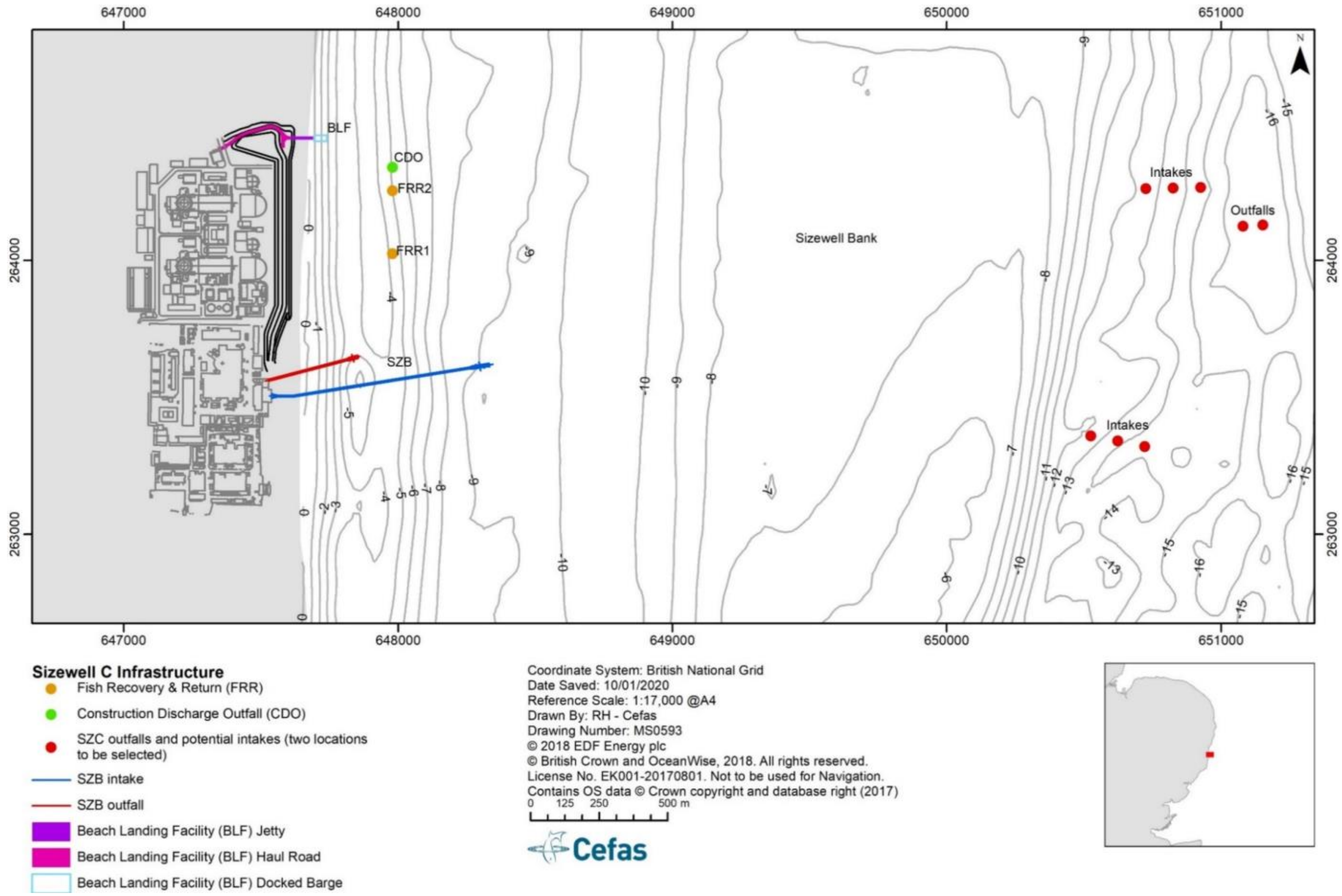

Figure 28: Marine components of Sizewell C and the intake and outfall locations for Sizewell B. The black lines east and north of Sizewell C mark the HCDF.

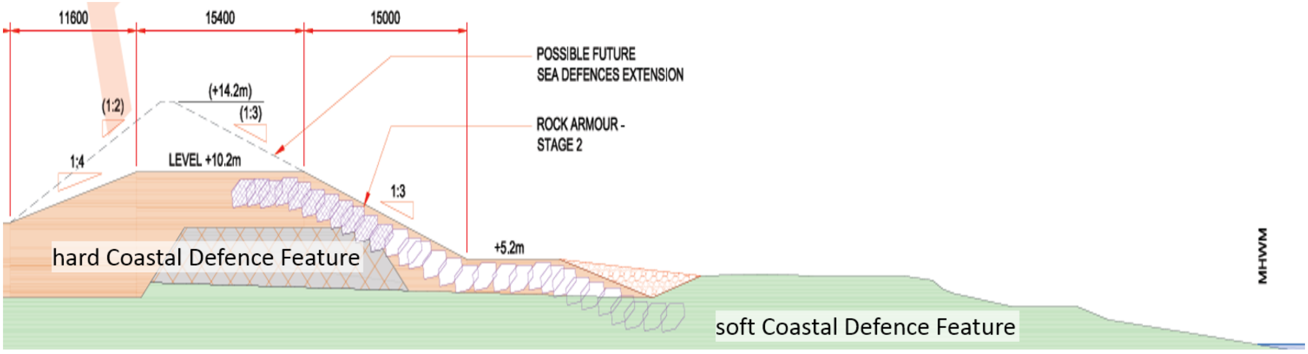

Figure 29 Cross-section of the hybrid coastal defences at Sizewell C. Note that this drawing is indicative and does not include the correct foundation depths.

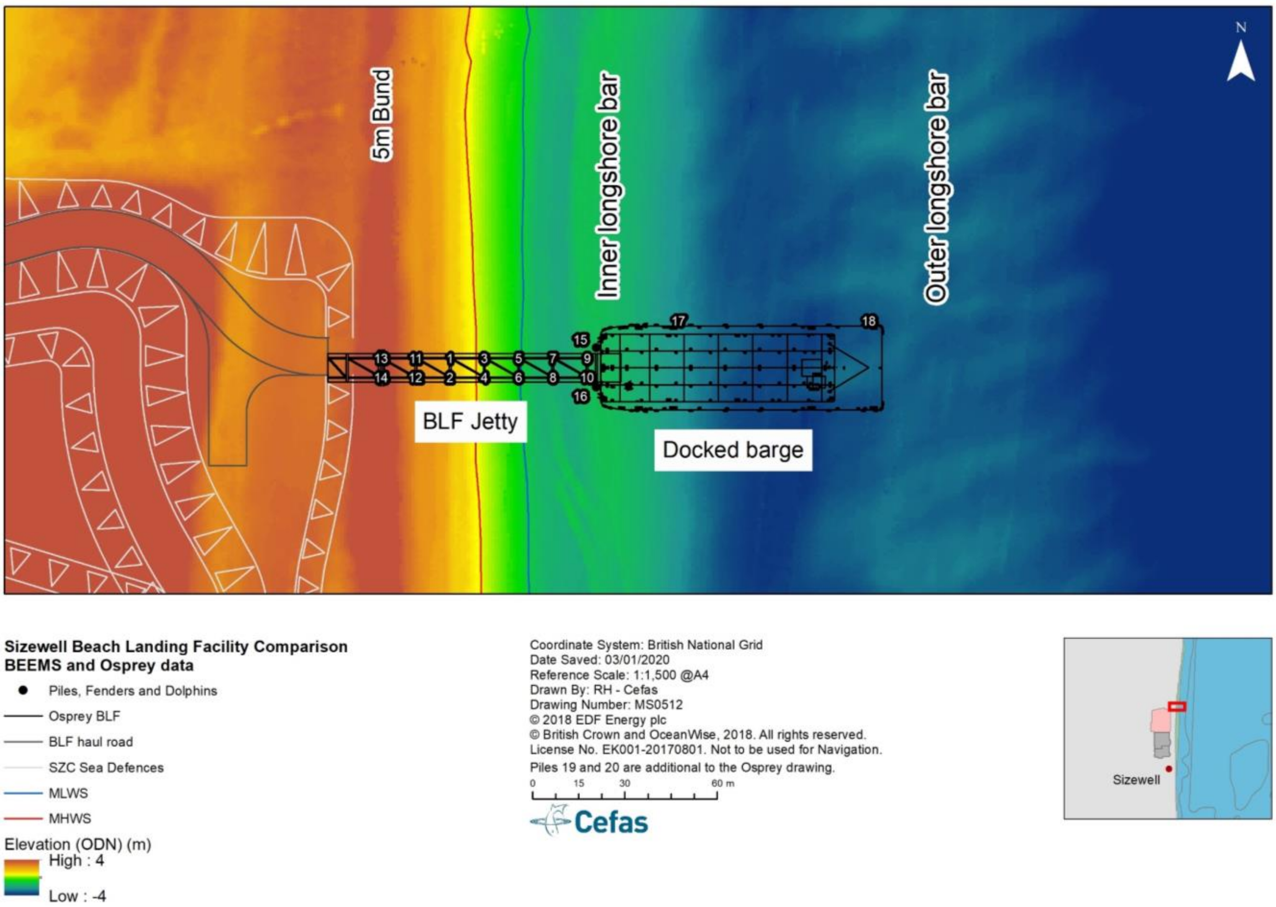

Figure 30: Beach Landing Facility (BLF) deck, fenders (labelled 15 and 16) and dolphins (labelled 17 and 18) shown together with a docked barge.

The BLF would be located at c. 264446 N and be 176 m long (from the HCDF to the seaward dolphin). It would consist of an 85 m long piled deck plus additional 11 m of fenders and ramp. The last 36.5 m of the BLF deck would be seaward of MHWS, and mooring dolphins would be positioned at approximately 66 m and 128 m from MHWS. Additionally, the BLF would consist of mooring dolphins (2, north side), fenders (2) and a piled deck that would connect to the HCDF and the abutment terminating at the AIL haul road (c. 5.2 m ODN). The modelled worst-case deck piles would be approximately 1 m ∅ and the fender/dolphin piles approximately 1.55 m ∅. Three BLF deck pile pairs are landward of low tide, one pair is close to the low tide mark, and three pairs are seaward (i.e. the final four pile pairs are within the marine environment below

MHWS. With shoreline retreat, the number of marine piles would rise to a maximum of 18 (i.e., all) piles. The deck piles would have an approximate 11.2 m cross-shore spacing and 6.3 m between each pair. The BLF deck would overhang each pile centre by 1.5 m (so the deck would be 9.3 m wide).

The landward sections of the BLF deck would be constructed by a terrestrial piling machine operating from land or the BLF deck itself using a cantilever approach. The marine piling would be undertaken using the cantilever method or a walking jack-up barge or similar (e.g., WaveWalker). Dolphins would be installed from a standard or walking jack-up barge.

Barges would transit over the nearshore bars to the end of the deck at high tide and would become grounded as the tide falls; offloading is expected to be completed within one tidal cycle. Finally, a plough dredger (Figure 31) would be used to dredge the longshore bars for navigational access and a grounding pocket for docked barges (for more details and potential impacts on coastal geomorphology see Section 4.2).

3.4 Vessel movement

Vessel movement will be considered as part of a separate navigational assessment (Chapter 24 of this volume) and is a requirement for dredging required to allow vessels to berth for the BLF. A route for navigational access to the vicinity of the BLF will need to be guaranteed in the longer term as the ability to deliver AILs on occasion throughout site life will be critical to commercial viability. Access to the BLF is needed in order to perform operational maintenance; there is no planned emergency usage.

Although the existing depth and width of channel inshore of the Sizewell – Dunwich Bank serve this function in the near term, long term geomorphological change may reduce this to the point where some dredging works might be required, although this is an unlikely issue for barges and accompanying tugs due to their shallow draught. The need for any such works would be reviewed on a case by case basis in advance of any planned AIL delivery in the event of significant bathymetric change. As location and quantity of material to be extracted is unknown (or not reasonably foreseeable) it has been excluded from the assessment. However, dredging required to clip the outer bar for navigational clearance and the inner bar for barge grounding pocket is considered (see Sections 4.2.1.3 and 4.2.2.5).

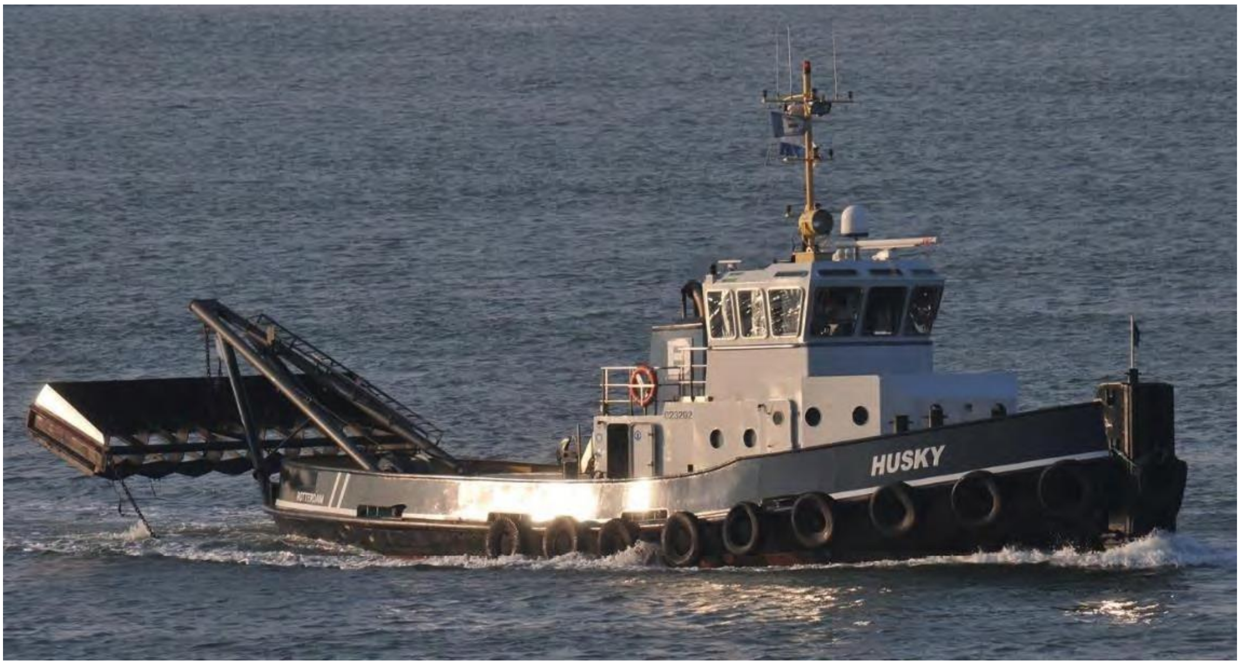

Figure 31: Example of a plough dredger

3.5 Intakes and outfalls

Two subterranean 8 m diameter cooling water intake tunnels (internal tunnel diameter 6 m) and one 9 m diameter outfall tunnel (internal tunnel diameter 8 m), both approximately 3 km long, would be excavated by Tunnel Boring Machines (TBMs) from land. Excavated arising's will be transported landward on a conveyor to a muck bay. The TBM heads would be left at the end of each tunnel run, approximately 30 m under the seabed. Additionally, two vertical connecting shafts will be driven down to meet each tunnel, done 'in the wet', offshore of the Sizewell – Dunwich Bank (total of six shafts – four intakes, two outfalls). Associated drill arisings would be disposed of locally. Seabed headworks structures would be mounted at the head of each shaft (total of four intake (32.5 x 10 m intake heads would protrude above seabed level) structures and two outfall (16 x 16 m) structures).

The outfall design would have four piles drilled into the bedrock (of approximately 16.6 m in length and 2.1 m in diameter), securing the heads to the bedrock to achieve seismic qualification. The outfall would have a foundation chamber with dimensions of 16 m x 16 m length and width, and 4.9 m in height. The top of the outfall head is likely to be trapezoidal with a width of 10.5 m at the outlet, and about 6.6 m at the rear; it is 9.3 m long and 3.2 m high. Thus, the total height would be approximately 8.1 m from the bedrock and 4 m above the seabed level. The outlet would consist of two openings, each approximately 4.1 m wide and 2 m high. The top of the outfall head would contain a removable cover slab of radius 3.3 m. Both the intake and outfall heads are intended to have an 85-year lifetime (to include operation and decommissioning phases). It is understood that the outfalls would discharge cooling water at a rate of 66 m^3/s each.

The design of the Sizewell C intake structures is not yet complete but they will be Low Velocity Side Entry (LVSE) designs based upon the Hinkley Point C (HPC) designs. Finally, scour protection would be placed around the headworks if necessary (see Section 4.4.2). These system components will be sized for ~132 m^3/s total flow.

3.6 Fish Return and Recovery (FRR), and Combined Drainage Outfall (CDO)

The design of the FRR outfall heads has not yet been undertaken but would comprise a concrete block approximately 3 m long, 4.5 m high, and 3 m wide (subject to final engineering design). The northerly position of the two FRRs is designed to be in alignment with the forebays of each reactor, thus minimising the required tunnel length and hence the time taken for fish to be returned to the marine environment. The optimal easterly position was determined by a number of antagonistic factors, including;

- The depth of the water at the point of discharge, i.e. water depth must be sufficient at all stages of the tide to reduce predation by surface feeding.
- Avoidance of mobile geomorphic features; the two nearshore bars at Sizewell are important to sand transport and move naturally in response to the prevailing wave climate. The bars must be cleared to avoid burial of the system and so the FRRs (and CDO) have been positioned on the seaward flank of the outer longshore bar. They are unlikely to affect expected landward migration of the bars (with rising sea levels).
- Proximity of the Sizewell B discharge plume; the Sizewell B outfall is positioned 150 m offshore (from mean water level). A short FRR tunnel was not selected because it would release fish into the Sizewell B TRO plume on the ebb tide. The Sizewell B cooling water discharge is chlorinated throughout the year.
- Risk of fish re-impingement into Sizewell B; the Sizewell B intake is 600 m offshore and there is a risk that on the flood tide some of the fish discharged from the FRR outfall could be re-abstracted at the Sizewell B intake.

The current positions of these nearshore outfalls are:
- FRR 1 head: Easting 647980, Northing 264024.
- FRR 2 head: Easting 647980, Northing 264254.
- CDO head: Easting 647980, Northing 264340.

The final position of the CDO relative to the FRRs would be planned to minimise the possibility of fish released from the northern FRR being exposed to any potential operational discharges that may be released from the CDO.

4 Potential effects of Sizewell C on the coastal geomorphology receptors

This section details the potential effects of the construction and operation of the marine components of Sizewell C on the coastal geomorphology receptors, which are the shoreline/beach, longshore bars, Sizewell – Dunwich Sandbank and the Coralline Crag near Thorpeness. The marine components of Sizewell C, which are detailed in Section 3, would be:

- Soft and Hard Coastal Defence Features (SCDF and HCDF);
- a Beach Landing Facility (BLF);
- cooling water intakes and outfalls;
- Fish Recovery and Return outfalls (FRRs) (2); and a
- Construction Discharge Outfall (CDO) (sewage and chemicals).

An activity–pressure matrix relevant to coastal geomorphology and hydrodynamics was developed to identify the pressures of each activity that could affect the receptors (i.e., a pathway to impact) and this forms the basis of the effects discussed in this section (Figure 32).

The EIA also requires separate consideration of potential effects arising under a future baseline. A future shoreline baseline, and the additional effects that could arise from future shoreline recession at Sizewell C, is considered in Section 7.

Note that the terms construction and operation are generally used to refer to the construction and operation phases of the Sizewell C.

4.1 Coastal defence features

The Sizewell C coastal defence features would be built landward of MHWS and consist of a hard and a soft (sedimentary, sacrificial, landscaped and vegetated) components (Figure 28 and Figure 29). The HCDF would be terrestrial, set well back from the coast and landward of the present 5 m (ODN) dune/barrier. As the HCDF would be constructed terrestrially, and it is unlikely to affect coastal processes and geomorphic receptors until the middle or late stages of station operation, it is considered in Section 7 (future baseline).

The construction and future presence of SCDF does have potential to affect coastal processes on the present baseline (through provision of additional sediment that would not otherwise be available) and is discussed in the following two sections.

4.1.1 Soft Coastal Defence Feature – construction

The SCDF would be constructed of beach grade materials placed on the Sizewell C frontage between the HCDF and MHWS at an elevation of 5.2 m ODN, with a suitable slope to MHWS at its seaward extent. The initial estimated volume would be 120,000 m^3, based on adding sediment to the present-day topography. These sacrificial sediments would be landscaped and planted with suitable vegetation for aesthetics and to aid in longevity (erosion resistance). Machinery used to deliver and landscape the sediments would mostly be operating in the terrestrial environment (i.e. no marine effects), however in order to place and landscape sediment near the MHWS mark, it is inevitable that vehicles would need to access the beach face. The use of heavy vehicles would cause limited compaction of beach sediments, temporarily increasing their resistance to erosion. The effect on geomorphology would be localised and insignificant. As the 5 m bund is likely to remain in place and the work is likely to take place in summer (minimal wave run up), any additional wave run up would have minimal effect on the beach. We note other similar activities with heavy vehicles (e.g. cable landfalls) have occurred (i.e. been permitted) on this beach.

	Pressure Theme	Hydrological changes (inshore / local)						Physical damage				
		H1	H2	H3	H5	L1	L2	D1	D2 - Abrasion sub-categories		D3	D4
Construction Process at SZC	Pressure / Activity	Temperature changes - local	Salinity changes - local*	Water flow (tidal current) changes - local	Wave exposure changes - local	Physical loss (to land or freshwater habitat)	Physical change (to another seabed type)	Habitat structure changes - removal of substratum (extraction)	Penetration and/or disturbance of the substrate below the surface of the	Penetration and/or disturbance of the substrate below the surface of the	Changes in suspended sediment/solids	Siltation rate changes
Beach Landing Facility (BLF)	Heavy plant operations on beach								C			
	Installation of rock platform			C	C	C	C				C	C
	Wave walker			C	C	C	C		C		C	C
	Piling			C	C	C	C		C		C	C
	Presence of structure			C O	C O	C	C		C		C O	C O
	Navigational dredging			C O	C O			C O	C O		C O	C O
	Mooring of barge at BLF			C O	C O	C O					C O	C O
	Ballast Water										C O	C O
	Spills and accidents										C O	C O
	Vessel Traffic										C O	C O
Coastal Protection Features	Heavy plant on beach								C			
	sCDF construction - additional sediment on beach			C	C				C			
	sCDF operation - additional sediment on beach during storms								O			

Figure 32 (a) Potential effects (pathway) for Sizewell C marine components. No works landward of the HCDF affect coastal geomorphology receptors, other than site drainage, which is assessed under the CDO. Continued on next page.

Cooling Water Intakes and Outfalls heads (including drilling of connecting shafts and tunnelling for Cooling Water Systems	Dredging			C	C	C	C	C	C	C	C
	Dredge spoil			C	C					C	C
	Drilling for connection tunnels					C	C		C		
	Seismically-qualified head installation			C	C				C		
	Construction platform operations			C	C	C	C		C	C	C
	Physical presence of structure			C O	C O	C	C			C O	C O
	Water abstraction			O	O						
	Operational discharges			O	O						
Fish Return and Recovery System, connecting tunnel and construction drinage outfall (FRR x 2, CDO)	Dredging			C	C			C	C	C	C
	Dredge spoil			C	C			C		C	C
	Drilling for connection tunnels					C	C		C		
	Seismically-qualified head installation			C	C				C		
	Construction platform operations			C	C	C	C		C	C	C
	Physical presence of structure			C O	C O	C	C			C O	C O
	Discharge			O						O	O
	Return of biota									O	O
Main Site construction (drainage assessed under CDO)	All site works landward of the HCDF										

	Impact pathway, assessment required
	Impact pathway, but negligible so not assessed
	No impact pathway
C	Construction Phase Impact
O	Operations Phase Impact

Figure 32 (b): Potential effects (pathway) for Sizewell C marine components. No works landward of the HCDF affect coastal geomorphology receptors, other than site drainage, which is assessed under the CDO.

4.1.2 Soft Coastal Defence Feature – use

The placement of the SCDF would alter the beach in two ways – it would change the profile above MHWS and increase the beach volume in that area. As a result of the larger back-beach volume, SCDF sediment would increase coastal resilience by reducing the rate of shoreline retreat through episodic introduction of relatively small volumes of extra sediment during storms with high water levels. The SCDF is expected to last for decades before it would be fully depleted (see Section 7) – due to the low rates of erosion (0.09 m/yr ± 0.76 m/yr between northings 263094 and 264498) on the Sizewell frontage. However, note that the retreat rates increase toward and beyond the north-east corner of the Sizewell C frontage i.e., the Southern Barrier (-0.58 ± 0.32 m/yr (northing: 264002 to 264498 with a maximum of -1.37 m/yr at northing 264498). Material eroded from the SCDF during storm event would supply extra sediment in a fashion similar to a small-scale sand/shingle engine.

The effect on the coastal geomorphology receptors of the extra sediment would be minor because the small volumes of material would be released incrementally and dispersed during each erosive storm. Evidence from Storm Emma and the weakening polar vortex (The Beast from the East) in March 2018 showed substantive reprofiling of the intertidal and supra-tidal beach. However, there was limited erosion of the shingle barrier itself – the 1 m^3/m eroded from the barrier toe (above 3 m ODN; see BEEMS Scientific Position Paper SPP094) equates to approximately 3 cm of raised beach elevation were the material spread evenly over the active beachface.

Unmitigated beach recession would eventually lead to exposure of the HCDF. This potential outcome could disrupt longshore sand and shingle transport, and may require intervention (bypassing, beach recycling or beach recharge) to prevent or minimise any disruption. Such Additional Mitigation would be some decades into the future and so is considered under the future baseline in Section 7. Any such required mitigation will also be specified in the proposed ML Geomorphology / coastal monitoring conditions should construction proceed.

4.2 Beach landing facility (BLF)

4.2.1 Beach landing facility (BLF) construction

The terrestrial piles of the BLF deck would be installed from a terrestrial piling machine or using a cantilever method from the HCDF (no effects on coastal geomorphology). Although the method for marine piling has yet to be determined, it is likely to use a cantilever method from the HCDF (no effects on coastal geomorphology) or a jack-up barge (tugboat or self-propelled). The BLF dolphins would be installed from a jack-up barge.

4.2.1.1 *Terrestrial piling vehicle (beach / nearshore) and vehicle traffic*

The nearshore pile locations cannot be accessed by a jack-up barge and would use either the cantilever method (no effect) or a terrestrial piling vehicle to be driven over the intertidal, and potentially onto the shallow subtidal (i.e., for piles near the low tide mark) beach. Such vehicles typically move on caterpillar tracks and would cause localised disturbance on the beach face (up to 100 x 100 m). Localised compression and resuspension can be expected, but this would be time-limited (mostly when the vehicle is in motion). Within the area of movement, depressions can be expected where the vehicles have been moving but the depth of disturbance will be significantly less than the natural variability in the beach elevation and so any depressions will be quickly infilled and would be superficial. Nearshore piling by the terrestrial piling vehicle is expected to take around two months. As the effects of constructing the intertidal sections of the BLF deck would be localised, superficial and short lived, they would be expected to have no significant effect on the shoreline.

4.2.1.2 *Insertion of marine piles*

At the time of writing of this report, the method of piling is unknown, but is not considered to have a detectable effect on the coastal geomorphology or the shoreline (though sediment suspension levels in the water column may rise slightly if fines are brought up from beneath the sea floor). The effect of a short and localised rise in sediment in suspension would be sufficiently small that it would not affect the shoreline or geomorphology and would be difficult to detect. The drill arisings per pile (18 m^3) represent 2.4% of the volumes drilled for the intake shaft (754 m^3) where suspended sediment concentrations were shown to be undetectable above background conditions; see Section 4.4.1.2. It is currently expected to take two months to complete the BLF deck using piling rigs. The presence of the piles is discussed in Section 4.2.2.1.

4.2.1.3 *Vessel / jack-up barge anchoring*

If used, jack-up barges would be at anchor to pile and assemble the BLF deck. As noted, they would be used for the BLF dolphins as these cannot be installed using the cantilever method. Anchoring would displace a very small quantity of bed sediment as it is laid. The feet of jack-up barges would cover a small area of the sea floor (e.g., total of 12 m^2 for a four-legged jack-up barge with legs of 2 m diameter) and some depressions and scour may result. Scour would increase with distance from shore, due to the increasing (though generally weak; < 0.3 m/s) currents. As the currents are slow and the jack-up barge will only be present in each position for up to a few days, very little scour would develop. As the scour would not have time to reach equilibrium, scour equations cannot be used. As the effects of constructing the subtidal sections of the BLF deck would be localised, superficial and short lived, they would be expected to have no significant effect on the shoreline.

4.2.2 Beach landing facility (BLF) in use (during Sizewell C construction and operation)

During the Sizewell C construction phase, the BLF would be used to receive 178 AIL deliveries, rock armour and other marine freight. Abnormal Indivisible Loads (AIL) that cannot be delivered via the road and rail network, would be required during the construction and operation phases of Sizewell C. During the operational phase of the station, it would only be used once every 5-10 years for maintenance activities. Notwithstanding unexpected poor weather, the BLF would be in operation for less than four weeks during each maintenance phase. The most likely operational window would be April to October, when wave heights are typically low, however this does not rule out potential transit during calm conditions in the winter months.

When in use (during Sizewell C construction and operation), a barge would approach an hour before high tide with the assistance of two tugboats, aiming to dock at high tide. One tugboat would guide from the aft of the barge and the other from the port side (to counter the flood tidal current still flowing southwards just before high tide). Once the barge has docked against the end of the BLF, it would alter its ballast and sink with the falling tide, such that it would rest fully on the seabed at low tide. Once the delivery is unloaded, the barge would re-float with the rising tide and leave with the subsequent high tide.

Dredging would be required when the BLF is in use for safe access over the outer longshore bar, and for the barge to ground flat on the seabed. A North Sea Barge was assumed as the preferred option for assessing the required dredging (BEEMS Technical Report TR481). Figure 33 is a schematic of the required dredge profile, in red, compared to the current (2017) bathymetric profile in front of the BLF. The outer bar would be clipped to a height of -3.5 m ODN to allow clearance of the tugs over the outer bar. Figure 34 shows the area over which the dredging would be required. The total area dredged would be 9,068 m^2, or 0.91 ha. The total area includes the width of the barge to rest centred in front of the BLF plus the length of the tugboat to work tangentially to the barge to provide clearance for safe working. Using the 2017 bathymetry the total dredge volume for the reprofiled approach and grounding pocket (Figure 33) would be 3,850 m^3. As the height of the inner and outer bar are variable, an additional 750 m^3 was added to the total dredge volume modelled (i.e. an extra 10 cm depth), to be conservative, giving a total of 4,600 m^3.

Once construction of Sizewell C has completed, the BLF deck deck would be removed, but the piles would remain in place. The deck would then be reinstated as required for any maintenance activities and the sea floor would need to be surveyed to establish dredging requirements for access and barge grounding. Once Sizewell C has stopped operating and the BLF is no longer needed, it’s piles would be removed to below the sea floor following standard practice for similar structures in the North Sea.

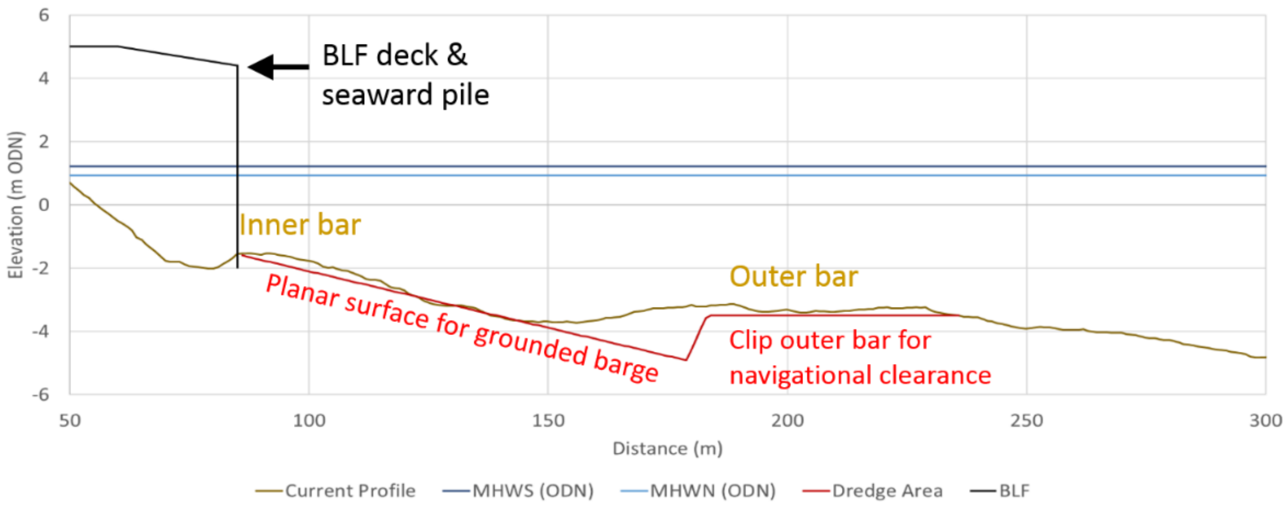

Figure 33: Schematic of the dredged profile (red) required for a North Sea barge to dock at the BLF (black), compared to the current (2017) bathymetry (brown). Mean High Water Spring and Neap are shown in blue (BEEMS Technical Report TR481). Vertical exaggeration is 6.6x.

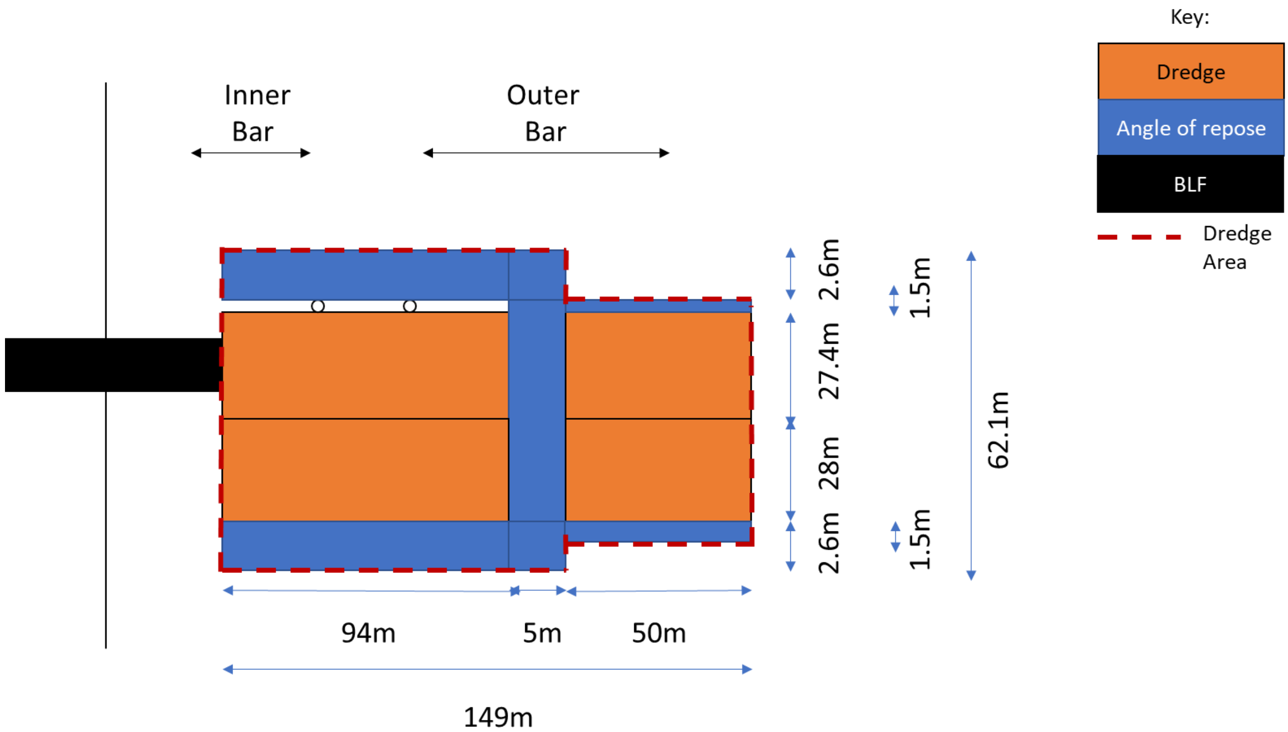

Figure 34: Schematic of the dredge area required for the BLF approach. The dredge area includes barge grounding and clearance for tugboat safe working (BEEMS Technical Report TR481).

4.2.2.1 *Piles*

A total of seven pairs (14) of 1 m diameter steel tubular piles would be installed to support the deck and its marine freight. Additionally, there are two fenders and two dolphin piles, of 1.52 m diameter. Four pile pairs (8 piles) and the fenders and mooring dolphins would be seaward of MHWS, with 1 pair in the intertidal zone near low tide. The low density of slim, circular piles (spacing is 11.2 m cross-shore and 6.3 m alongshore) means that the BLF deck would be transmissive to water and sediment movement, and the local effect on current flow and wave energy transmission would be minimal. The BLF would be located at approximately 264,446 m Northing (BNG) and connect to the haul road on the HCDF 5.2 m ODN platform. The layout of the BLF deck piles, fenders and dolphins are shown in Figure 35.

The effect of the BLF deck piles and reprofiled bathymetry (needed for access and barge docking/grounding) on the local hydrodynamics and bed shear-stress was investigated using very high-resolution numerical simulations of waves and currents using the ARTEMIS and TELEMAC2D models. Figure 36 shows the mesh design for the BLF with incorporation of the piles, fenders and dolphins. The fine mesh density around the piles (0.2 m) is fundamental in order to simulate reflection and diffraction of waves from the piles.

Eight scenarios were modelled with different combinations of wave heights, return periods, wave directions and water levels. A detailed analysis of the bed shear stress is presented for the worst-case scenario (i.e., the largest percentage change), which corresponds to waves from a south-easterly direction with a significant height of 2.20 m (1:0.2-year return interval) and the peak ebb tidal current. The potential effects of the BLF are considered for phases when it is not in use (i.e., the effect of piles only) and when it is in use (the effect of piles and the reprofiled bathymetry).

4.2.2.1.1 Pile scour

Once constructed, the seabed surrounding the BLF deck, fender, and dolphin piles would scour and the local bathymetry would be lowered as a result. It is expected that lowering of the bed would cease when continuity of volume of the tidal flow has been re-established. An empirical assessment of the localised scour was carried out to estimate scour depth and extent around the BLF piles, accounting for hydrodynamic conditions, sediment characteristics and assumed structure dimensions (BEEMS Technical Report TR310 Edition 2).

The most appropriate method for scour depth prediction was conservatively selected from a range of applicable empirical methods. Piles were assumed to be surface piercing cylinders of 1 m ∅ (deck piles) and 1.52 m ∅ (dolphin and fender piles) in the configuration shown in Figure 36, and it is assumed that no scour protection would be installed. Interaction between piles located sufficiently close to each other acts to increase predicted scour depth; this applied to the most seaward deck pile pair (9 and 10) and the two adjacent fender piles (15 and 16).

For geomorphological assessment and for input to the bathymetry used in the BLF numerical modelling (BEEMS Technical Report TR481), scour predictions were based on the spring peak southerly tidal current and assumed scour pit sides with an angle of repose of 32º (Soulsby, 1997). In this scenario, scour depth was likely to be as much as 1.5 m at the most offshore dolphin pile, and 0.7 m at the most landward deck pile pair located in the intertidal zone. The horizontal extent of scour around the piles ranged from 1.1 m for the most landward deck piles to 2.4 m for the most offshore dolphin pile. Predicted scour is detailed in Table 5. The total area affected, including the footprint of the structures themselves, would be 186 m^2. Scour predictions were also computed for modelled climate change and shoreline retreat scenarios: changes were up to -11% decrease and +8% increase in scour depths respectively, and these were incorporated into the model bathymetry for the corresponding BLF modelling scenarios (BEEMS Technical Report TR481).

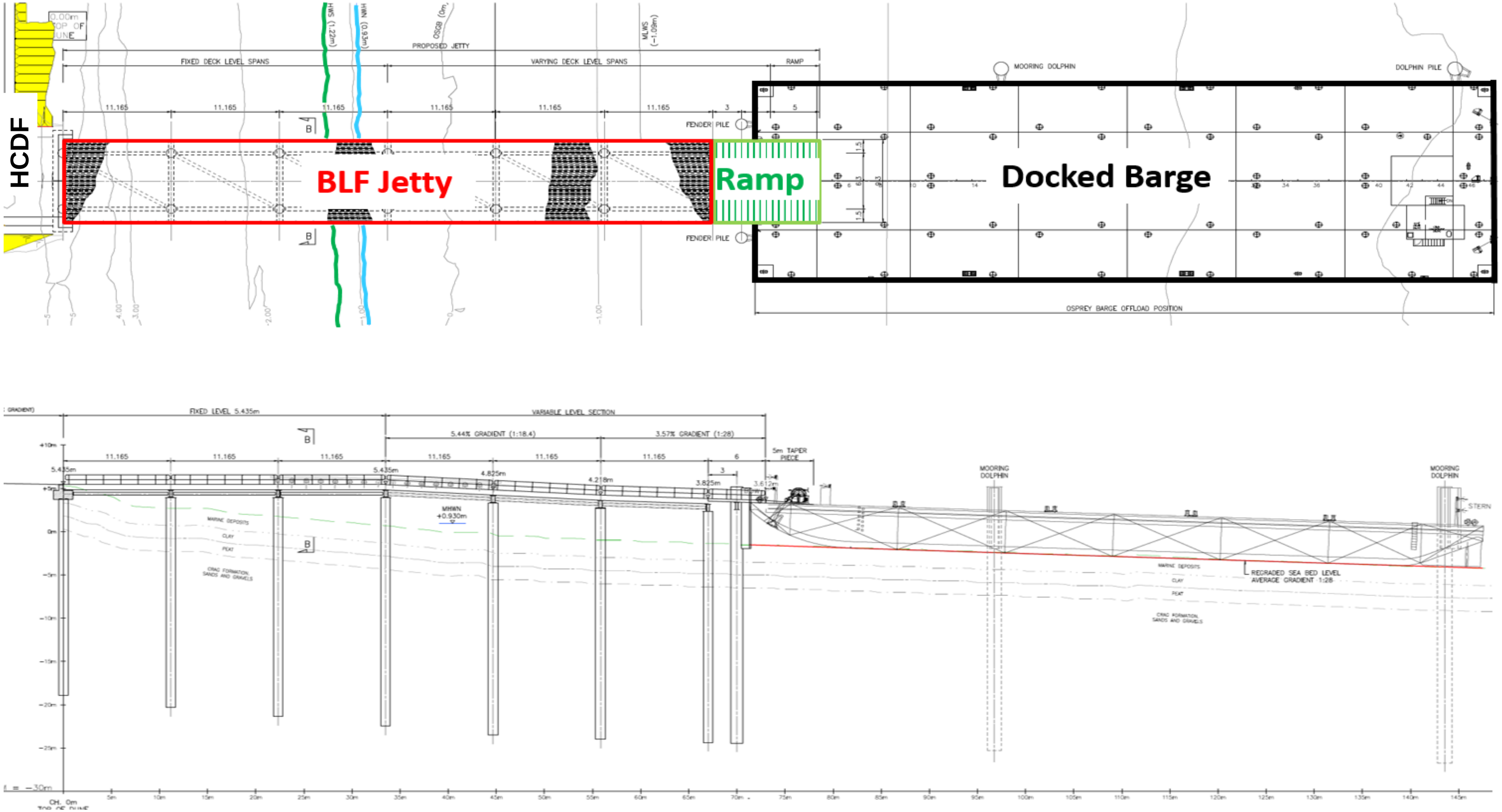

Figure 35: Proposed layout (plan view and cross-section) of the BLF deck piles, fenders, dolphins and a docked barge (BEEMS Technical Report TR481).

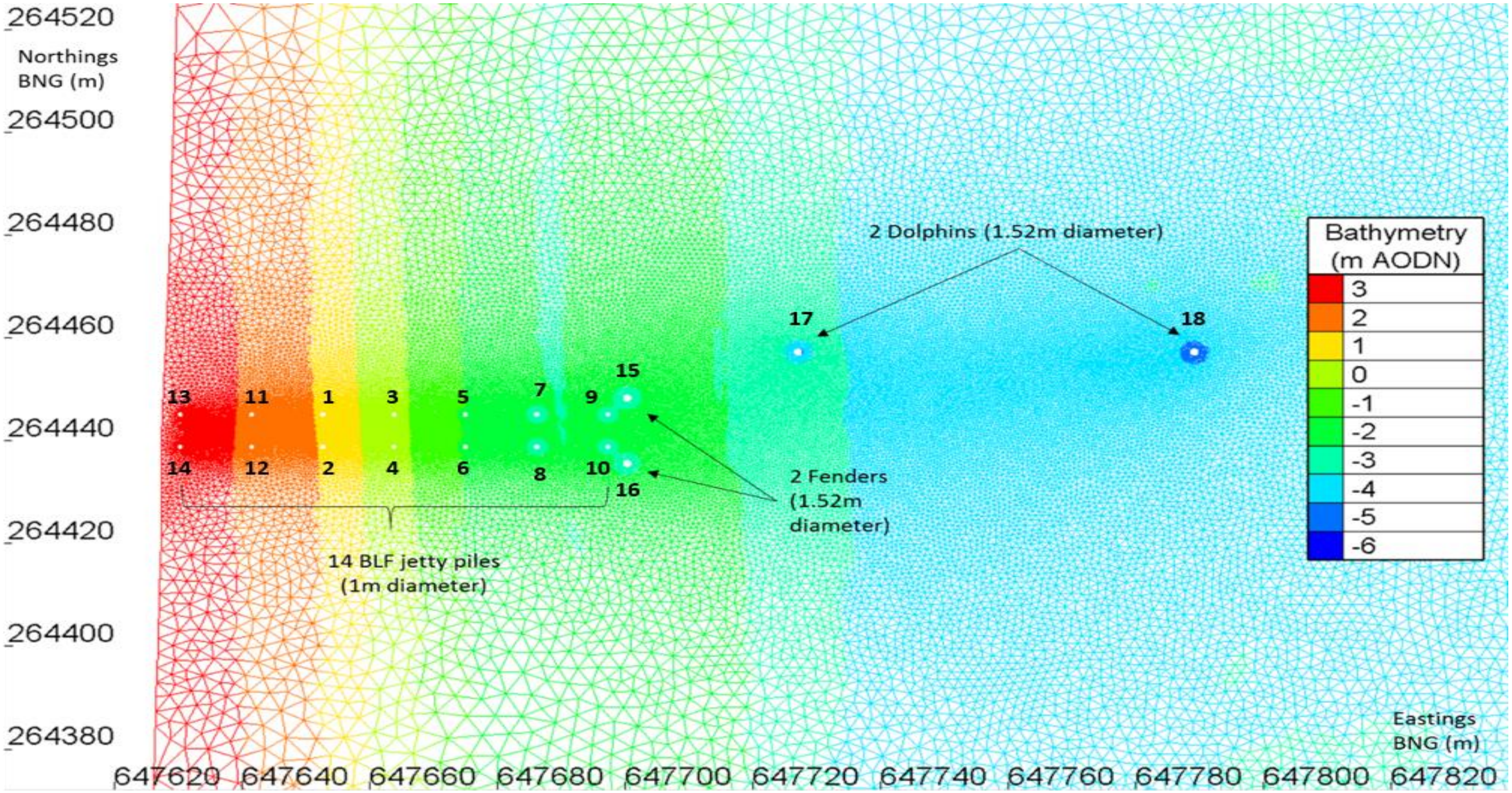

Figure 36: Mesh design for the BLF deck and incorporation of piles, fenders and dolphins. Detail of the bathymetry around the deck, including the addition of lowering the bathymetry around the piles due to scour (BEEMS Technical Report TR481).

For ecological assessments, it was necessary to estimate a conservative area of potential habitat disturbance, based on the modelled peak tidal current during a storm surge event (worst-case). This predicted the total area that could be disturbed by scour development during infrequent extreme conditions, however such scour depths and extents would be rapidly infilled and so not persist outside of such events. Within this scenario, the bases of all piles were submerged except deck piles 11 to 14. The potential for scour due to wave run-up around piles above the modelled storm surge water level was also considered to allow for the most conservative case. As there are no established predictive methods for this phenomenon, it was assumed that scour depth was the same as the largest value predicted for deck piles not exhibiting group scour effects (piles 7 and 8). In this scenario, scour depth ranged from 0.6 m at piles 1 and 2, up to 2.0 m at the fender piles (15 and 16), with corresponding horizontal scour extents along the tidal axis (north – south) of between 2.1 and 7.1 m. Further conservatism was added to the predicted area of influence by assuming the formation of elliptical scour holes due to reversing tidal currents (Harris *et al*., 2010). The total area affected, including the footprint of the structures themselves, was predicted to be 852 m^2. For perspective, this would be equivalent to a c. 10 by 10 m square box per pile pair. The total volume of displaced sediment for all piles was predicted to be 583 m^3. Predicted scour is detailed in Table 5.

Geotechnical vibrocore data approximately 130 – 190 m offshore from the proposed BLF location (see Figure 59) indicated that the thickness of sediment overlying the bedrock was deeper than the predicted scour, and therefore it is unlikely that bedrock would be exposed or limit scour development.

It was predicted that during 1:0.2 and 1:20 year return period wave events, waves would act to backfill the scour pits formed due to tidal currents without waves. Therefore, the predicted values are unlikely to be present during average conditions and represent a worst-case scenario.

Table 5: Predicted scour depths and extent for structures at the BLF. Structure references are labelled in Figure 36.

Structure Reference	Peak spring tidal current scour scenario (input to BLF hydrodynamic modelling)			Peak tidal current storm surge scour scenario (worst case estimates of habitat disturbance)			
	Scour depth (m)	Horizontal extent from structure (m)	Scour pit area per structure (m^2)	Scour depth (m)	Horizontal extent from structure (m)		Scour pit area per structure (m^2)
					N - S	E - W	
13 & 14	-	-	-	1.24	4.32	2.47	36.6
11 & 12	-	-	-	1.24	4.32	2.47	36.6
1 & 2	-	-	-	0.61	2.12	1.21	9.2
3 & 4	-	-	-	1.01	3.54	2.02	24.8
5 & 6	0.70	1.12	8.2	1.16	4.06	2.32	32.4
7 & 8	0.82	1.31	10.3	1.24	4.32	2.47	36.6
9 & 10	1.16	1.86	17.5	1.72	6.01	3.43	69.4
15 & 16	1.38	2.21	27.8	2.03	7.08	4.04	97.8
17	1.35	2.15	26.6	1.80	6.26	3.58	77.1
18	1.50	2.40	31.3	1.93	6.74	3.85	88.8

4.2.2.1.2 Effects of deck piles – BLF not in use

In order to investigate the effect of the BLF deck piles, the changes to peak velocities, wave energy and bed shear stress were compared to the baseline (no BLF) conditions (BEEMS Technical Report TR481). Modelling shows that the two end deck piles combined with the fender piles (worst case) slightly interrupt the shore parallel tidal flow, with a small decrease in the currents in the lee of the piles up to a maximum distance of 45 m (Figure 37). Closer to shore the effects lessen, due to the lower current speeds in shallower water.

Figure 38 shows that the observed changes in wave energy due to the BLF piles are small, with a maximum increase of 20% and a maximum decrease of 17%, over a very small area. The area of seabed where the magnitude of change in wave energy is greater than five percent (±5%) corresponds to approximately 1,130 m^2 (or 0.1 ha) over a distance of 65 m. The total longshore impact (any modelled difference) was slightly larger at approximately 115 m. Most changes are to the north of the structure due to the ebb tidal conditions and the south-easterly waves being considered. It is important to note that the change in wave energy *per se* is not relevant to geomorphology or ecology, however the bed shear stress induced by those waves is.

Figure 39 shows the percentage change in combined wave and current bed shear stress, which was calculated as the bed shear in excess of the critical threshold for sediment entrainment. Although the patterns and magnitudes of the percentage change in bed shear stress are broadly similar to those for wave energy, the effect on bed shear stress is generally very small, with the highest changes (± 14%) occurring amongst the deck piles, where the deck piles/fenders are closer and interact with each other. Immediately adjacent to the piles a significant reduction (up to 60%) in the bed shear stress is observed due to wave shadowing (a decrease in the near bed orbital velocity) and scour. However, that reduction is very localized and extends only for 2.4 m from the piles, corresponding to the maximum horizontal scour (see Table 5). The area of seabed where the magnitude of change in bed shear stress greater than five percent (±5%) corresponds to approximately 937 m^2 over a distance of 60 m.

Results shown in Figure 37, Figure 38 and Figure 39 represent the areas of change due to the worst single case (percentage change) considered: south-easterly waves combined with peak ebb tidal currents. To encompass the full extent of potential change required consideration of the tide flowing in both directions, and the two primary wave directions. The total extent of change was determined by taking the spatial union of the 5% change in bed shear stress for all combinations of ebb currents, flood currents, NE waves and SE waves. The total area of change in bed shear stress greater than 5 percent (± 5%) of 0.14 ha is shown in Figure 40. This area is exclusively outside of the Minsmere-Walberswick SPA frontage. There is no impact on the Leiston-Aldeburgh SSSI to the south of the proposed development.

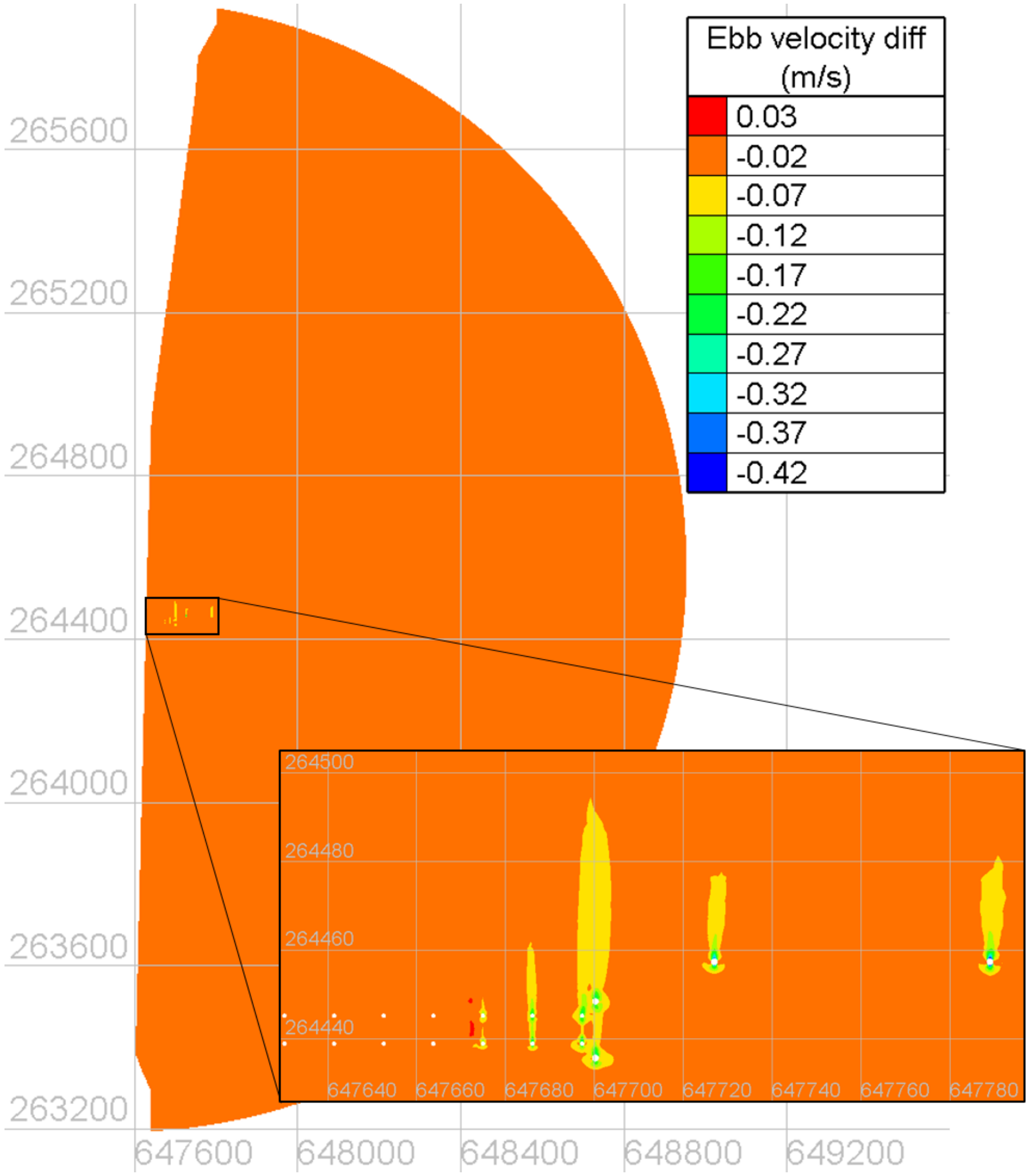

Figure 37: Difference in peak ebb velocity from background due to the presence of the BLF not in use (BEEMS Technical Report TR481).

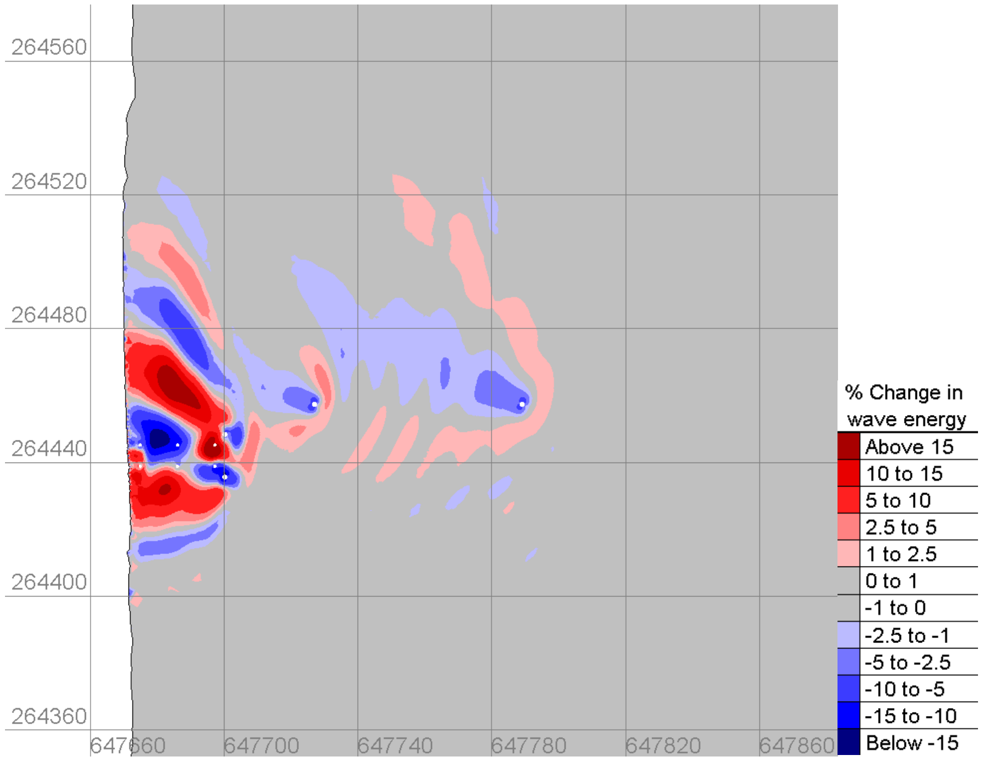

Figure 38: Percentage change in wave energy from background due to the presence of the BLF not in use, with SE waves (BEEMS Technical Report TR481).

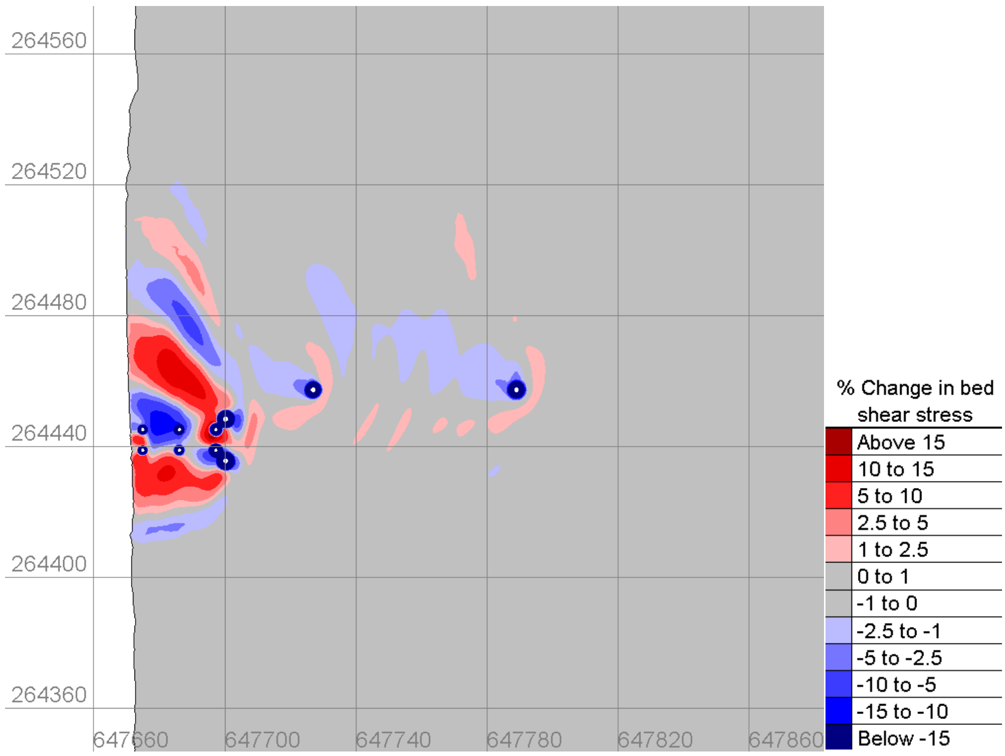

Figure 39: Percentage change in combined wave and current bed shear stress from background due to the presence of the BLF not in use, with SE waves and peak ebb tidal currents (BEEMS Technical Report TR481).

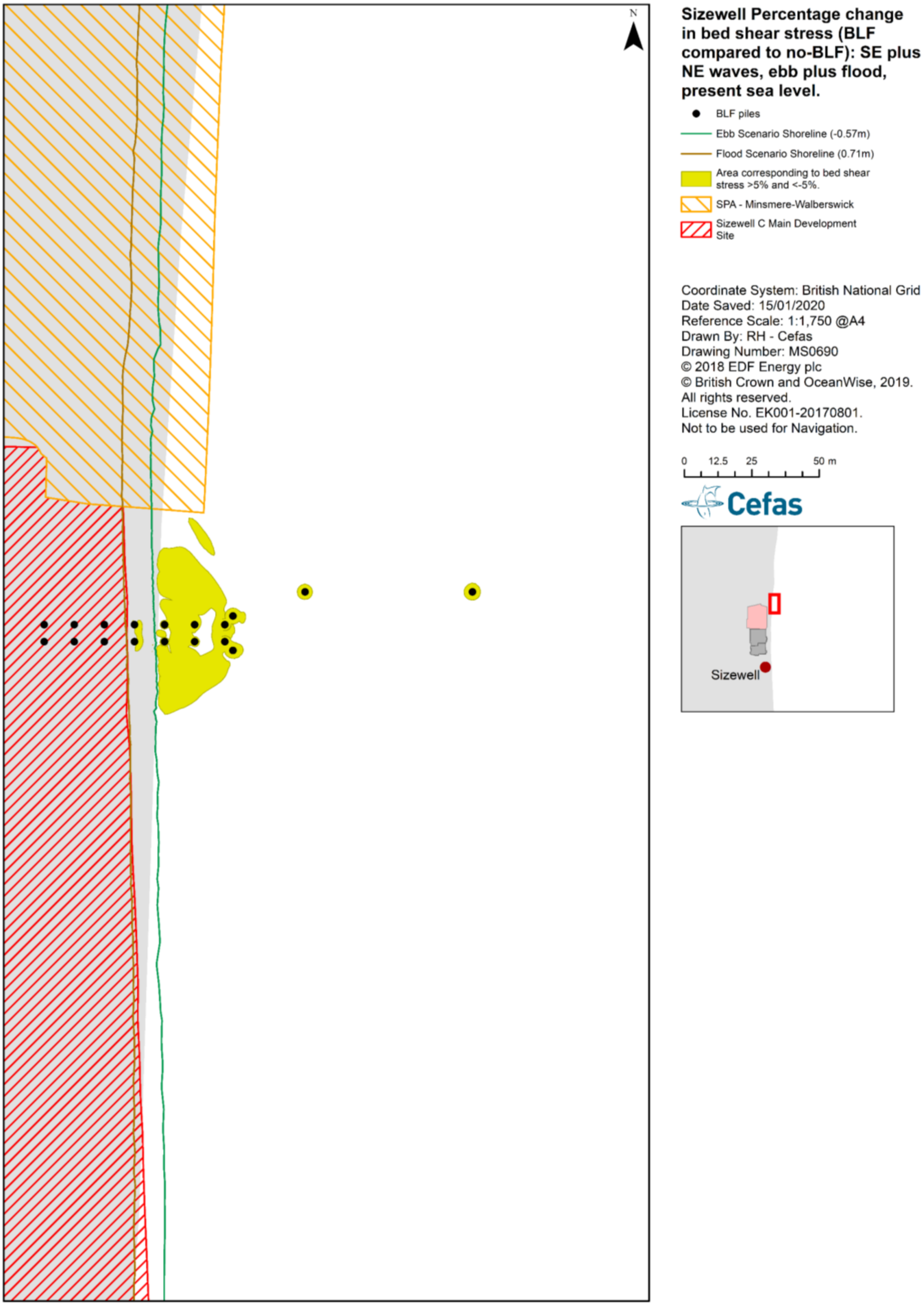

Figure 40: The total area corresponding to a magnitude of change greater than 5 percent (± 5%) for the BLF not in use compared to no BLF, for both wave and tidal current directions (BEEMS Technical Report TR481).

4.2.2.1.3 BLF in-use (piles and reprofiled bed)

To investigate the influence of the reprofiled (dredged) bathymetry for the BLF approach and the grounding pocket, the bathymetry of the model was lowered as per the schematic in Figure 33. The greatest impact dredging depth was on the inner flank of the outer longshore bar (at the most seaward dolphin) where depths would be increased by up to 1.9 m in order to accommodate the stern of a grounded barge, as shown by the yellow – red area in Figure 41. The greatest change to the inner bar (near the fenders) was a maximum of 0.7 m.

To investigate the effect of the dredged bathymetry in combination with the deck piles (i.e., BLF in use) the change in peak velocities, wave energy and bed shear stress was compared to the no BLF conditions. Modelling shows that the reprofiled channel would cause a local reduction in velocities (up to 0.11 m/s) within the confines of the dredging (see Figure 42). When comparing the differences in peak ebb velocities for the BLF not in use, and for the BLF in use, with the baseline (no BLF) conditions (Figure 37 and Figure 42), the magnitude of change due to dredging is smaller than the reduction caused by the BLF alone. However, the spatial extent of change is greater (355 m of frontage compared to 60 m).

The peak increase in wave energy is approximately 150%, although this is for a very small area of around 0.05 ha (see Figure 43). The peak decrease in wave energy is 52% and is observed around the first mooring dolphin. The area of seabed where the magnitude of change in wave energy is greater than five percent (±5%) corresponds to about 1.88 ha over a frontage of 400 m. These results show that dredging works (BLF in use) would have a greater impact on wave energy, both in spatial extent and magnitude, compared to when the BLF is not in use (no dredging) (see Figure 38). However, as noted previously, the change in wave energy *per se* is not relevant to geomorphology or ecology, but the bed shear stress induced by those waves is.

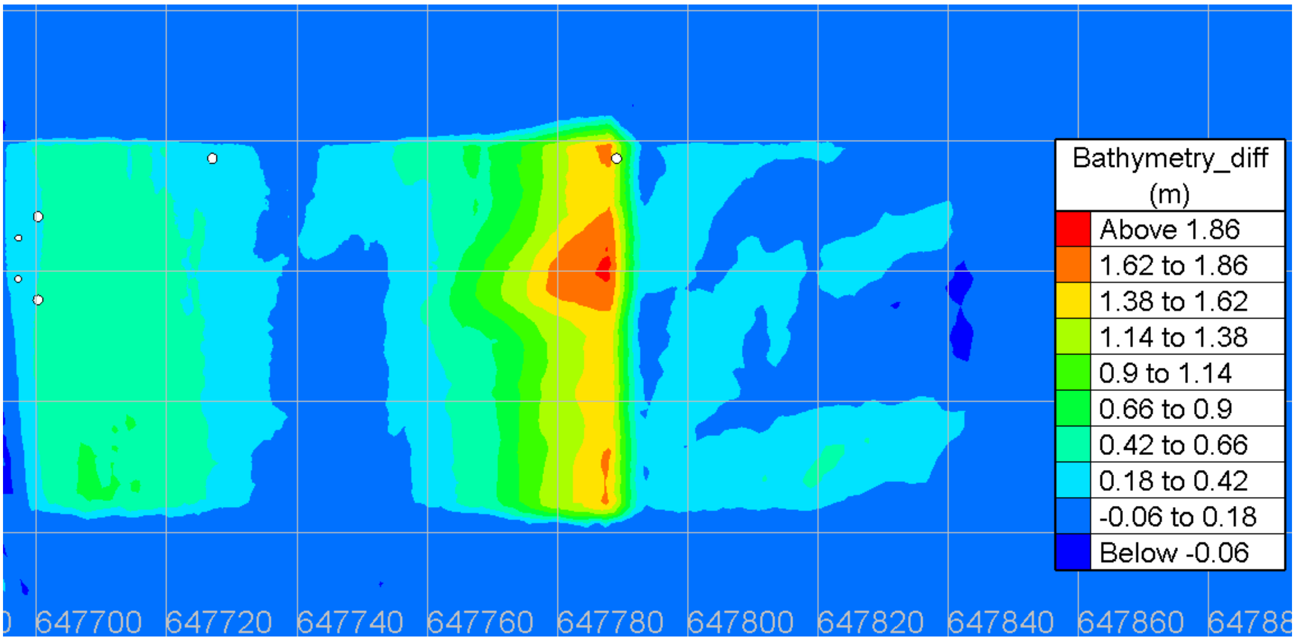

Figure 41: Bathymetric differences due to dredging (BEEMS Technical Report TR481).

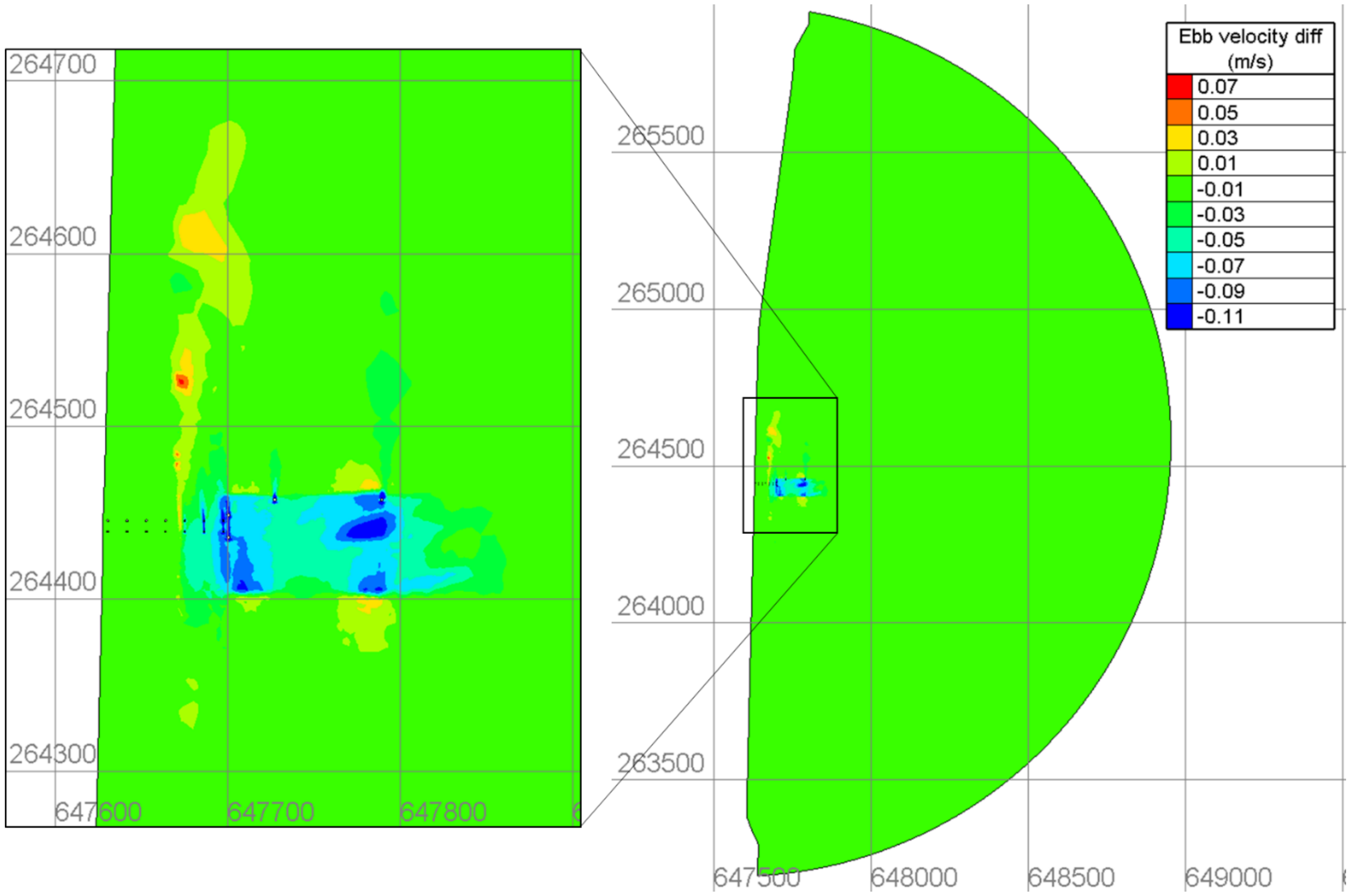

Figure 42: Peak ebb velocity difference due to the BLF in use compared to no BLF (BEEMS Technical Report TR481).

Modelling results show that when the BLF is in use there would be a higher impact on combined wave and currents bed shear stress, in both spatial extent and magnitude, in comparison to the no BLF case (see Figure 44). Reductions in bed shear stress can be seen up to 50 m offshore of the last dolphin pile, due to the dredging of the outer bar there. Maximum increases, of approximately 80%, were obtained near the two most offshore pairs of the deck piles, with the maximum decreases (approximately 47%) at the dolphins, in the dredged region. The percentage of change in bed shear stress is also larger at the north and south sides of the dredged regions. The area of seabed where the magnitude of change in bed shear stress is greater than five percent (±5%) corresponds to 2.1 ha over 410 m of frontage.

Results shown in Figure 42, Figure 43 and Figure 44 represent the areas of change due to the worst single case (percentage change in bed shear stress) considered: south-easterly waves combined with peak ebb tidal currents. To encompass the full extent of potential change requires consideration of the tide flowing in both directions, and the two primary wave directions. The total extent of change for the BLF in use was determined by taking the spatial union of the 5% change in bed shear stress for all combinations of ebb currents, flood currents, NE waves and SE waves (Figure 45). The total area of change in bed shear stress greater than 5 percent (± 5%) is 3.4 ha.

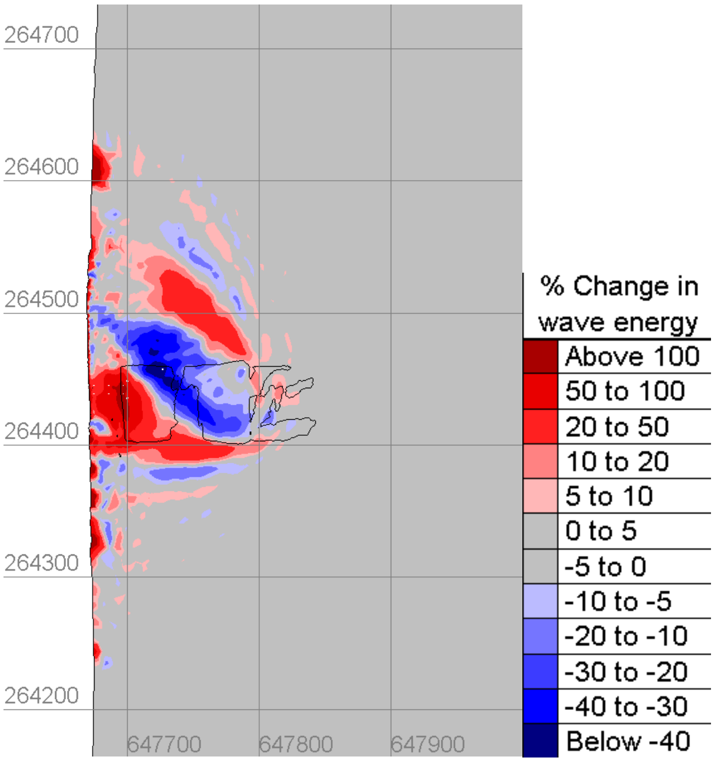

Figure 43: Percentage change in wave energy due to the BLF in use compared to no BLF. The black isoline corresponds to change in bathymetry due to dredging (BEEMS Technical Report TR481).

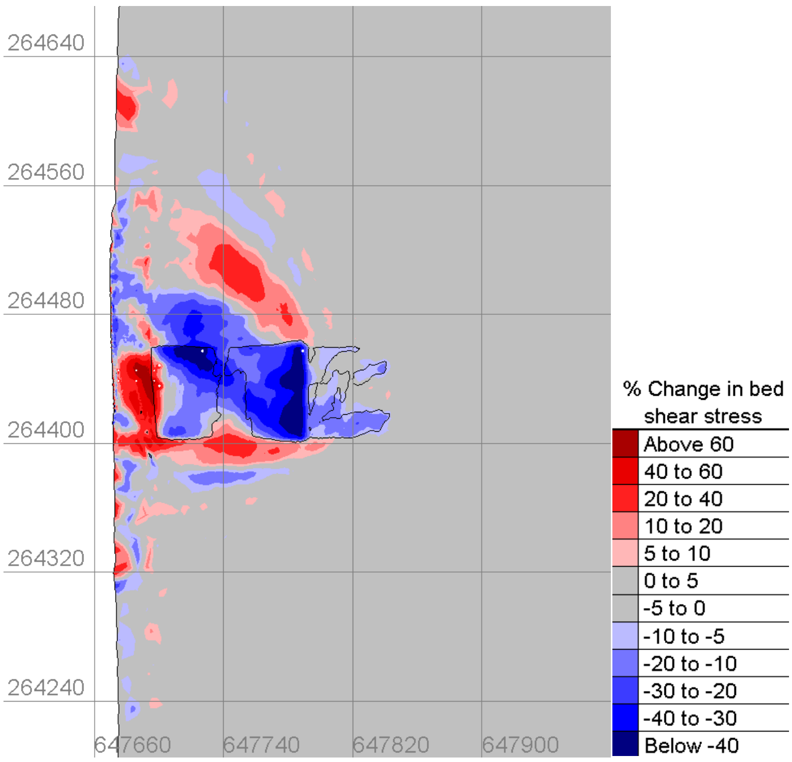

Figure 44: Percentage change in combined wave and current bed shear stress due to the BLF in use compared to no BLF. The black isoline corresponds to change in bathymetry due to dredging (BEEMS Technical Report TR481).

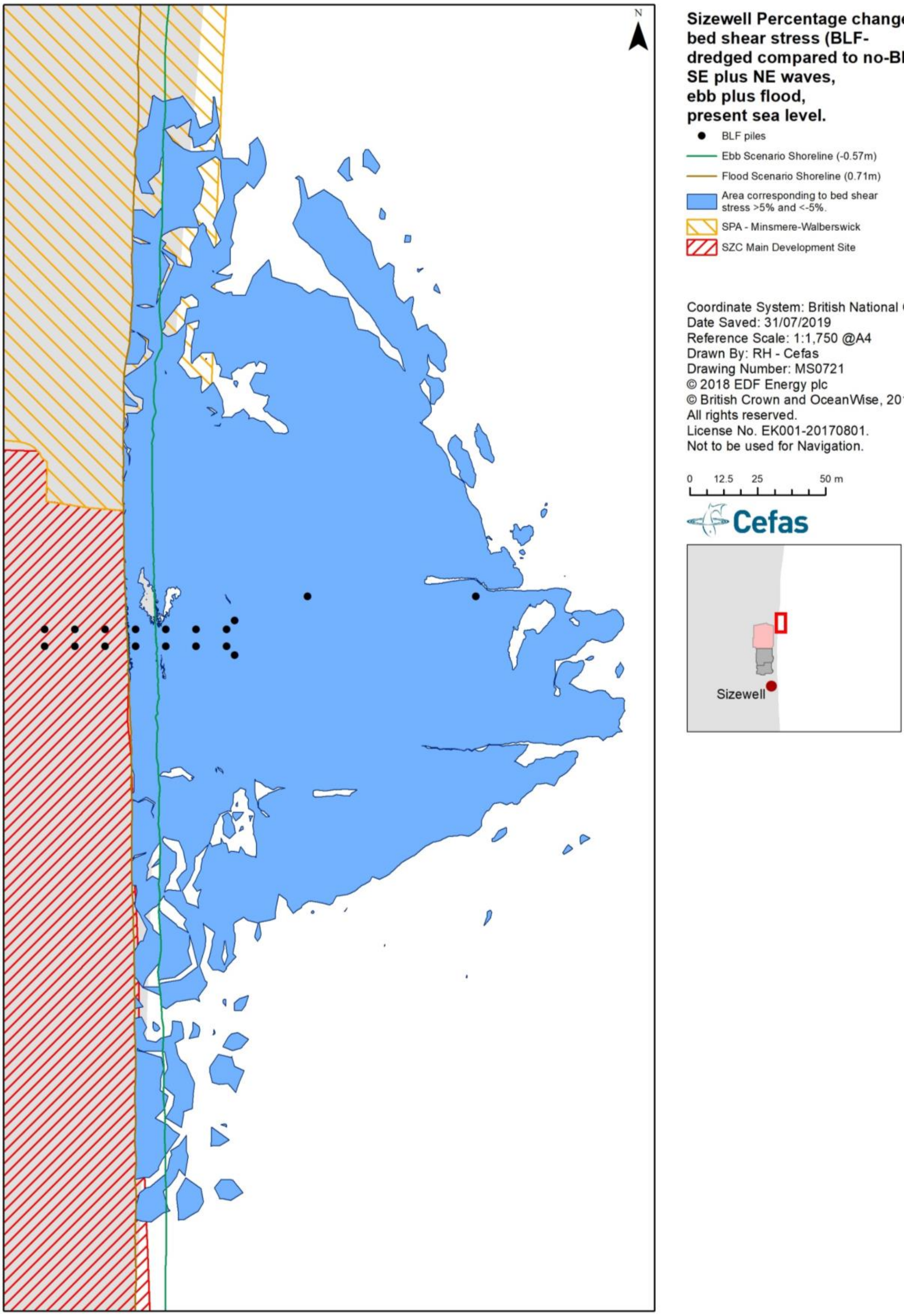

Figure 45: The total area corresponding to a magnitude of change greater than ± 5% for the BLF in use compared to no BLF, for both wave and tidal current directions (BEEMS Technical Report TR481).

Although the area of change in bed shear stress extends to the SPA/SAC frontage, approximately 125 m (Figure 45), the magnitude of change is very small (1-2 N/m^2) compared to the maximum background bed shear stress. Whilst there is a small area (125 m of SPA frontage) with an approximately 35% increase in bed shear stress, the value of the maximum bed shear stress at this point is 33 N/m^2, which compares to typical baseline values of along the inner bar of between 25-55 N/m^2 – that is, the bed shear stress remains within its natural envelope. Most of the change in the SPA (100m of the 125m) is a reduction in bed shear stress. These conditions would vary spatially with the state of tide and waves, meaning the peak impact is not persistent over a tidal cycle and therefore would have no effect on the shoreline, the supra-tidal annual vegetation of drift lines habitat (not present since 2010-2011) (Minsmere to Walberswick Heaths and Marshes SAC) or the potential nesting sites for little tern (*Sterna albifrons*) (Minsmere to Walberswick SPA). As dredging would not take place during poor sea conditions and elevated water levels, supra-tidal shingle is unlikely to be affected; further, in the very unlikely circumstance that storm surge, waves and a reprofiled bed did coincide, the minor changes in bed shear stress would not alter the supra-tidal shingle in any way that was not already naturally occurring. The BLF piles are transmissive and not expected to block sediment transport, however localised scour is predicted. Results show that the piles cause a patch work of increased and decreased bed shear stress depending on the state of the tide and direction of waves. Patches of altered bed shear stress are sufficiently small in duration (of a storm), magnitude and scale that they are not expected to cause detectable change to the shoreline. There would be no impact on the Leiston-Aldeburgh SSSI to the south of the proposed development.

4.2.2.1.4 BLF effects under higher sea level

Once Sizewell C is operational, the BLF would be used infrequently, every 5-10 years, for maintenance activities. The effects of the BLF under rising sea level was investigated using climate change scenario RCP4.5 (UKCP18) and two time periods: 2055 with a predicted sea level rise (SLR) of 0.398 m and 2085 with a predicted SLR of 0.689 m. 2055 is approximately 25 years from the start of Sizewell C operations and 2085 is near the expected end of operations.

SLR was applied to the Sizewell regional model to provide new boundary conditions for the smaller high-resolution mesh around the BLF. The changes to the tidal velocities at the high-resolution mesh boundary were minor with a difference of 2 cm/s for the 2085 SLR case (smaller for the 2055 case). Additionally, a conservative 10% increase in offshore wave height was considered in TOMAWAC simulations (BEEMS Technical Report TR319); this is a conservative approach as the UKCP18 predictions for Sizewell show a decrease in mean and maximum significant wave heights (Section 2.4.2).

The effect of the BLF not in use decreases as SLR increases (Figure 46). Whilst the pattern of change is similar between the present day and SLR, both the magnitude of change and spatial extent are reduced. Under SLR the BLF in use shows a very similar pattern and extent (Figure 47). Whilst the areas of peak change have reduced, the total area of change is slightly smaller for SLR=0.398 m and slightly larger for SLR=0.689 m but similar to the present-day conditions. This is due to non-linear combination of wave and current bed shear stress, which change subtly with the increase in water depth. The reduction in currents (1-2 cm/s) will be counteracted by the slight increase in wave height (10-14 cm). The longshore extent of change does not change with SLR. Table 6 summarises the area of percentage change greater than ±5% for both the BLF in use and not in use under SLR.

The area with a percentage change greater than ±5% when the BLF is not in use (Figure 46) does not intersect with the SPA/SAC frontage, whereas, the BLF in use does intersect. Although the area of change in bed shear stress extends to the SPA/SAC frontage, approximately 170 m (Figure 47), the majority is a reduction in bed shear stress. Whilst there is a small area (70 m of SPA frontage) with an increase in bed shear stress, the value of the maximum bed shear stress at this point is 49 N/m^2, which compares to typical baseline values of along the inner bar of between 25-55 N/m^2 – that is, the bed shear stress remains within its natural envelope. As shown with the BLF in use under present-day conditions (Section 4.2.2.1.3), results show that the piles cause a patch work of increased and decreased bed shear stress depending on the state of the tide and direction of waves. Patches of altered bed shear stress are sufficiently small in duration (of a storm), magnitude and scale that they are not expected to cause detectable change to the shoreline. There would be no impact on the Leiston-Aldeburgh SSSI to the south of the proposed development.

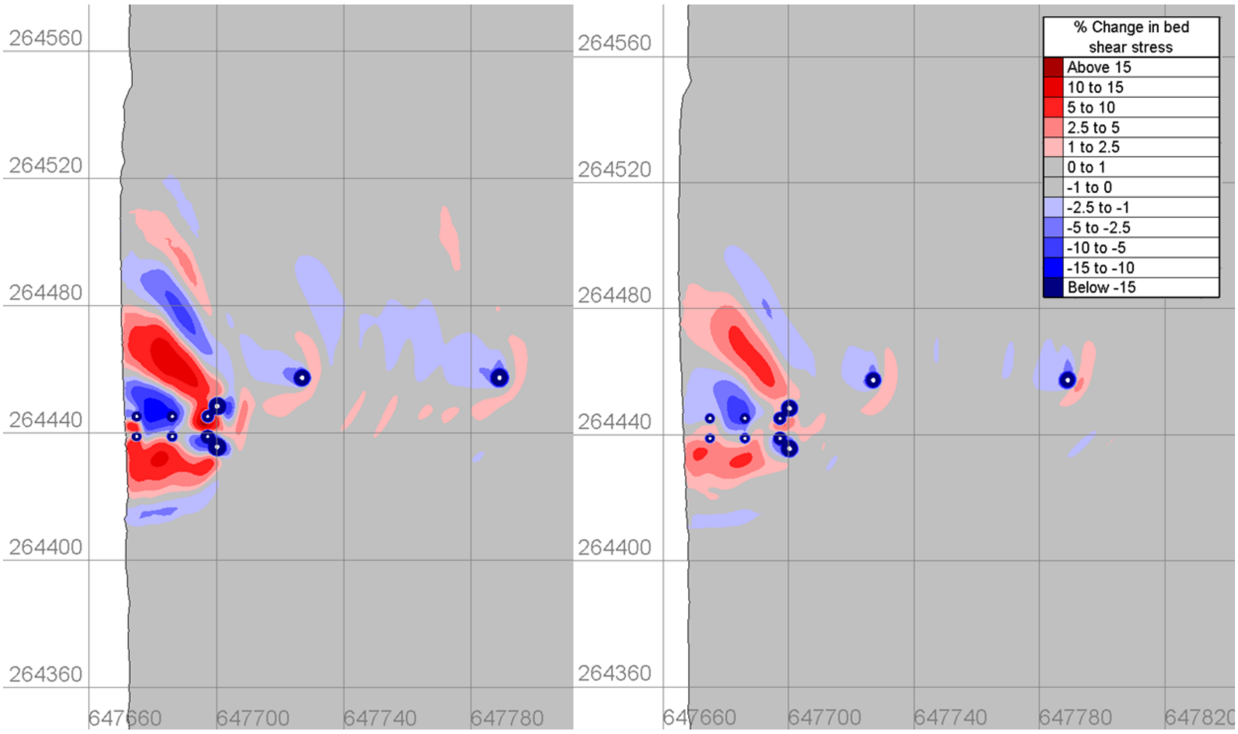

Figure 46: Percentage change in bed shear stress (BLF not in use compared to no BLF) with present sea levels (left) and SLR = 0.689 m (right) (BEEMS Technical Report TR481).

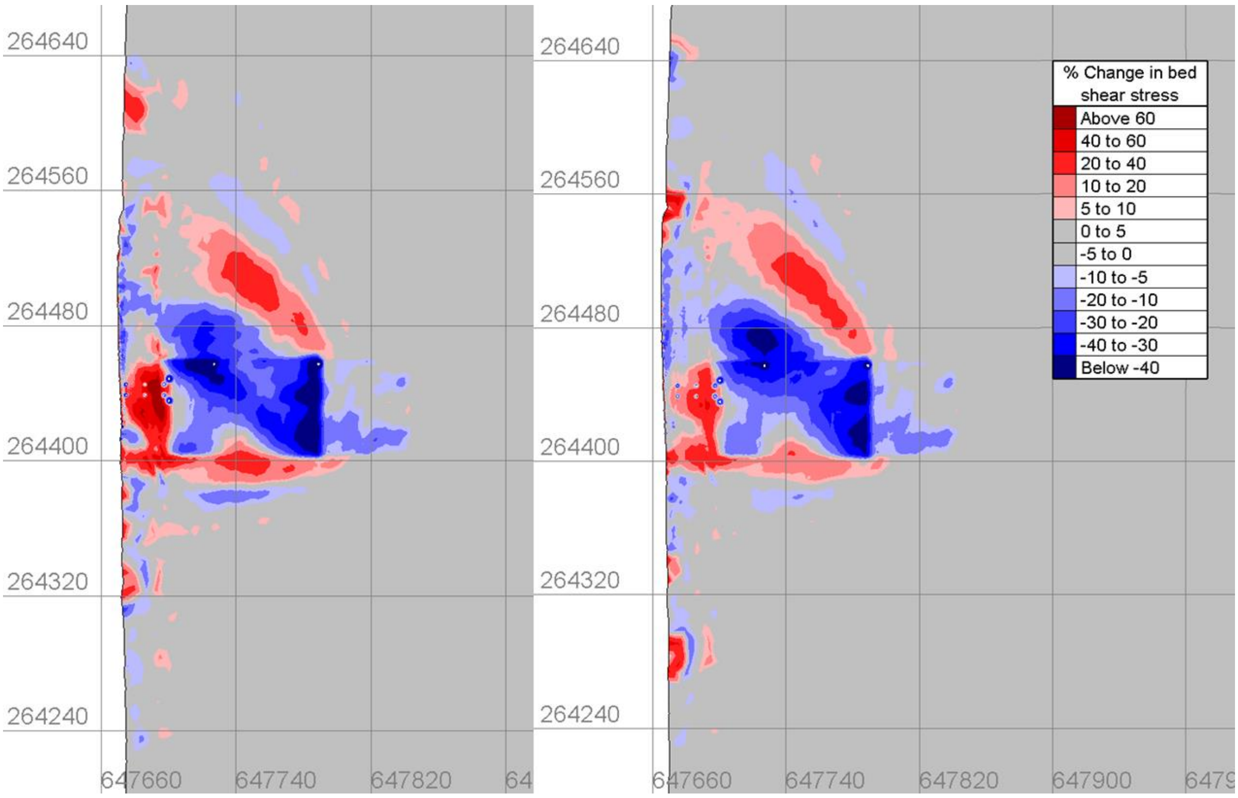

Figure 47: Percentage change in bed shear stress (BLF in use compared to no BLF) with present sea levels (left) and SLR = 0.689 m (right) (BEEMS Technical Report TR481).

Table 6: Areas (m^2) correspondent to the percentage change in bed shear stress greater than 5% considering sea level rise.

BLF not in use – no BLF Area greater than ± 5% (m^2)			**BLF in use – no BLF Area greater than ± 5% (m^2)**		
SLR = 0 m	SLR = 0.398 m	SLR = 0.689 m	SLR = 0 m	SLR = 0.398 m	SLR = 0.689 m
937.2	443.6	349.0	20,979	20,355	21,601

4.2.2.1.5 BLF effects under migrating beach and bars

As the beach profile and longshore bars are dynamic features and can move over storm, seasonal and longer timescales, the influence of the BLF has been tested for a landward profile translation of 20 m. This degree of translation is likely to occur over time periods of decades and linked shoreline retreat. To investigate this, the models used the same mesh structure as the present-day scenarios, but with the bathymetry translated 20 m to west (i.e. a landward migrated profile) and a new dredging profile created to meet navigation and docking needs. As a conservative measure, sea level rise was not considered – SLR would have meant less dredging on the outer bar due to deeper water and raised hull clearance. As a result of the migrated profile, whilst a slightly deeper cut (1.92 m compared to 1.88 m) would be required in the outer bar, the inner bar would no longer require dredging and less of the outer bar would be required to be dredged for navigational clearance resulting in a reduction in the overall dredge volume of 58% or 2,665 m^3 for the full capital dredge.

As with the SLR case, the effect of the BLF not in use reduces as the shoreline retreats (Section 4.2.2.1.4). However, due to an increase in the area of the outer bar that would be reprofiled, the area with a percentage change greater than ±5% when the BLF is in use increases by approximately 25% (Figure 49). This gives the worst-case scenario for the BLF in use condition, whereas the present-day conditions provide the worst case for the BLF not in use (Figure 48). Table 7 summarises the area of percentage change greater than ±5% for both the BLF in use and not in use.

The area with a percentage change greater than ±5% when the BLF is not in use (Figure 48) does not intersect with the SPA/SAC frontage, whereas, the BLF in use does intersect. Although the area of change in bed shear stress extends to the SPA/SAC frontage, approximately 185 m (Figure 49), the majority is a reduction in bed shear stress. Whilst there is a small area (75 m of SPA frontage) the value of the maximum bed shear stress at this point is 44 N/m^2, which compares to typical baseline values of along the inner bar of between 25-55 N/m^2 – that is, the bed shear stress remains within its natural envelope. As shown with the BLF in use under present-day conditions (Section 4.2.2.1.3), results show that the piles cause a patch work of increased and decreased bed shear stress depending on the state of the tide and direction of waves. Patches of altered bed shear stress are sufficiently small in duration (of a storm), magnitude and scale that they are not expected to cause detectable change to the shoreline. There would be no impact on the Leiston-Aldeburgh SSSI to the south of the proposed development.

Whilst the effect of BLF in use increases with shoreline retreat and remains similar with SLR. The BLF not in use will be the persistent for its lifetime but will reduce with time due to SLR.

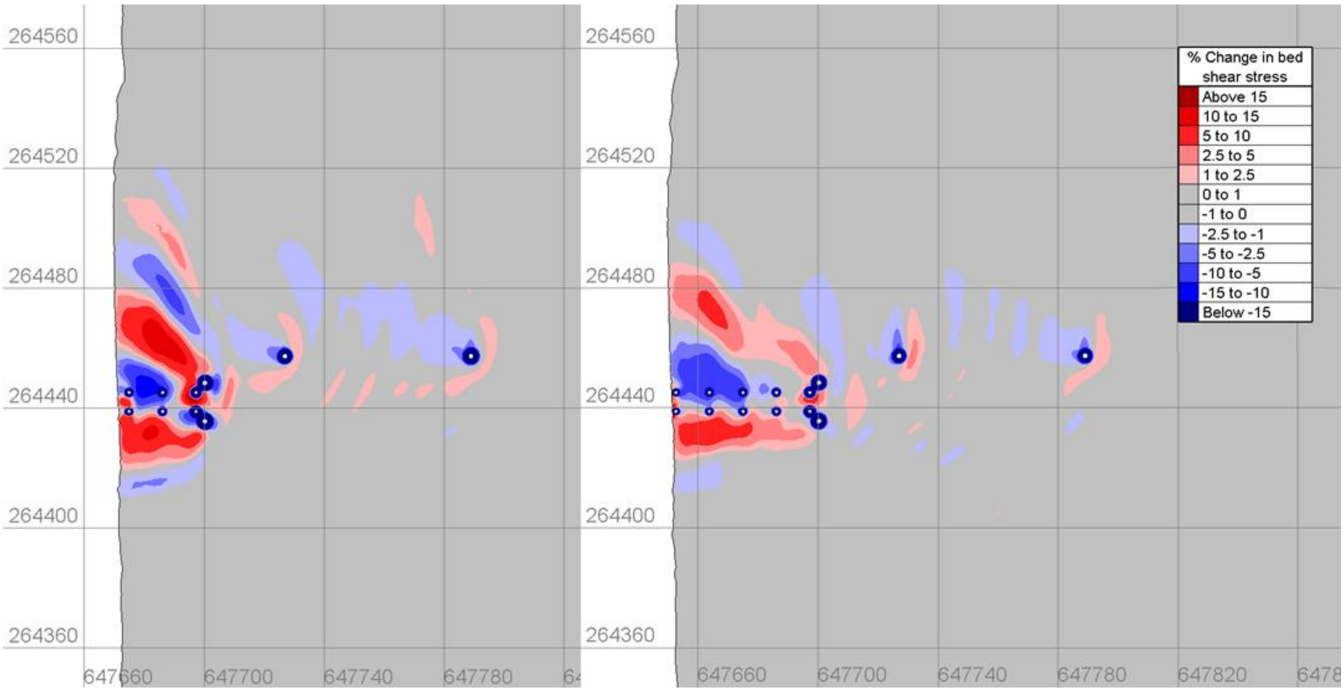

Figure 48: Percentage change in bed shear stress (BLF not in use compared to no BLF) with present shoreline (left) and 20 m shoreline and bar migration (right) (BEEMS Technical Report TR481).

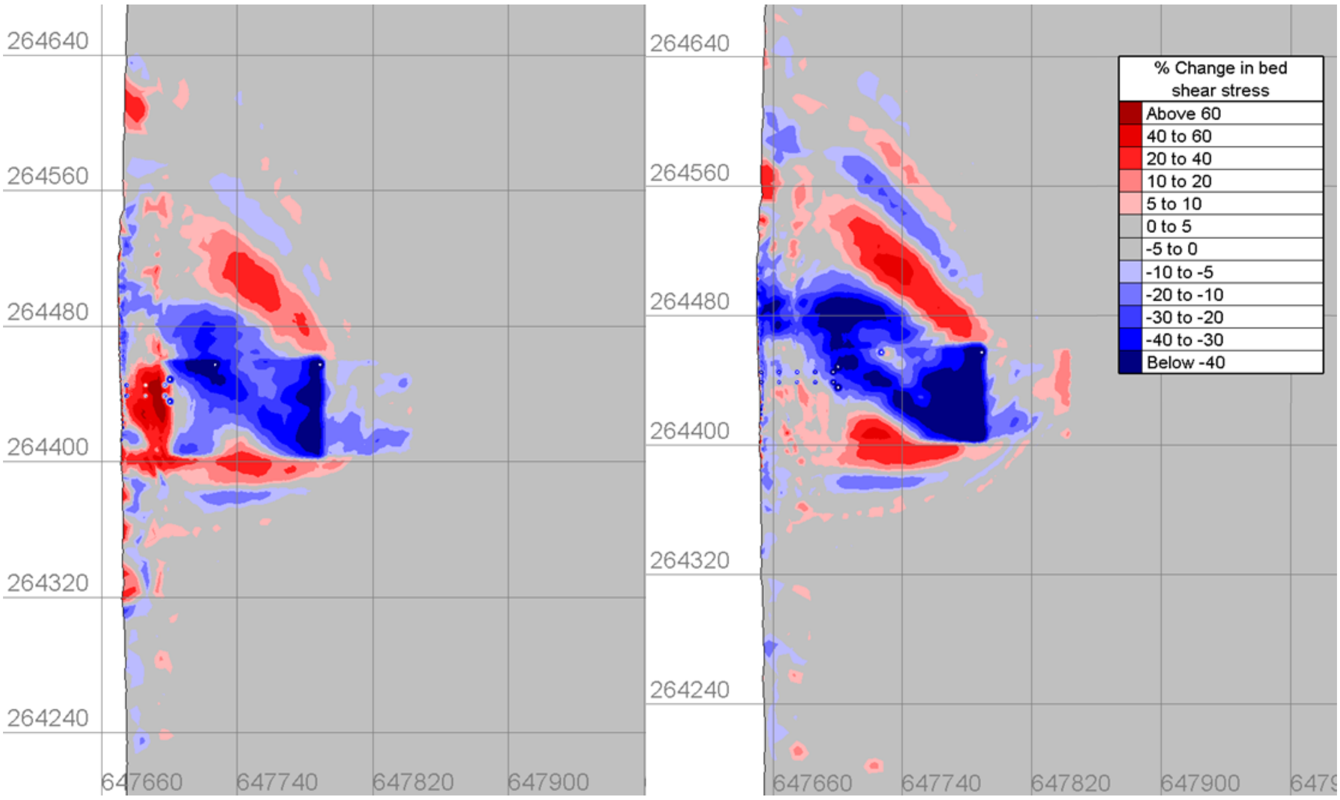

Figure 49: Percentage change in bed shear stress (BLF in use compared to no BLF) with present shoreline (left) and 20 m shoreline and bar migration (right) (BEEMS Technical Report TR481).

Table 7: Areas (m²) correspondent to the percentage change in bed shear stress greater than 5% considering shoreline retreat.

BLF not in use – no BLF Area greater than ± 5% (m²)		**BLF in use – no BLF Area greater than ± 5% (m²)**	
Present	20 m Beach and Bar Migration	Present	20 m Beach and Bar Migration
937.2	663.2	20,979	26,513

4.2.2.2 *Nearshore dredging for the BLF approach*

The validated Delft3D model of Sizewell (BEEMS Technical Report TR132) with its accompanying particle tracking module was used to estimate the re-distribution of sediment, resulting from a range of potential dredge and sediment disposal scenarios planned for the construction and operation of Sizewell C (BEEMS Technical Report TR480). Dredging at the BLF would use a plough dredger, which involves the mechanical agitation of the bed with the ambient flows and moving the suspended sediment away from the dredge area. Sediment is not removed from system, meaning that no supply is lost from the local sediment pathways. Vibrocore survey data from the vicinity of the proposed BLF approach (Figure 59, VC30) indicate fine to medium sands (top metre) overlaying a metre of coarse to medium sand and a subsequent layer of silty to medium sand (BEEMS Technical Report TR480). A sediment grain size of 210 µm was used to represent the fine to medium sand in the model.

The total area dredged would be 9,068 m², or 0.91 ha, as per the profile shown in Figure 33 and Figure 34. The total dredge volume would be 4,600 m³. A single pass by the plough was assumed to be 0.1 m deep. However, a large proportion of this material is likely to resettle within the BLF dredge area on each pass. Assuming that 10% of the material would be moved outside of the dredge area in suspension each pass, 6.2 m³ would be released per 1 minute (or 372 m³ per hour). Assuming dredging operations would occur around the clock, the reprofiling would take approximately 2.1 days to complete with 742 cycles of 1 minute of dredging, followed by 3 minutes of transit (BEEMS Technical Report TR480).

The modelling results (Figure 50) show that the area affected by sediment disturbed during dredging of the BLF approach channel would be confined to close inshore, extending north-south along the coast. The maximum momentary SSC values encountered at any time during the model runs are typically around 200 mg/l above background. Concentrations of more than 50 mg/l above background are confined to within 6.5 km to the north and south of the dredge area on spring tides and to within 5 km on neap tides.

Following the completion of the dredge, the plume quickly disperses. On spring tides, sediment in suspension would be at concentrations of less than 20 mg/l above background within three days. On neap tides, the plume concentrations in suspension also quickly return to values which are close to background, however some resuspension of material would be expected once the larger range spring tides occur. The model predicts that the resuspension of deposited dredge material could result in SSC greater than 100 mg/l in some localised areas. However, these are expected to be short lived (of the order of hours) and are unlikely to be discernible from the background SSC.

On both spring and neap tides, the sediment only settles on the bed over a relatively small area close inshore. Away from the immediate dredge area, the maximum sediment deposition thickness is approximately 20 mm, with typical values less than 2 mm. These limited deposits may be discernible on the beach at certain locations (e.g. thin sand veneers of less than 3 mm thick along the Sizewell frontage, less than 1 mm along the Minsmere frontage and less than 0.5 mm thick at Thorpeness), however sand veneers are natural and it would not be possible to distinguish the two. There is no impact on the Leiston-Aldeburgh SSSI. Following the dredging of the BLF, a relatively large proportion of material would remain on the seabed (approximately 25% on neap tides and 30% on spring tides). This material would be deposited in shallow inshore areas where slow tidal flows are not sufficient to re-erode material deposited on the seabed. This material is likely to be detectable on the intertidal beach but would be redistributed and mixed with beach sediment under wave activity.

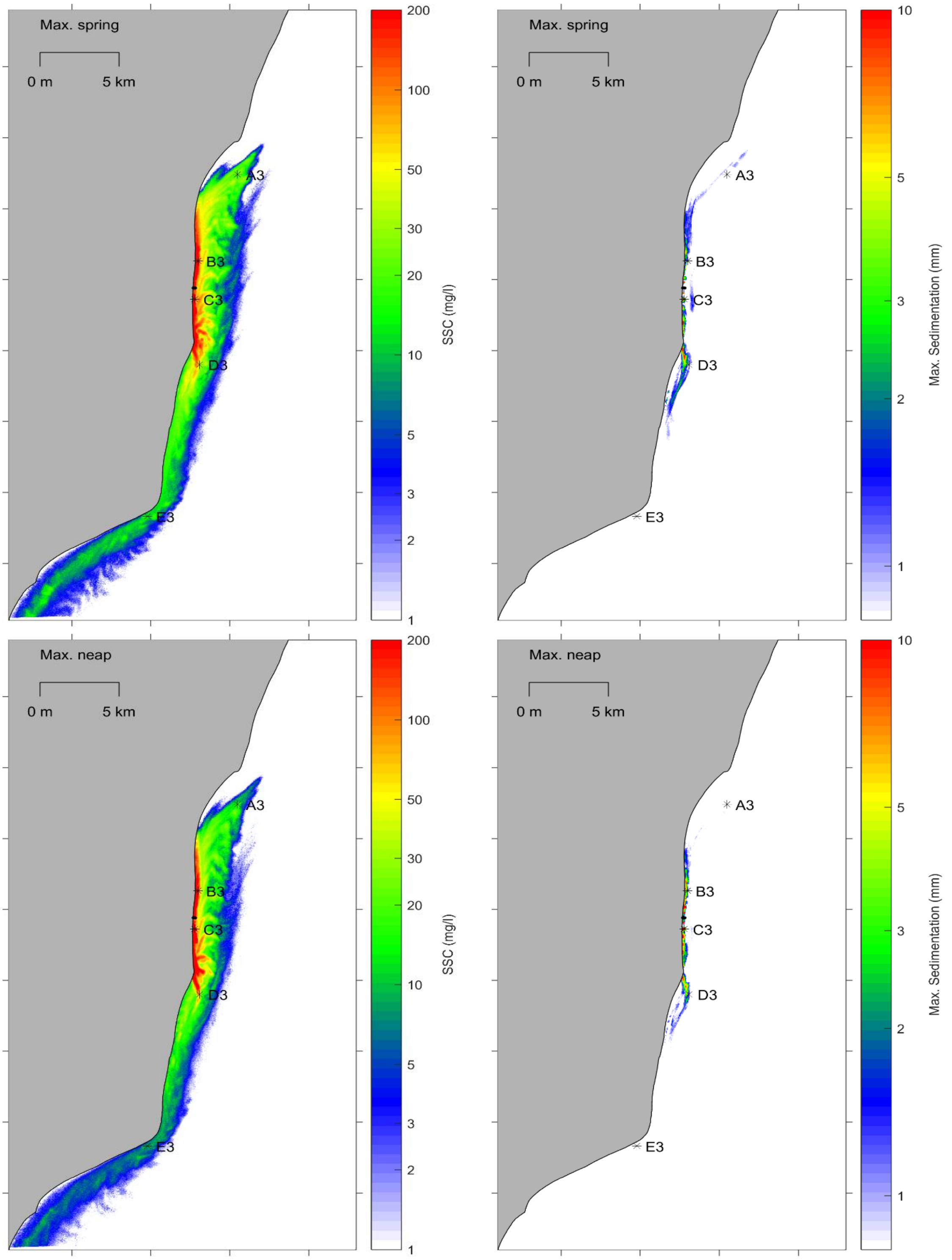

Figure 50: Maximum momentary SSC (left) and sediment deposition depth (right) on spring (top) and neap (bottom) tides during dredging of the BLF approach channel (at any moment in model run-time). (Source: BEEMS Technical Report TR480).

Instantaneous values provide no indication of the duration of either the periods of settling on the bed or the plume in suspension. To provide some indication of how long the conditions prevail, the plume areas in suspension (in the surface layer of the model) and on the bed have been calculated for different threshold values and durations (Table 8 and Table 9).

Table 8: Duration and spatial extent of the SSC plume in the surface layer of the model resulting from excavation of the BLF approach channel.

SSC > mg/l	Duration (>= hours)	Area (ha)	
		Neap Tides	Spring Tides
50	1	1660	2350
50	3	62	172
50	6	0	0
50	12	0	0
100	1	551	866
100	3	13	43
100	6	0	0
100	12	0	0
300	1	141	159
300	3	0	2
300	6	0	0
300	12	0	0

Table 9: Duration and spatial extent of deposited sediment thickness resulting from excavation of the BLF approach channel.

Sediment Thickness > mm	Duration (>= hours)	Area (ha)	
		Neap Tides	Spring Tides
20	1	6	5
20	3	6	5
20	6	6	4
20	12	6	3
50	1	3	2
50	3	3	1
50	6	3	1
50	12	3	0
300	1	1	0
300	3	1	0
300	6	1	0
300	12	1	0

4.2.2.3 *BLF infilling rates*

The BLF access channel would require routine maintenance to allow continual safe navigational clearance over the outer longshore bar and a clear grounding pocket, when the BLF is in use. The maintenance dredging would be performed by a plough dredger. As plough dredging does not remove sediment from the system, there is no supply lost from the local sediment pathways and the dredged area is likely to naturally infill. Estimates of the predicted infilling due to tidal currents and waves were calculated for the two portions of the access channel: the grounding pocket and the outer bar (BEEMS Technical Report TR487). The deepest part of the grounding pocket is 1.88 m below the current (2017) bathymetry profile. The average thickness of the outer bar from the top surface to the depth clipped for navigational clearance is 0.2 m.

Estimated average infilling rates and required number of maintenance dredges were based upon 11 years of offshore hindcast wave data (2001-2012; Met Office 'ReMap' European wave model). The wave climate at the BLF was generated by transforming the offshore wave field to the BLF site using the TOMAWAC wave model (BEEMS Technical Report TR319). Infilling due to tidal currents was calculated using a tidal time series (currents and water depths) extracted at the model node closest to the BLF access channel; results were from the Sizewell regional TELEMAC2D model updated with new bathymetry collected in 2017, which was run for 1 month (07/11/2013 to 06/12/2013) (BEEMS Technical Report TR481) and consecutively repeated to create a time series matching the length of the 11-years of modelled wave data.

Figure 51 and Figure 52 present the average infill rate at the grounding pocket and outer bar, with the monthly standard deviation (black whiskers). In the nearshore environment of the BLF access channel, the infill rate was dominated by waves, in particular waves with significant wave height greater than > 0.8 m. When waves were lower than this, slower infilling was caused by tides only. There was a strong seasonal cycle in the infill rates, with higher and more frequent infilling under common winter waves compared to the more quiescent summer conditions. Table 10 summarises the average monthly infill rate for the summer (June to August), winter (defined here as November to January) and the total average infill for the likely operating window of the BLF (April to October). While a range of sediment grain sizes was investigated[13] the values in the summary are reflective of a grain size of 350 µm, which is representative of the overall conditions observed at the dredge locations.

As the barge operator has yet to be contracted, it is not yet known how much infilling can occur before the barge is no longer able to use the access channel. Therefore, a range of thresholds, at which the level of infilling would trigger a maintenance dredge, were tested for both the grounding pocket and the outer bar. The estimated number of maintenance dredges was calculated by the dredge depth, starting at its maximum dredge depth and updated at each time step to account for the cumulative infill. Once the cumulative infill rate reached an infilling threshold, the dredge depth was reset to the initial dredge depth. The calculated infilling rates considered all 11 years (2001-2012) of the modelled wave and tidal data and averages were derived for each month so that seasonal trends could be assessed. The three infilling thresholds considered for the grounding pocket, which would instigate maintenance dredging, were 0.25 m, 0.5 m and 0.75 m. The three thresholds tested for the outer bar were 0.05 m, 0.1 m and 0.15 m. Table 11 provides a summary of the number of maintenance dredges required to keep the access channel operational for a range of thresholds and the percentage volume that the maintenance dredge represents from the initial capital dredge volume (4,600 m^3).

[13] Different particle sizes alter the infill rates.

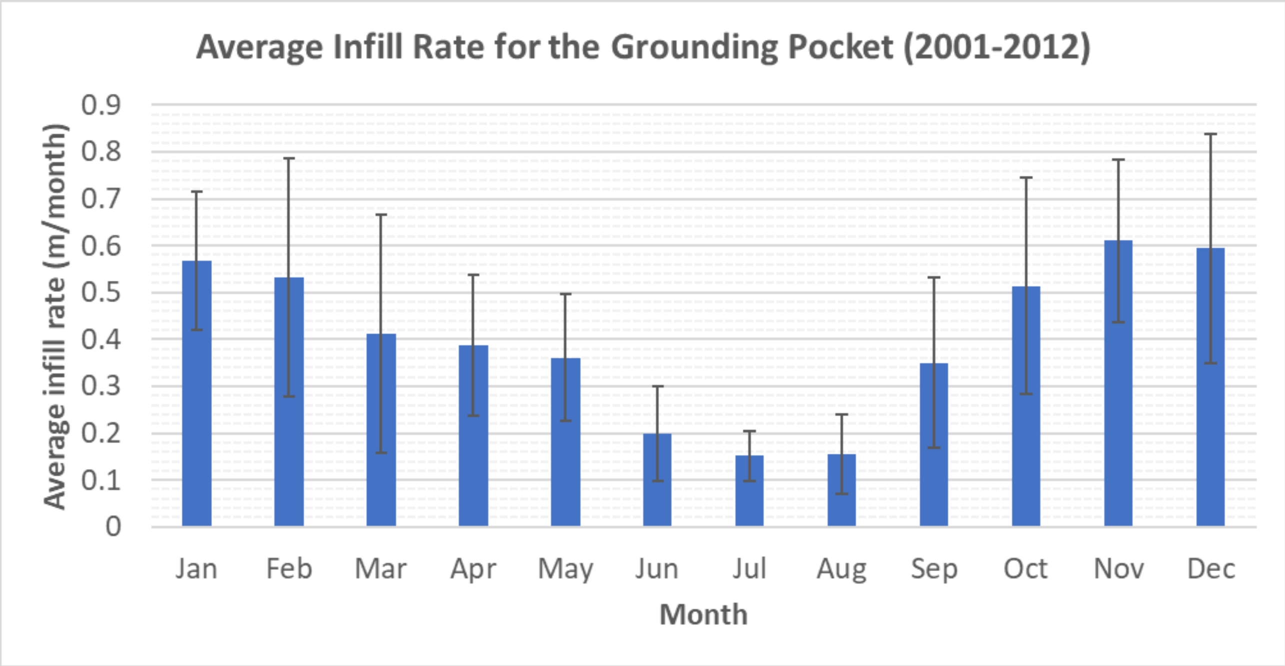

Figure 51: Average monthly infill climatology (m/month) at the grounding pocket using modelled wave data. Whiskers in black indicating inter-annual variability (1 standard deviation).

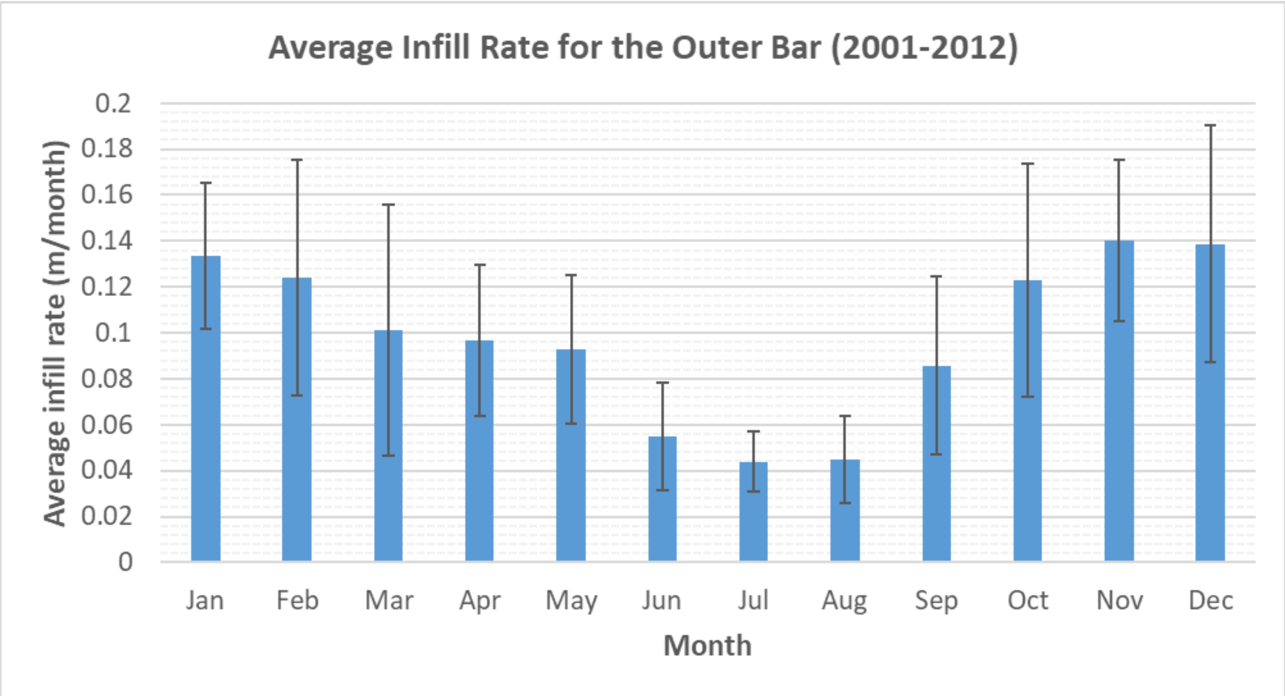

Figure 52: Average infill rate (m/month) at the outer bar using modelled wave data. Whiskers in black indicating inter-annual variability (1 standard deviation). Note the different scale on the y-axis compared to Figure 51.

Table 10: Summary of dredge infill rates for the BLF access channel per seasonal windows.

Seasonal windows	Average monthly infill rate (m per month)		Total infill rate (m)	
	Grounding Pocket	Outer Bar	Grounding Pocket	Outer Bar
Summer (Jun-Aug)	0.168	0.048	0.504	0.143
Winter (Nov-Jan)	0.590	0.137	1.771	0.412
Operational Window (Apr-Oct)	0.302	0.077	2.116	0.542

Table 11: Summary of the required number of maintenance dredges for the BLF access channel for each of the considered infilling thresholds, that would trigger a maintenance dredge.

Conditions	Required number of maintenance dredges											
	Grounding pocket						Outer bar					
	0.25 m		0.5 m		0.75 m		0.05 m		0.1 m		0.15 m	
	Avg.	Max	Avg.	Max	Avg.	Max	Avg.	Max	Avg.	Max	Avg.	Max
Summer (June-August)	2.1	5	1.1	3	0.6	3	2.6	3	1.0	3	0.5	3
Winter (November-January)	6.7	10	3.5	6	2.3	4	7.2	11	3.1	5	1.8	3
Operational window (April-October)	8.6	16	4.3	8	2.7	8	9.5	15	4.2	9	2.0	7
% Volume of initial dredge (4600 m^3)	1		3		12		3		7		10	

As would be expected, the lower the threshold, the more frequently maintenance dredging will be required. Conversely, a more frequent dredging requirement would take less time and remove a smaller volume of material. At the intermediate threshold considered (0.5 m), dredging of the grounding pocket would be required approximately 4.3 times during the 7-month operational window (maximum 8 times) and require the removal of 3% of the initial capital dredge volume per maintenance dredge. On the outer bar the intermediate threshold is 0.1 m, which would require dredging 4.2 times during the 7-month operational window (maximum 9 times) and would require the removal of 7% of the initial capital dredge volume per maintenance dredge.

The final dredge profile and vessel tolerance to infilling are yet to be confirmed. For assessment purposes of the potential effects of dredging in the Environmental Statement, a precautionary stance is adopted based on the predicted infilling rates and the assumed profile outlined above. During the 7-month operational window, assessments will consider a single annual capital dredge event of 4,600 m^3, followed by monthly maintenance dredges of the grounding pocket and outer bar. The maintenance dredging would equate to a total of 10% of the initial capital dredge volume per maintenance dredge.

It should be noted that were deliveries to occur throughout the year, a greater frequency of maintenance dredging would be required as storm events would infill the dredged area more rapidly and could cause complete infilling and the requirement for dredging at the scale of a capital dredge. Furthermore, any opportunistic winter deliveries are likely to require a full capital dredge.

Sea level rise would deepen the BLF access route. As the depths increase, the number of maintenance dredges will reduce to the point such that the depths over the outer bar are sufficient for navigational access without dredging. At this point dredging for access would no longer be needed, reducing the total capital dredge volume and the maintenance dredge volume. Therefore, sea level rise would lead to reduced dredging and so the current assessment represents the worst case.

4.2.2.4 *BLF maintenance dredging*

The same validated Delft3D model of Sizewell (BEEMS Technical report TR132) with its accompanying particle tracking module was used to estimate the re-distribution of sediment, resulting from a maintenance dredge representing 10% of the full capital dredge volume (4,600 m³) (BEEMS Technical Report TR480). Dredging at the BLF would use a plough dredger, which involves the mechanical agitation of the bed with the ambient flows moving the sediment in suspension away from the dredge area. Sediment is not removed from system, meaning that no supply is lost from the local sediment pathways. Vibrocore survey data from the vicinity of the proposed BLF approach (Figure 59, VC30) indicate fine to medium sands (top metre) overlaying a metre of coarse to medium sand and a subsequent layer of silty to medium sand. (BEEMS Technical Report TR480). A sediment grain size of 210 µm was used to represent the fine to medium sand in the model.

A single pass by the plough was assumed to be 0.1 m deep. However, a large proportion of this material is likely to resettle within the BLF dredge area on each pass. Assuming that 10% of the material is moved outside of the dredge area in suspension each pass, 6.2 m³ would be released per 1 minute (or 372 m³ per hour). If the dredging operations were around the clock, the reprofiling would take approximately 5 hours to complete with 74 cycles of 1 minute of dredging, followed by 3 minutes of transit (BEEMS Technical Report TR480 Edition 2).

The modelling results (Figure 53) show that the area affected by sediment released during the maintenance dredging is confined close inshore, with an elongated shape extending north-south along the coast. The maximum momentary SSC values are typically around 200 mg/l above background, any concentrations above this are very localised and short lived. The lateral plume extent with concentrations of more than 50 mg/l above background is confined to within 4.25 km to the north and south of the dredge area on spring tides and to within 4 km on neap tides.

Following the completion of the dredge, any sediment in suspension quickly disperses. On spring tides material in suspension is at concentrations of less than 20 mg/l above background within half a day of the completion of the dredge. On neap tides, the plume concentrations in suspension also quickly return to values which are close to background.

On both spring and neap tides, the sediment only settles on the bed over a relatively small area close inshore. These limited deposits are unlikely to be discernible from the baseline sediments, with no deposits exceeding 10 mm thickness at the end of the model simulation. Overall, the results of this study follow a similar trend to that of the full capital dredge, albeit at a much smaller magnitude.

Instantaneous values provide no indication of the duration of either the periods of settling on the bed or the plume in suspension. To provide some indication of how long the conditions prevail, the plume areas in suspension (in the surface layer of the model) and on the bed have been calculated for different threshold values and durations (Table 12 and Table 13).

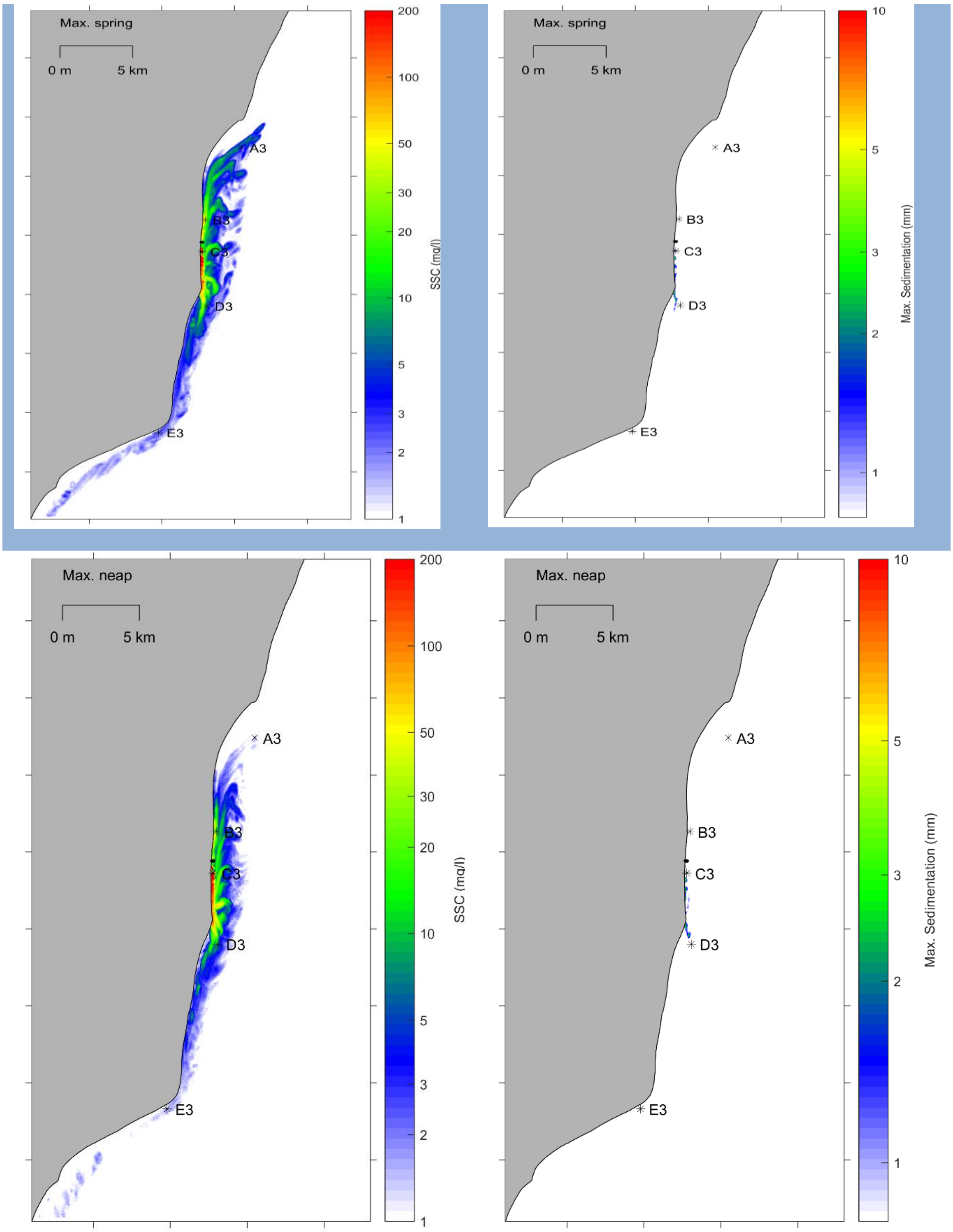

Figure 53: Maximum momentary SSC (left) and sediment deposition depth (right) on spring (top) and neap (bottom) tides during maintenance dredging of the BLF approach channel (at any moment in model run-time). (Source: BEEMS Technical Report TR480 Edition 2).

Table 12: Duration and spatial extent of the SSC plume in the surface layer of the model resulting from the maintenance dredge excavation of the BLF approach channel.

SSC > mg/l	Duration (>= hours)	Area (ha)	
		Neap Tides	Spring Tides
50	1	120	170
50	3	13	6
50	6	0	0
50	12	0	0
100	1	40	56
100	3	3	1
100	6	0	0
100	12	0	0
300	1	11	6
300	3	0	0
300	6	0	0
300	12	0	0

Table 13: Duration and spatial extent of deposited sediment thickness resulting from the maintenance dredge excavation of the BLF approach channel.

Sediment Thickness > mm	Duration (>= hours)	Area (ha)	
		Neap Tides	Spring Tides
20	1	0	0
20	3	0	0
20	6	0	0
20	12	0	0
50	1	0	0
50	3	0	0
50	6	0	0
50	12	0	0
300	1	0	0
300	3	0	0
300	6	0	0
300	12	0	0

4.2.2.5 *BLF barge docking*

During the delivery of AILs, rock armour and other marine freight, the docked barge will temporarily act as a blockage to the undisturbed tidal currents, causing the flow to divert around it. To assess the effect of the barge blockage, the barge was included in the high-resolution TELEMAC2D model by raising the bathymetric nodes it would occupy to 2 m ODN (i.e., above HAT), which forces the flow around it (BEEMS Technical Report TR481 Edition 2). A 94 m long and 27.4 m wide North Sea Barge was assumed. The mesh domain included the presence of the BLF deck piles and the associated dredge channel for the North Sea Barge. The proposed docking procedure is for the barge to approach at high tide, ground on the seabed with the falling tide and then leave on the following high tide. However, for the assessment, the barge was conservatively grounded for an entire tidal cycle. As the barge is only likely to dock when waves are small (< 0.6 m) to enable safe operations, only the effect of tidal currents was considered. The model was run for a 48-hour period encompassing the peak flood and ebb tidal conditions.

Figure 54 shows the difference in tidal velocities due to the presence of the barge for the peak ebb and peak flood tidal conditions. The velocity reduced in the lee of the barge with flows diverted around both the landward and seaward ends of the barge. The velocity reduction extends 600 m north on the ebb tide and 500 m south on the flood. Whilst the tidal currents are strongest on the flood, the reduction in velocity extends further on the peak ebb. This is due to the deeper water depths during the flood tide, allowing more flow to divert around the landward end of the barge as well as the seaward end, meaning the flow returns to an undisturbed condition quicker. During the ebb tide, there was a small increase in currents, 0.16 m/s, extending 50 m seaward of the barge and a larger increase of 0.38 m/s between the landward end of the barge and the MHWS mark.

Figure 55 shows the difference in current-only bed shear stress, due to the presence of the barge blockage, for the peak ebb and peak flood tidal conditions, respectively. The solid black line represents the ±5% change in bed shear stress contour. Under the peak ebb conditions, the area of seabed where the change in bed shear stress was greater than five percent (±5%) corresponded to 394,357 m^2 (or 39.44 ha) extending 1,100 m north, 775 m south and 300 m seaward of the end of the barge. Under the peak flood conditions, the area of seabed where the change in bed shear stress was greater than five percent (±5%) corresponded to 375,932 m^2 (or 37.59 ha) extending 890 m north, 950 m south and 300 m seaward of the end of the barge.

Figure 56 shows the difference in current only bed shear stress, due to the presence of the barge blockage, for the peak ebb and peak flood tidal conditions, respectively, with the solid black line representing the area where the bed shear stress has increased above the critical threshold of motion. This area of increase represents 0.215 ha and 0.184 ha during peak ebb and flood conditions, respectively. Whilst this represents a small area where sediment transport was not previously initiated due to tidal currents being diverted around the landward end of the barge, the impact of the barge is small. The peak increase in bed shear stress, between the beach and the landward end of the barge, is 1.6 N/m^2 and is approximately 30 times smaller than the bed shear stress contribution due to waves from a 1:0.2-year storm event. Any change is likely to be reprofiled by a small wave event. Furthermore, the impact due to a docked barge would be temporary and not persistent as the barge will not be present for the whole tidal cycle (as assessed here) nor will it be present on each tide.

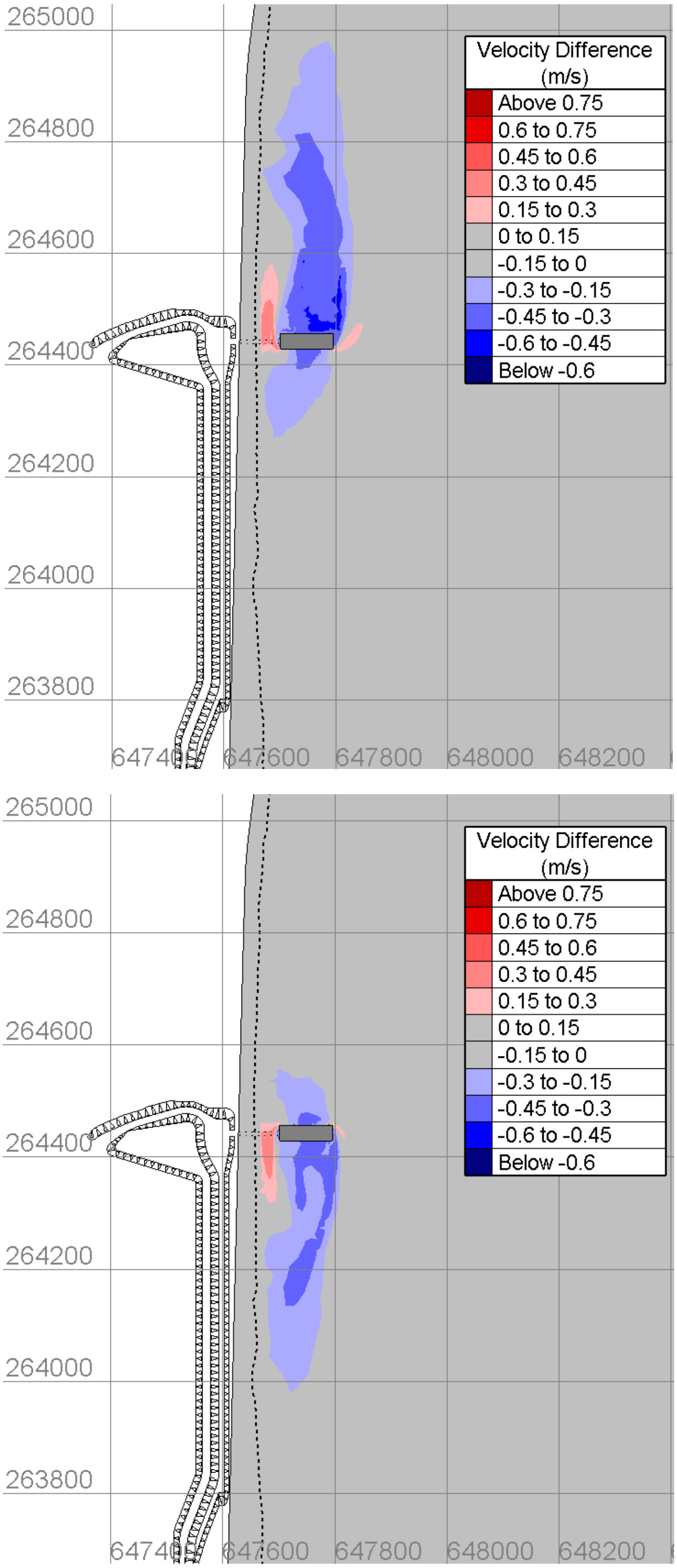

Figure 54: Velocity difference due to the barge during peak ebb (left) and peak flood (right) tidal conditions. The North Sea barge is represented by the dark grey block. The MWHS water line (1.22 m ODN) is shown by the black dashed line.

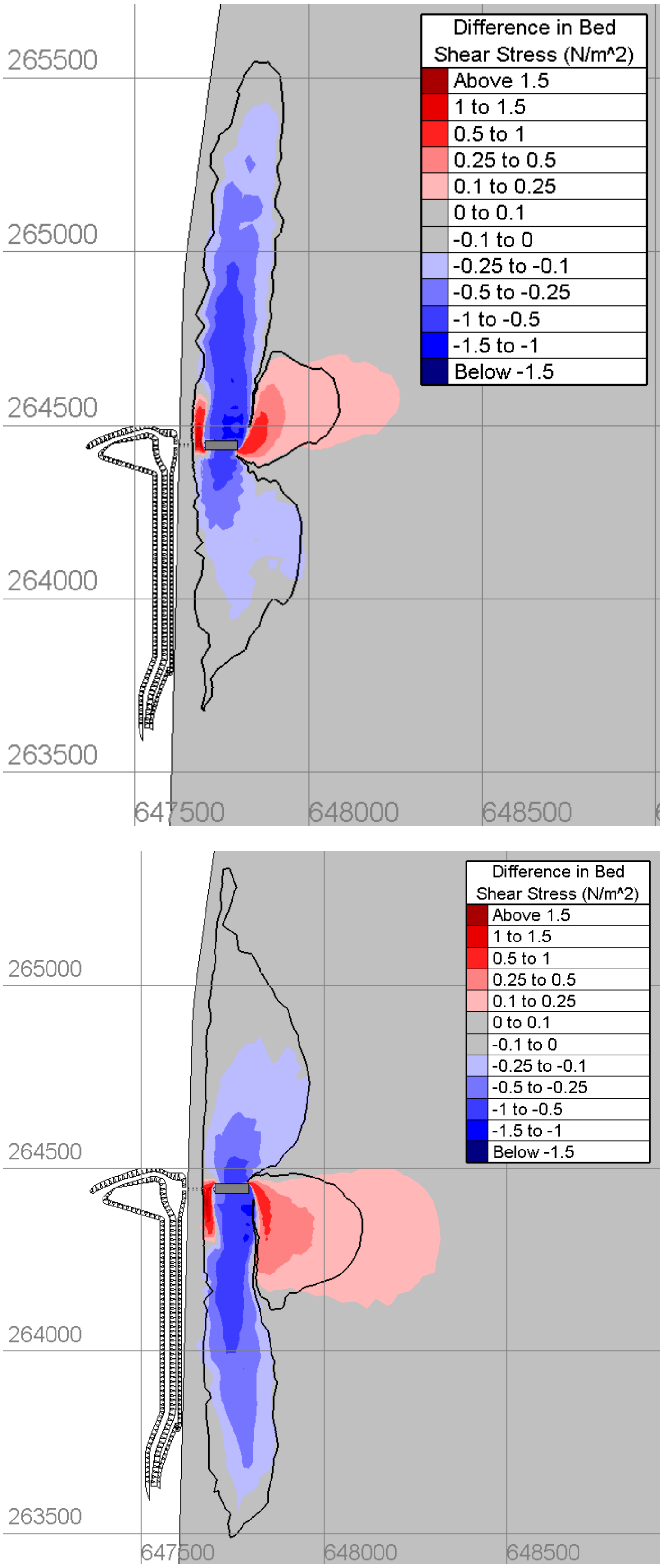

Figure 55: Difference in bed shear stress due to the barge during peak ebb (left) and peak flood (right) tidal conditions. The North Sea barge is represented by the dark grey block. The solid black line represents the ±5% change in bed shear stress contour.

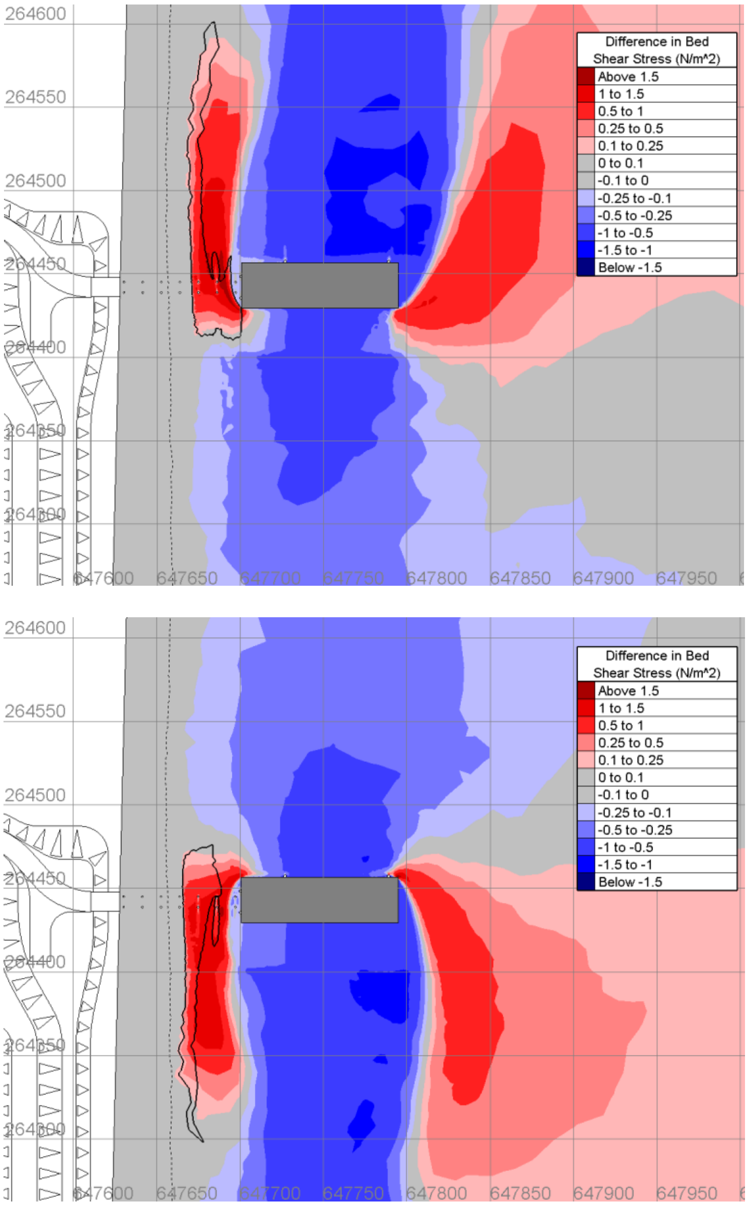

Figure 56: Difference in bed shear stress due to the barge during peak ebb (left) and peak flood (right) tidal conditions. The North Sea barge is represented by the dark grey block. The solid black line represents the area where bed shear stress has increase above the critical threshold.

4.3 Nearshore outfalls

4.3.1 Nearshore outfalls – installation

The nearshore outfalls consist of the fish recovery and return (FRR) systems and the combined drainage outfall (CDO), all of which would be on the seaward slope of the outer longshore bar and landward of the Sizewell-Dunwich Bank.

The FRR system would consist of two FRR outfalls, one for each reactor, each with a discharge tunnel with a mean discharge rate of 0.3 m^3/s. The outfall heads would be seabed mounted. The head design has not yet been undertaken, but for the purpose of this report has been assumed to comprise a concrete block approximately 3 m long, 4.5 m high and 3 m wide (subject to final engineering design) (BEEMS Technical Report TR333). The northing positions of each FRR align with the two reactor forebays and would have a subterranean tunnel length of c. 400 m, although the exact easting is yet to be finalised. The tunnel would be approximately 0.65 m diameter and would be directionally drilled from onshore with drill cuttings returned to land. The tunnels would be subterranean and have no effect on coastal processes.

The method of construction for the CDO has not yet been finalised but is likely to similar to that of the FRR and include a single directional-drilled subterranean tunnel with a terminating outfall block. Maximum discharge rates are likely to be around 0.124 m^3/s. The design of the outfall head has not yet been undertaken but for the purpose of this report has been assumed to comprise a concrete block approximately 3 m long, 4.5 m high and 3 m wide (subject to final engineering design), similar to the FRR block.

The only activity in the marine environment for the nearshore infrastructure would be dredging and emplacement of the FRR and CDO outfall heads. The heads would be designed to raise the discharge off the bed and provide a small amount of turbulence so that the system is self-scouring, ensuring that the outfalls do not block. For comparison, the nearby Sizewell B cooling water outfall has a discharge of 53 m^3/s.

4.3.1.1 *Dredging for nearshore outfalls*

The validated Delft3D model of Sizewell (BEEMS Technical report TR132), with its accompanying particle tracking module, has been used to estimate the distribution of sediment resulting from a range of potential dredge and sediment disposal scenarios planned for the construction and operation of Sizewell C (BEEMS Technical Report TR480). Dredging would be via Cutter Suction Dredger. The heads would be below the surface. The area of seabed affected is 44 m by 30 m, or 1,320 m^2, per outfall. The excavated volumes per head would be 1,845 m^3, or 5,535 m^3, for all three structures. The local bed sediments comprise of 95% sands (210 µm) and 5% fines (63 µm). Local disposal of dredged surficial sediment was assessed as a worst case, with the Cutter Suction Dredger discharging the dredge material through a pipe approximately 500 m from the dredge site to retain sediment within the system. The disposal of dredge material would occur simultaneously with the disturbance of the cutter head.

As the dredge dimensions are equal for each FRR and CDO, a single structure (FRR1) was modelled to represent the plume of the three individual activities. The dredge would take approximately 9.5 hours to complete for each FRR and the CDO (with 12 cycles of 19 minutes of dredging, followed by a 30-minute interval for repositioning).

The modelling results show that a narrow north-south shore orientated plume forms with maximum momentary SSC values greater than 100 mg/l above background confined to within 6.5 km to the north and 5.5 km to the south (Figure 57). These elevated concentrations are relatively short lived (see Table 14). The northern extent of the plume would be reduced on neap tides, however the southern extent of the plume with maximum concentrations above 200 mg/l would be slightly extended. This is due to sediment deposited on the bed during the smaller neap tides and subsequent re-erosion of material as the tidal flows increase on springs.

Following the completion of the dredge, the plume would quickly dissipate. On spring tides, material in suspension has concentrations less than 20 mg/l within two days of the completion of the dredge. Some of the dredge sedimentation close to the shore may interact with beach sediment at certain locations due to flows being insufficient to resuspend deposited material. This is likely to be deposited in thin sand veneers, of fine sand on the coarse sand beach, less than 1 mm thick along the Sizewell frontage and less than 1.5 mm thick at Thorpeness. These will be much like those that naturally occur along the Suffolk coastline and are unlikely to persist for long as they are naturally moved and mixed by wave action. There would be no measurable impact to the Leiston-Aldeburgh SSSI.

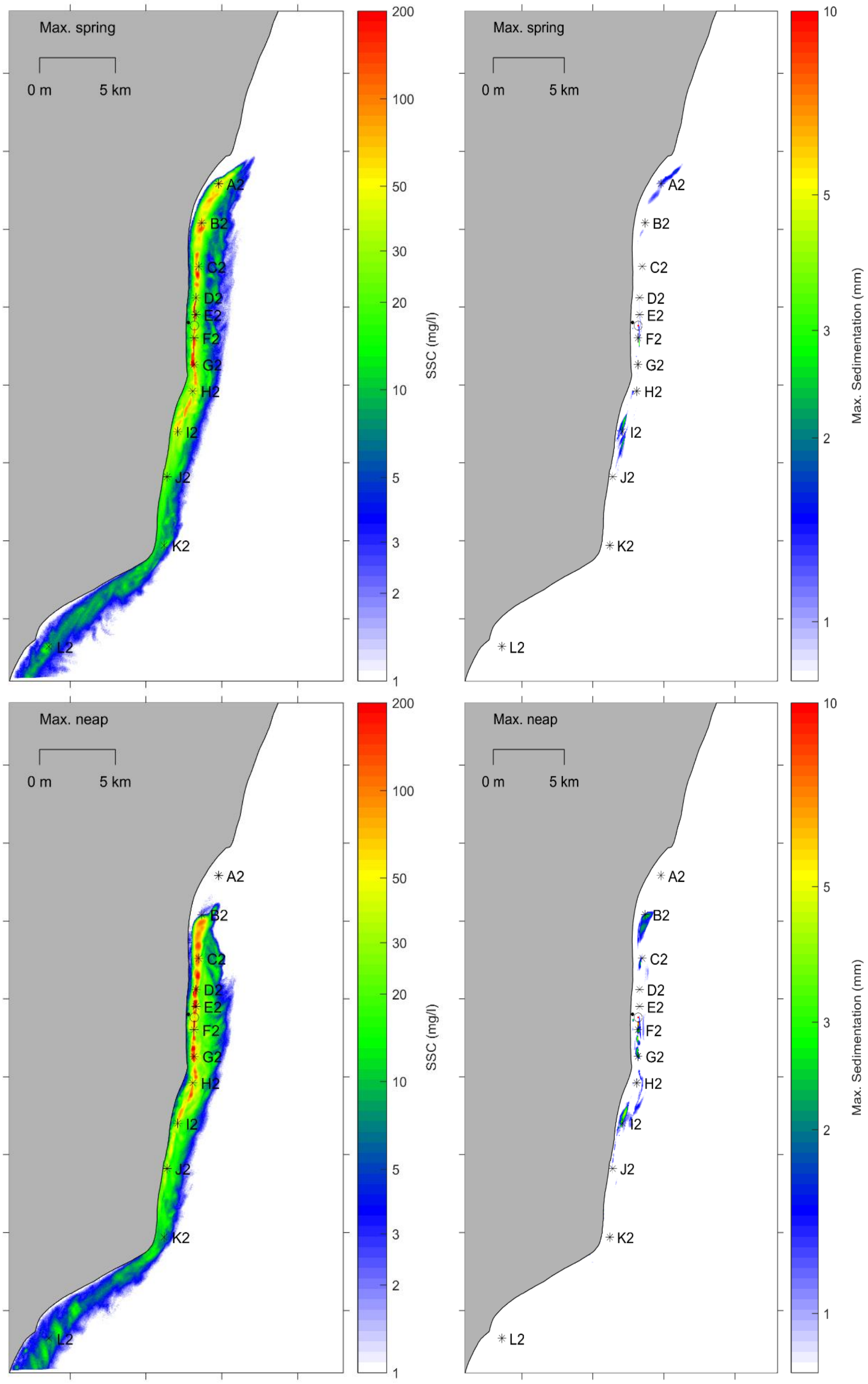

Figure 57: Maximum momentary SSC (left) and sediment deposition depth (right) on spring (top) and neap (bottom) tides during dredging of a single FRR/CDO (at any moment in model run-time). (From BEEMS Technical Report TR480).

Table 14: Duration and spatial extent of the SSC plume in the surface layer of the model resulting from excavation of the FRRs and CDO.

SSC > mg/l	**Duration (>= hours)**	**Area (ha)**	
		Neap Tides	**Spring Tides**
50	1	1,025	956
50	3	13	1
50	6	0	0
50	12	0	0
100	1	294	257
100	3	0	1
100	6	0	0
100	12	0	0
300	1	36	40
300	3	0	<1
300	6	0	0
300	12	0	0

Table 15: Duration and spatial extent of deposited sediment thickness resulting from excavation of the FRRs and CDO.

Sediment Thickness > mm	**Duration (>= hours)**	**Area (ha)**	
		Neap Tides	**Spring Tides**
20	1	1	1
20	3	0	0
20	6	0	0
20	12	0	0
50	1	0	0
50	3	0	0
50	6	0	0
50	12	0	0
300	1	0	0
300	3	0	0
300	6	0	0
300	12	0	0

Furthermore, the plume concentrations would be less than 200 mg/l above background values within hours of the dredge completion and as such may be difficult to detect. On neap tides, the plume concentrations in suspension also quickly return to values which are close to background. The model predicts some resuspension of material once spring tides occur, which could result in SSC of more than 20 mg/l in some localised areas. The model shows these increases in SSC are short lived (of the order of days).

The maximum momentary deposition would be highest at the disposal site (20 mm). Beyond the disposal site, maximum values of less than 5 mm occur in localised patches over an area between 7 km to the north and 8 km to the south. These patches correspond to the location of the plume at the time of slack water. The observed plume mainly results from the disposal of the dredged material. The plumes associated with sediment release at the cutter head would be indiscernible from the disposal plume on spring tides. On neap tides, the plumes from the cutter head can be identified. This deposition would be localised (extending approximately 1 km to the north and south) and at low thicknesses of bed accumulation (of around 1-2 mm).

4.3.1.2 *Vessel anchoring*

The placement of the outfall head will most likely be performed from a jack-up barge whose feet would cover a small area (e.g. total of 12 m^2 for a four-legged jack-up barge with legs of 2 m diameter) of the sea floor and some scour may result. However, whilst scour may occur for the duration of the operation, it would be short lived and may not have time to reach equilibrium.

Whilst the effect of vessel anchoring is anticipated to be minimal with natural infilling leading to recovery, it is not certain how deep the depressions left by the jack-up barge feet would be. Therefore, estimates of the predicted infilling due to tidal currents and waves, were calculated for a range of depression depths (BEEMS Technical Report TR487). The depression depths considered were 0.5 m, 1.0 m and 2.0 m. To allow a conservative estimate of the depression width, it was assumed that a scour pit would fully form around the jack-up leg. The total width of the depressions (width of the leg plus the scour either side) considered were 3.99 m, 5.98 m and 9.96 m respectively, with an area of seabed affected of 17.2 m^2, 42.2 m^2 and 124.8 m^2.

Estimated average infilling rates were based upon 11 years of offshore hindcast wave data (2001-2012; Met Office 'ReMap' European wave model). The wave climate was generated by transforming the offshore wave field to the outfalls using the TOMAWAC wave model (BEEMS Technical Report TR319). For the contributions to the infilling rate due to tidal currents, a time series of the modelled tidal currents and water depths were extracted from the updated Sizewell regional TELEMAC2D model, which was run for 1 month (07/11/2013 to 06/12/2013) (BEEMS Technical Report TR481). This was consecutively repeated to create a time series matching the length of the 11-years of modelled wave data.

Figure 58 presents the average monthly infill rate for different depression depths, with the monthly standard deviations (black whiskers). In the nearshore environment, the infill rate was dominated by waves. When waves were lower, slower infilling caused by tides only was observed. There is a strong seasonal cycle in the infill rates, with higher and more frequent infilling under common winter waves compared to the more quiescent summer conditions. Table 16 details the time taken for each of the depression to infill using the average infill rate for the summer (June-August), winter (November-January) and likely operational window (April-October).

Table 16: Time to infill for range of scour depressions.

Conditions	**Time to infill (days)**		
	0.5 m	**1.0 m**	**2.0 m**
Summer (June-August)	13.7	30.6	90.3
Winter (November-January)	4.5	9.3	25.6
Operational (April-October)	8.2	17.5	50.0

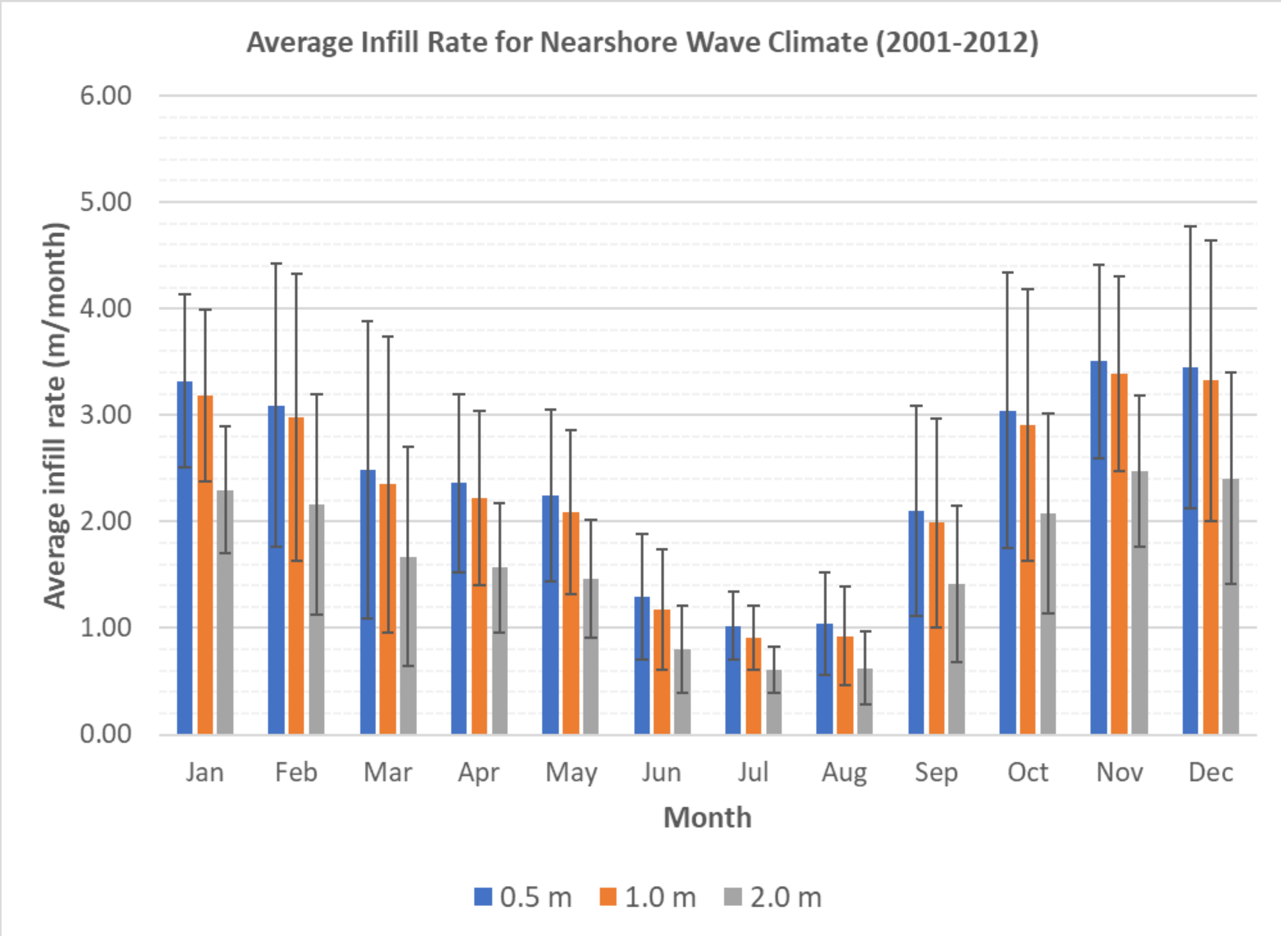

Figure 58: Average infill rate (m/month) for range of scour depressions in the nearshore using modelled wave data. Black whiskers indicate inter-annual variability (1 standard deviation) (BEEMS Technical Report TR487).

4.3.2 Nearshore outfalls – presence

4.3.2.1 *Scour around FRR and CDO structures*

The presence of the CDO and FRR outfall heads would disrupt local hydrodynamic flow patterns, lowering the seabed around the structures to form scour pits. Scour pit depth and extent was estimated for the FRR and CDO outfall heads. Structure geometry was idealised as a rectangular block of 3 by 3 m, protruding 2.5 m above initial bed level (BEEMS Technical Report TR310 Edition 2). The most conservative empirical method was selected from a range of appropriate empirical methods, assuming that no scour protection or granular infill was installed and using a worst-case hydrodynamic scenario (peak tidal current during a storm surge event). Scour depth was predicted to be 2.07 m at each structure based on Zhao *et al*.'s (2012) methodology for rectangular subsea caissons (BEEMS Technical Report TR310 Edition 2).

In a combined wave and current scenario, 1:0.2 and 1:20-year return period waves were predicted to cause scour pit backfilling, reducing the scour depth when compared to the current only scenario. In a wave-only scenario with no currents, waves from these scenarios were predicted to have no influence on the seabed.

Geotechnical surveys suggested the seabed sediment is 3.81 m thick in the vicinity of FRR2 and the CDO (at vibrocore location VC21 as shown in Figure 59). The seabed thickness is assumed to be similar at the nearby FRR1 location (c. 340 m south of FRR2). As a result, scour would not reach the underlying geology at any of the nearshore structures.

The scour pits would be broadly elliptical due to reversing tidal currents, with a 7.2 m extent from each side of the structure along the tidal axis (north – south) and a 4.1 m extent across the tide (east – west). Therefore, the area influenced (including the 9 m^2 footprint of the structure itself) would be 170 m^2 (0.0170 ha) per structure and 510 m^2 (0.051 ha) for the three structures. The amount of sediment displaced due to the formation of the predicted scour pits would be 109 m^3 per structure and 328 m^3 for the three structures (excluding the volume of the structures themselves).

In addition to scour due to the presence of the structure, there would be potential for scour due to the continuous discharge of water from the CDO and FRR outfalls. By comparing the outputs from two empirical equations for jet scour, scour was conservatively estimated to be 0.76 m deep for the FRRs and 0.56 m deep for the CDO, which, being less than scour depth due to the structures themselves, is unlikely to have an additional influence on the seabed.

4.3.2.2 *Scour protection around FRR and CDO structures*

Full scour protection design is not yet complete, however, assuming that scour protection would extend a minimum of 10 m from the structure along the tidal axis (north – south) and 3 m across the tide (east – west) and would be installed with a rounded rectangular configuration, scour protection would have a total extent of 23 by 9 m per structure. The area of changed habitat (including the 9 m^2 footprint of the structure itself) would be 207 m^2 (0.0207 ha) per structure and 621 m^2 (0.0621 ha) for the three structures.

Secondary or edge scour would be likely to form around the perimeter of the scour protection, as observed at the Sizewell B intake and outfall heads (secondary scour depths of between 2 and 4 m have been; though note jet scour may affect the outfall), and therefore, were scour protection to be installed over the entire projected footprint, the secondary scour would mean that the total area influenced by the presence of the structure would be larger than if no scour protection was installed.

Once Sizewell C has stopped operating and the nearshore outfalls are no longer needed, they would be removed to below the sea floor following standard practice for structures in the North Sea.

4.4 Offshore cooling water infrastructure

4.4.1 Cooling water intakes and outfalls – installation

The four intake and two outfall structures would be emplaced during the Sizewell C construction phase and be present for the entire operational life of the station and a substantial part of the decommissioning phase. The two intake tunnels and one outfall tunnel would be bored from landward and would have no effect on coastal geomorphology and hydrodynamics because they are subterranean. The intakes and outfall heads would be seismically qualified, which means the local seabed sediments would be removed, connecting shafts drilled down through the bedrock and finally the heads lowered and pin-piled into place, at each of the structures.

The presence of the heads and their installation (e.g., drilling over vertical connection shafts) would have effects in the marine environment that are discussed in this section. The heads would stand proud of the seabed and cause scour beyond the construction zone, so it is necessary to determine the extent of scour induced habitat loss or change. Figure 59 shows the location of geotechnical measurements (Fugro, 2015) that were used to assess the substrate and undertake scour calculations.

4.4.1.1 *Intake and outfall dredging and disposal*

The validated Delft3D model of Sizewell (BEEMS Technical report TR132), with its accompanying particle tracking module, was used to estimate the distribution of sediment resulting from a range of potential dredge and sediment disposal scenarios planned for the construction and operation of Sizewell C (BEEMS Technical Report TR480). Dredging would be done via a Cutter Suction Dredger. The area of seabed affected would be 77.5 m by 65 m, or 5,037.7 m^2, per intake and 61 m by 61 m, or 3,721 m^2, per outfall. Using a worst-case surficial sediment depth of 6 m, the excavated volumes would be 17,400 m^3 and 11,750 m^3 per intake and outfall, respectively, or 93,100 m^3 for all intakes and outfalls. Three grainsizes were used in the modelling to represent the distribution of local bed sediments: 20% coarse sand (420 µm), 75%

medium sands (210 µm) and 5% fines (63 µm). A local disposal of dredged surficial sediment has been assessed as a worst case, with the Cutter Suction Dredger discharging the dredge material through a pipe approximately 500 m from the dredge site. The disposal of dredge material in the model occurs simultaneously with the disturbance of the cutter head.

For the purposes of modelling, dredging was considered for both a single intake and outfall separately. The dredging of surficial sediments would be expected to take approximately 8.5 hours to complete at each intake structure (with 9 cycles of 30 minutes of dredging, followed by a 30-minute interval for repositioning) and 7 hours at each outfall head (with 9 cycles of 20 minutes of dredging, followed by a 30-minute interval for repositioning).

The modelling results show that during the dredging and associated local disposal of surficial sediments, an elongate area extending approximately 13 km to the north, 22 km to the south and a couple of kilometres east-west is affected by increases in SSC of more than 100 mg/l (Figure 60). Maximum momentary SSCs within the plume peak at less than 2,000 mg/l above background away from the disposal site, with a maximum of 4,320 mg/l limited to the immediate disposal site. These elevated concentrations are relatively short lived (see Table 17). Whilst the 100th percentile produces typical values of approximately 100 mg/l above background, the 95th percentile produces values typically around 20 mg/l, which is reflective of the short nature of the plume. The observed plume is mainly associated with the disposal of the dredge material; the plumes from the local seabed disturbance at the cutter head are localised (extending approximately 1 km) and at low concentrations of around 40 mg/l above background. The plume is located offshore with no evidence of elevated SSC or sediment deposition near the coast. Following completion of the dredge, the plume quickly dissipates – the elevated concentrations decay to background levels within c. two days on both spring and neap tides after the completion of the disposal operations.

Deposition close to the disposal site is high, peaking at 1,400 mm (1.4 m). The maximum momentary deposition thickness reduces with distance from the disposal site with maximum values reducing to 50 mm at distances of more than 1 km. After a spring-neap cycle, approximately 7% of the material released remains and is comprised mainly of medium sands. The area affected by deposition is localised being mostly confined to a sheltered area inshore of Whiting Bank, approximately 4 km offshore of Aldeburgh and approximately 23 km south of the dredging, with the thickness of deposits less than 2-3 mm. The deposits do not reach the shore. Details on dredge disposal characterisation are given in Appendix 22K of this volume of the ES.

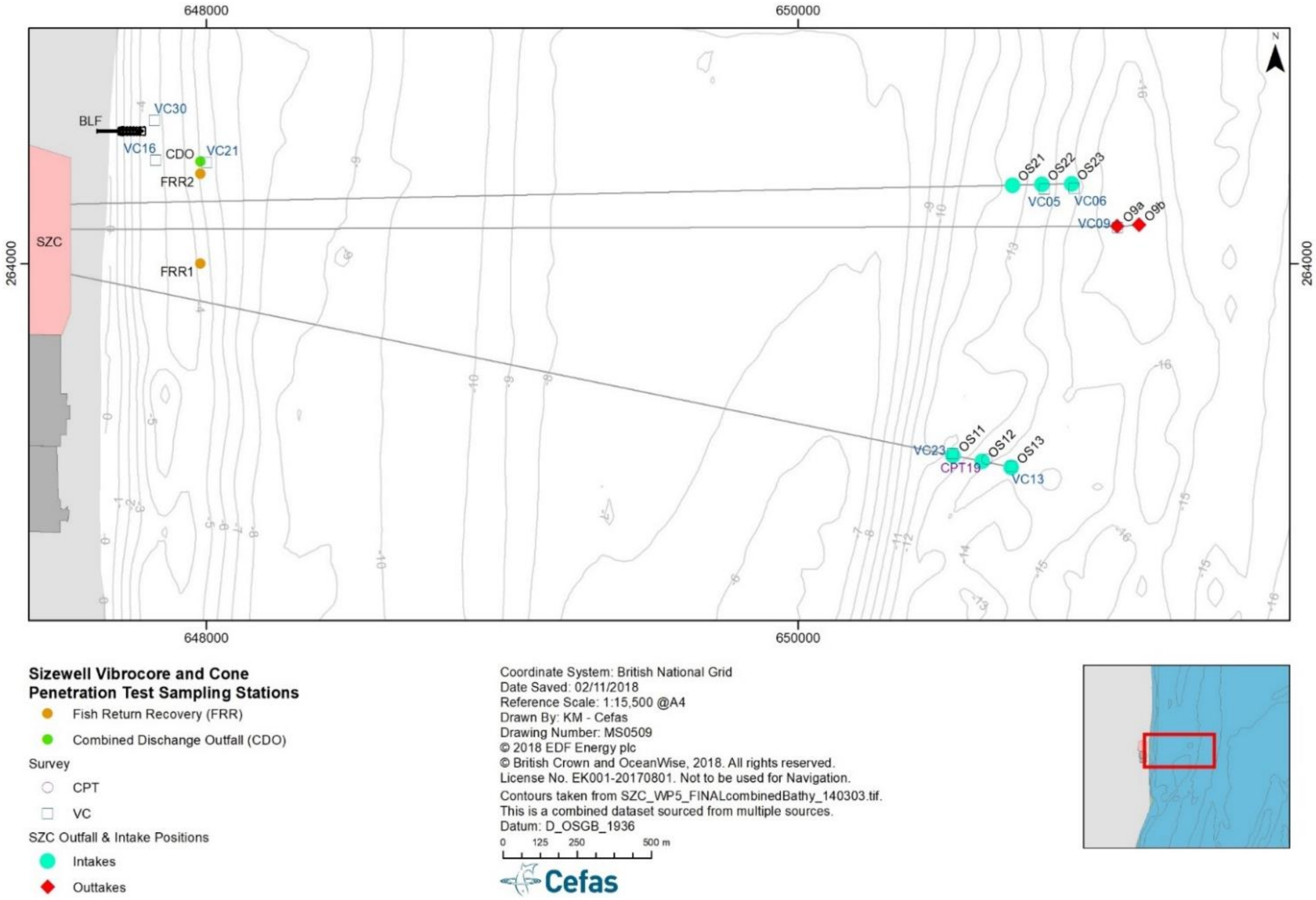

Figure 59: Geotechnical survey locations using vibrocores (VC) and cone penetrometer testing (CPT) (Fugro, 2015). Note that there are only two intake heads at each site – three are shown as the final locations have yet to be selected. 2019 geotechnical survey results are not yet available for reporting.

Table 17: Duration and spatial extent of the SSC plume in the surface layer of the model resulting from dredging at the cooling water intake and outfalls.

SSC > mg/l	Duration (>= hours)	Area (ha)	
		Neap Tides	Spring Tides
50	1	3,839	4,281
50	3	106	51
50	6	0	0
50	12	0	0
100	1	2,383	2,226
100	3	95	145
100	6	0	0
100	12	0	0
300	1	594	443
300	3	1	<1
300	6	0	0
300	12	0	0

Table 18: Duration and spatial extent of deposited sediment thickness resulting from dredging at the CWS intake and outfalls.

Sediment Thickness > mm	Duration (>= hours)	Area (Ha)	
		Neap Tides	Spring Tides
20	1	106	15
20	3	4	0
20	6	4	0
20	12	4	0
50	1	7	4
50	3	3	0
50	6	3	0
50	12	3	0
300	1	2	1
300	3	1	0
300	6	1	0
300	12	1	0

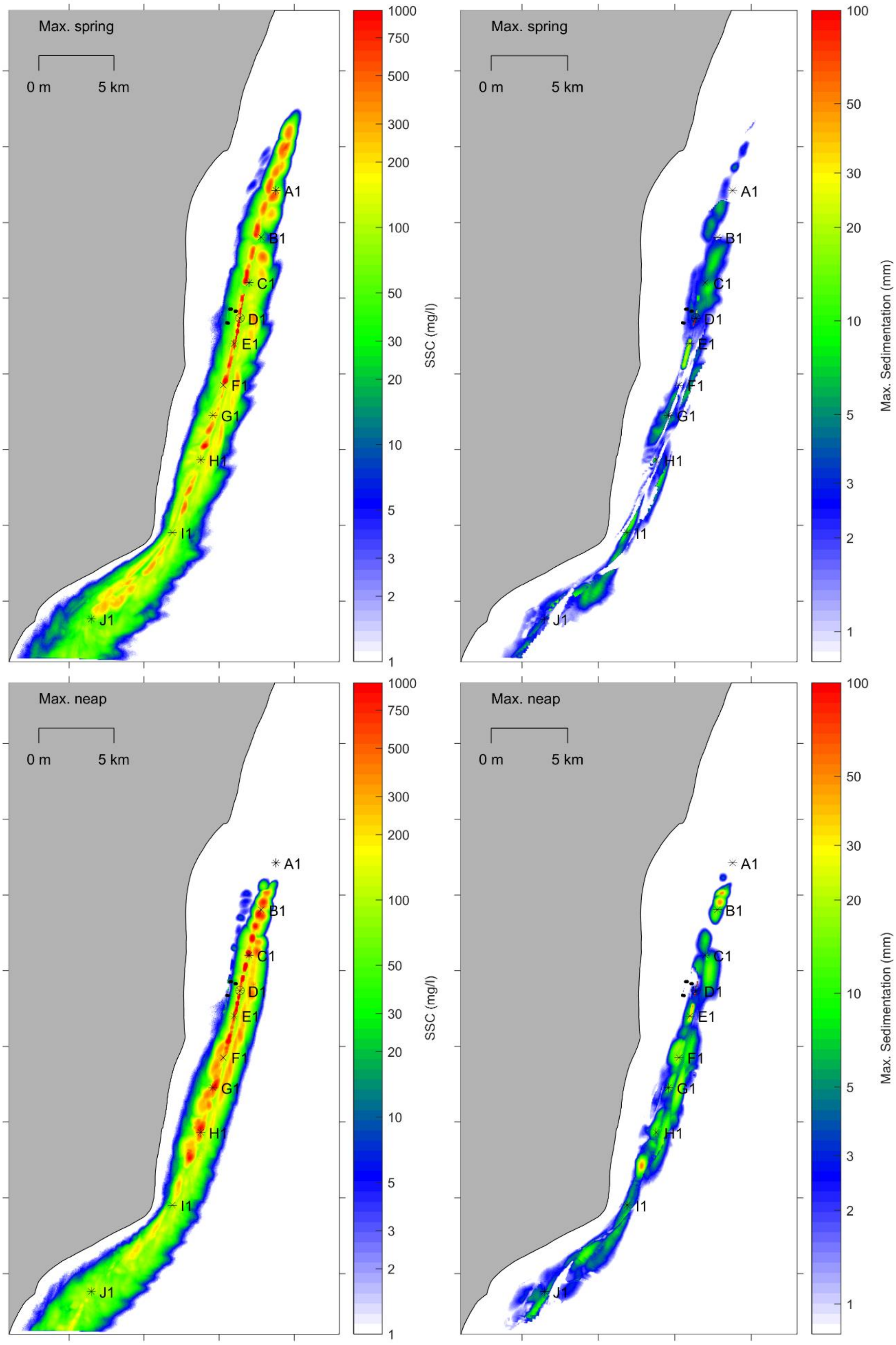

Figure 60: Maximum momentary SSC (left) and sediment deposition depth (right) on spring (top) and neap (bottom) tides during dredging of a CWS intake (at any moment in model run-time). (From BEEMS Technical Report TR480).

4.4.1.2 *Intake and outfall drilling*

Once the dredging has been completed, drilling would connect the intake/outfall heads to the tunnels below. The vertical drilling through the bed rock is modelled as a separate activity from dredging.

The drill arisings from drilling through bedrock would be clasts that would settle immediately adjacent to the drill sites (50%) and fine sediments of c. 63 μm (50%). The *in-situ* mass of the material removed during the drilling operation would have a density of 2160 kg/m^3, which is based on the material properties of a 'medium strength limestone' (BGS, 1993). The in-situ volume of material to be drilled for each intake and outfall are estimated to be 754 m^3 and 954 m^3, respectively. These results represent a worst-case scenario based upon pessimistic estimates of vertical shaft diameters. A bulking factor of 45% has been applied to the drill arisings.

For the purposes of modelling a realistic worst case, the duration of drilling was condensed from 15 consecutive 12-hour shifts, to a continuous 180-hour period (7.5 days). This provides a higher intensity of release and avoids any constraints being placed on drilling times. Separate modelling was undertaken for a single intake and a single outfall.

The modelling results in Figure 61 and Figure 62 show that the plume would spread along the tidal excursion, in a relatively narrow band and with maximum momentary deposition thickness on the bed of <1 mm (BEEMS Technical Report TR480). Maximum momentary SSC's were typically less than 10 mg/l above background concentrations and as such would not be detectable against the background variability. Following the completion of the drilling, the bedrock plume would disperse with no plume remaining 17 hours after drill completion.

A spoil heap of coarse material would be expected to settle close to the drill site. The footprint area of the spoil heap will depend on the release height for the drill arisings, which is not currently known. A release at the surface will result in a spoil heap extending up to 60 m from the drill location for the coarsest fractions (>10,000 μm), while the less coarse fractions (around 1,000 μm) will settle within 200 m (BEEMS TR480). The thickness of deposits at the spoil heap are likely to be of the order of centimetres to less than 20 cm.

Table 19: Drill depth of volume for both intake and outfall shafts.

Name	Type	Northings	Easting	Diameter (m) (worst case assumption)	Drill Depth below seabed (m)	In-situ Volume of Drilling (m^3)
O9a	Outfall	264125	651080	9	15	954
O9b	Outfall	264125	651155	9	15	954
I4a	Intake	263360	650526	8	15	754
I4b	Intake	263341	650624	8	15	754
I3a	Intake	264262	650726	8	15	754
I3b	Intake	264264	650826	8	15	754

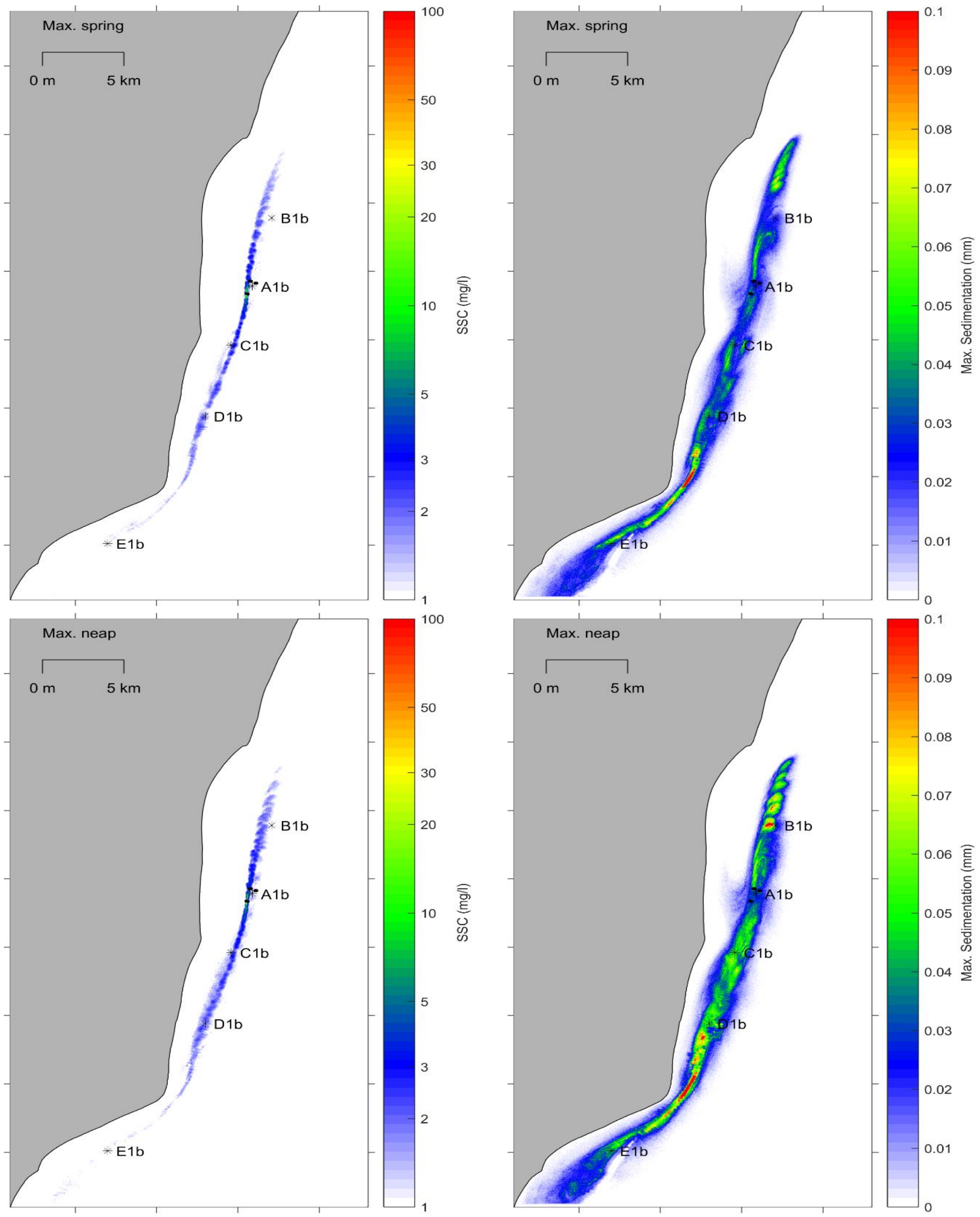

Figure 61: Maximum momentary SSC (left) and sediment deposition depth (right) on spring (top) and neap (bottom) tides during drilling of Intake I4a (at any moment in model run-time). (From BEEMS Technical Report TR480).

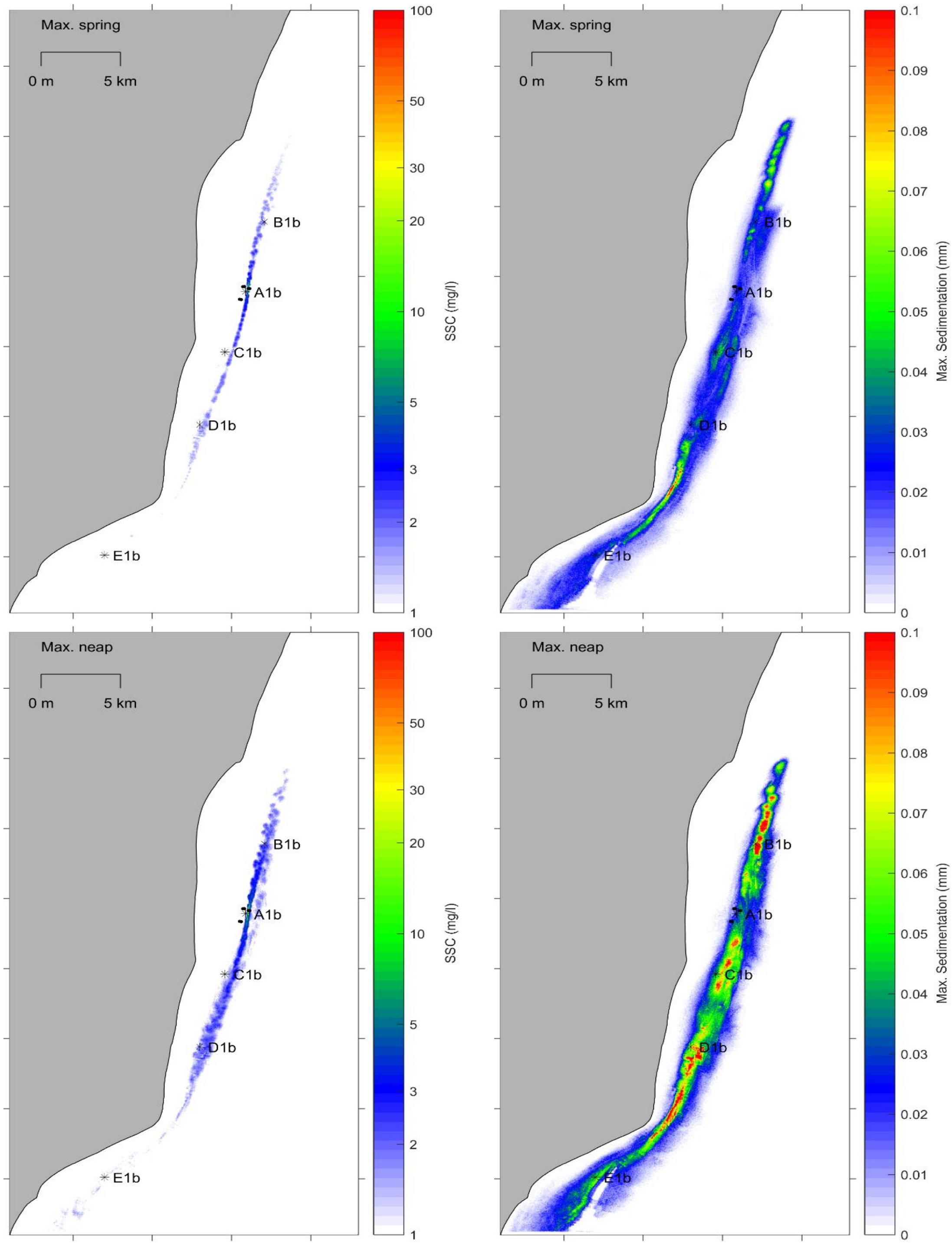

Figure 62: Maximum momentary SSC (left) and sediment deposition depth (right) on spring (left) and neap (right) tides during drilling of outfall O9a (at any moment in model run-time). (From BEEMS Technical Report TR480).

4.4.1.3 *Head emplacement*

Emplacement of the heads will imply the permanent loss of an area of seabed.

The outfall head assumed is a 16 x 16m block and hence a total area of 256m^2 per head and a total of 512m^2. The expected area of scour protection required is currently not known for these heads. The assumed worst-case intake head dimension is 32.5m x 20m, including nominal scour protection of the base plate, amounting to 650m^2 per head, or a total of 0.15ha for each of the north and south intakes locations.

Red Crag sands underly both the northern intake and outfall locations. Installation of southern intake heads would result in a permanent reduction in the area of Coralline Crag outcrops of approximately 0.15ha.

4.4.1.4 *Vessel anchoring*

Drilling would most likely be performed from a jack-up barge. The feet of the jack-up barge would cover a small area (e.g. total of 12 m^2 for a four-legged jack-up barge with legs of 2 m diameter) of the sea floor and some temporary scour may result. However, the impact of the jack-up barge is likely to be within the footprint of the scour that would result from the intake structures, which is more substantial.

Whilst the effect of vessel anchoring is anticipated to be minimal with natural infilling leading to recovery, the depth of jack-up barge feet depressions is unknown. Therefore, estimates of the predicted infilling due to tidal currents and waves were calculated for a range of depression depths (BEEMS Technical Report TR487). The depression depths considered were 2.0 m, 4.0 m and 6.0 m. To allow a conservative estimate of the depression width, it was assumed there was infinite depth of erodible sediment and that a scour pit would fully form around the jack-up leg. In the southern intakes area, vibrocores contained unerodable coralline crag with minimal overlying mobile sediment at the sampled sites (Fugro 2015) meaning any depression will be smaller and the infilling times are conservative. The width of the depressions considered were 9.96 m, 17.93 m and 25.86 m respectively, with an area of seabed affected of 124.8 m^2, 421.0 m^2 and 891.6 m^2.

Estimated average infilling rates were based upon 11 years wave data (2001-12). The modelled wave data was generated by taking the offshore wave field (Met Office ‘ReMap’ European wave model) and transforming it inshore using the TOMAWAC wave model (BEEMS Technical Report TR319). For the contributions to the infilling rate due to tidal currents, a time series of the modelled tidal currents and water depths, were extracted from the Sizewell regional TELEMAC2D model updated with 2017 bathymetry, which was run for 1 month (07/11/2013 to 06/12/2013) (BEEMS Technical Report TR481). This was consecutively repeated to create a time series matching the length of the 11-years of modelled wave data.

Figure 63 presents the average monthly infill rate for depression depth, with the monthly standard deviations (black whiskers). Due to the greater water depths outside the Sizewell-Dunwich Bank, the infill rate was dominated more by tidal currents than waves. As a result, there is a smaller seasonal cycle, compared to inside of the bank, with a smaller variation in infilling rates, with the higher and more frequent infilling under common winter waves compared to the more quiescent summer conditions.

Table 20 details the time taken for each of the depression to infill using the average infill rate for the summer (June-August), winter (November-January) and likely operational window (April-October). Whilst it may take four months to infill the deepest depression, this is not a significant to the geomorphology as the activity only occurs during installation of the intakes/outfalls. Once the activity is over, the depressions will naturally infill, and the depression will be gone. The area of seabed in question is ubiquitous soft sediment so will return to its natural state.

Table 20: Time to infill for range of scour depressions.

Seasonal windows	Time to infill (days)		
	2.0 m	4.0 m	6.0 m
Summer (June-August)	19.7	61.5	131.1
Winter (November-January)	14.5	44.0	93.0
Operational (April-October)	18.2	56.4	120.0

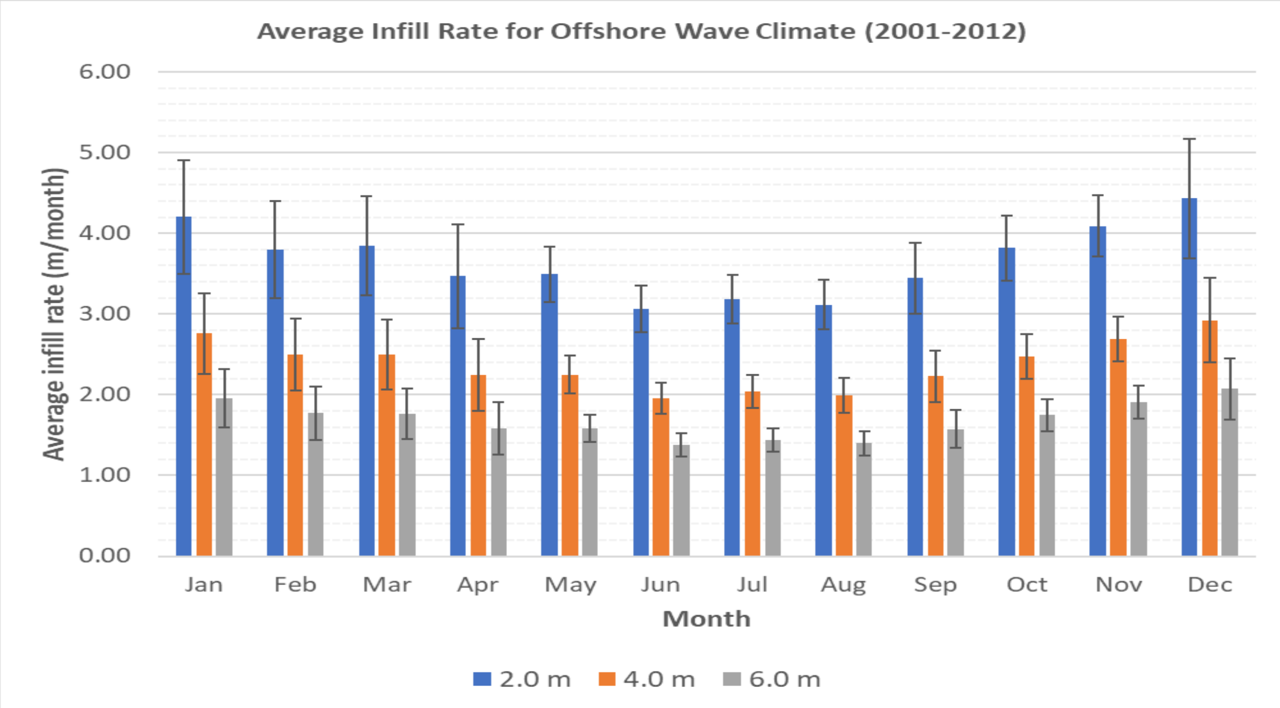

Figure 63: Average infill rate (m/month) for range of scour depressions in the nearshore using modelled wave data. Black whiskers indicating inter-annual variability (1 standard deviation) (BEEMS Technical Report TR487).

4.4.2 Cooling water intakes and outfalls – presence

The presence of the intake and outfall heads would disrupt local hydrodynamic flow patterns, lowering the sedimentary seabed around the structures to form scour pits. During Sizewell C decommissioning, once they are no longer needed, the cooling water intakes and outfalls would be removed to below the sea floor following standard practice for structures in the North Sea.

4.4.2.1 *Scour around intake structures*

Scour pit depth and extent was estimated for the cooling water intake head structures using the same methods and assumptions used for the nearshore FRR and CDO outfalls (Section 4.3.2). For the purpose of the assessment, structure geometry was idealised as a rectangular block of 32 by 10 m, protruding 4 m above the initial bed level. The most conservative estimate of scour depth (Zhao *et al*. 2012) was 4.26 m at each of the proposed locations in a currents-only scenario and assuming no scour protection was installed (BEEMS Technical Report TR310 Edition 2).

Waves from a 1:0.2 and 1:20 year event would not influence scour development, either on their own or in combination with tidal currents, due to their limited influence at the bed in the water depths at the proposed locations.

Geotechnical measurements at the locations in Figure 59, indicated that bedrock at the northern intakes is overlain by a 4.62 m thick sediment layer in the vicinity of site OS23, and at least 5.80 m thick at the other two potential sites. In the southern intakes area, vibrocores contained unerodable coralline crag with minimal overlying mobile sediment at the sampled sites (Fugro 2015). For the purpose of predicting worst case scour, it was assumed that the sediment thickness exceeded predicted scour depth at all sites, as vibrocores represent single point samples only which were not precisely at the locations of the intake heads, and in a worst case scenario sediment could be present at these locations, allowing scour development.

Scour pits would be broadly elliptical due to reversing tidal currents, with a 14.8 m extent from each side of the structure along the tidal axis (north – south), and an 8.5 m extent from each side of the structure across the tide (east – west). This did not account for tidal asymmetry at the site, and instead conservatively assumed that the north and south scour pit extents were as predicted for downstream scour extent under unidirectional flow.

The area influenced (including the 320 m^2 footprint of the structure itself) was estimated to be 1554 m^2 (0.1554 ha) per structure and 6217 m^2 (0.6217 ha) for all four structures. In the affected area, seabed depth would increase and sediment characteristics may be altered. The amount of sediment displaced due to the formation of the predicted scour pits would be 2297 m^3 per structure and 9189 m^3 for all four structures (excluding the volume of the structures themselves).

4.4.2.2 *Scour around outfall structures*

Scour pit depth and extent was estimated for the cooling water outfall head structures using the same methods and assumptions as the intakes (Section 4.4.2.1) and idealising structure geometry as a rectangular submerged block of 16 by 16 m, protruding 4 m above the initial bed level. The most conservative empirical method estimated a scour depth of 4.67 m at each structure. Waves from a 1:0.2 and 1:20 year event would not influence scour development at the proposed sites, due to the water depth.

Geotechnical surveys at the site of O9a (Figure 59) revealed that sediment is more than 6.21 m deep. Assuming that bedrock depth was similar at nearby O9b, predicted scour depth was less than the estimated depth of bedrock at both locations.

Scour pits would be elliptical due to reversing tidal currents, with a 16.3 m extent from each side of the structure along the tidal axis (north – south) and a 9.3 m extent from each side of the structure across the tide (east – west). The area influenced (including the 256 m^2 footprint of the structure itself) was estimated to be 1552 m^2 (0.1552 ha) per structure and 3105 m^2 (0.3105 ha) for both structures. The amount of sediment displaced would be 1873 m^3 per structure and 3746 m^3 for both structures (excluding structure volume).

The same approach used for the CDO and FRR outfalls was applied to predict scour potential due to the discharge of cooling water from the outfalls. Scour was conservatively estimated to be 6.75 m deep, which is greater than the scour depth predicted due to the structure alone, and could expose underlying bedrock. The scour pit formed would have an estimated length of 53 m extending away from the structure, a maximum width of 24 m, and would affect an area of 1013 m^2 (0.1013 ha) per structure or 2025 m^2 (0.2025 ha) for both (conservatively assuming an elliptical scour pit shape). Such estimates are likely to be an overestimation, as they assume the discharge at bed level, and that discharge comes from a single circular jet rather than two adjacent outlets.

4.4.2.3 *Scour protection around intake and outfall structures*

HPC intake head design drawings include scour protection extending 10 m around the perimeter of the structure. Detailed engineering design of scour protection has yet to be completed for Sizewell C, however for the purposes of assessment it is assumed that, if installed, scour protection would also have a minimum 10 m extent from the perimeter of the Sizewell C structures. Intake scour protection would have a total extent of 52 by 30 m in a rounded rectangular configuration and the area of changed habitat (including the 320 m^2

footprint of the structure itself) would be 1474 m^2 (0.1474 ha) per structure and 5897 m^2 (0.5897 ha) in total for the four intakes. Outfall scour protection would have a total extent of 36 by 36 m in a rounded rectangular configuration. The area of changed habitat (including the 256 m^2 footprint of the structure itself) would be 1210 m^2 (0.121 ha) per structure and 2420 m^2 (0.242 ha) in total for both outfalls.

At the Sizewell B intake heads, scour protection with a 77.5 m long and 35 m wide footprint was deployed. This was shown to limit the depth of scour beyond the scour protection to around 2 m depth. Therefore, similar secondary or edge scour would be likely to form around the perimeter of the scour protection installed at the Sizewell C intakes and outfalls, as observed at the Sizewell B intake heads.

5 Cumulative environmental assessment for coastal geomorphology

The cumulative environmental effects of Sizewell C for coastal geomorphology occur if:

- Two (or more) Sizewell C activities overlap in time and space – called inter-relationships (Section 5.1.2), and
- the geomorphic extent of any Sizewell C activity overlaps in time and space with the effects of other neighbouring 3rd party projects – called cumulative effects (Section 5.1.3).

Definitions are provided in Section 5.1, whilst lists of the within the project activities and the 3rd party projects and plans are shown in Section 5.2. Additionally, the temporal and spatial overlap of the aforementioned components is presented in Section 5.3, based on the current Sizewell C construction and operation schedule for cumulative assessment. Where there are timing uncertainties, the schedule elements will occupy a longer period to ensure potential effects are captured. That is, some effects may not occur, and the schedule used is specifically for the CEA. Finally, it should be noted that the assessment of the inter-relationship and cumulative activities' impacts will be presented in the Environmental Statement – this section sets out the evidence base for the assessment.

5.1 Definitions

5.1.1 Zone of influence

The Zone of Influence (ZoI) for coastal geomorphology is the Greater Sizewell Bay (GSB - Figure 3) with no significant cumulative effects to geomorphic receptors foreseen beyond that coastal sediment cell.

5.1.2 Inter-relationship impacts

These activities are also known as 'intra-project', 'synergistic', 'Type 1 cumulative', or 'interactive' impacts/ relationships. Any inter-relationship impacts would occur if different individual environmental effects of the proposed development combine together to influence the geomorphic receptors. However, if considered in isolation, these individual environmental impacts may not lead to significant effects.

5.1.3 Cumulative impacts

These activities are also known as 'inter-project', 'Type 2 cumulative', or 'additive' impacts. They would occur if effects from the proposed development combine with those from other planned / potential 3rd party projects/activities (in this case within the Sizewell C geomorphology ZoI), resulting in a change to the overall magnitude of impact acting on a receptor. These projects are categorised in Tiers relevant to Sizewell C MDS DCO application status / timeline, according to their planning stage or present status (e.g. operational), as described in Planning Inspectorate Advice Note 17 as shown in Table 21.

Table 21: Tiers and descriptions of 3rd party projects relevant to Sizewell C's DCO application, as set by the Planning Inspectorate (Advice Note 17)

Tier	Description
Tier 1	Operational projects – no potential for overlap in the construction phase of Sizewell C.
Tier 2	Marine infrastructure projects currently under construction and will be operational prior to the construction of Sizewell C.
Tier 3	Marine infrastructure projects that have been consented but for which construction has not yet started.
Tier 4	Marine infrastructure projects which have been submitted to the relevant regulatory body but not yet determined or projects consented but on hold due to legal challenge or appeal.
Tier 5	Marine infrastructure projects which the regulatory body are expecting to be submitted for determination.

5.1.4 Stages of project wide cumulative environmental assessment

The stages - steps followed for the project wide cumulative environmental assessment are shown in Figure 64. Stage 4 will be discussed and presented in the Environmental Statement.

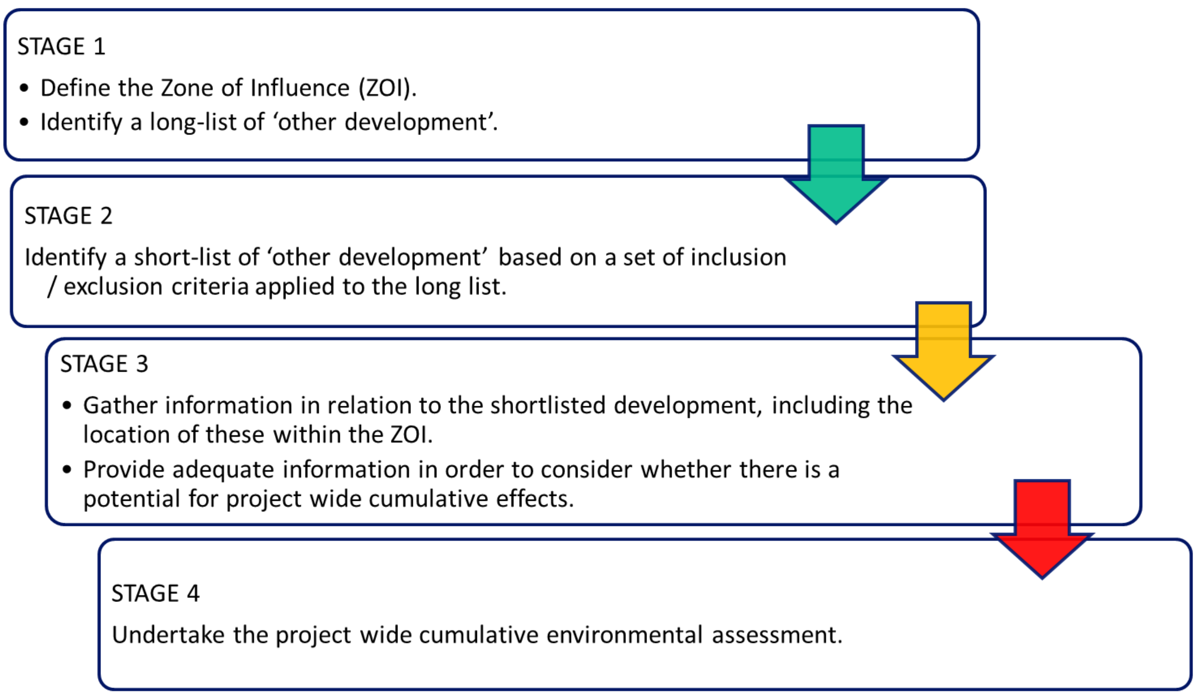

Figure 64: Stages for the project wide cumulative environmental assessment.

5.2 Sizewell C project components (for inter-relationships) and 3rd party plans – programmes – projects (for cumulative effects)

5.2.1 Sizewell C project components for inter-relationship impacts

The Sizewell C marine / coastal project components are described in Sections 3 and 4 and shown in Table 22. It should be noted that the HCDF is considered as a terrestrial component of the MDS (see Section 4), and thus not included for the CEA of coastal geomorphology and hydrodynamics. However, the case of the HCDF becoming exposed, and thus a marine component, is addressed and discussed in detail in Section 7.

5.2.2 3rd party plans – programmes – projects for cumulative impacts

In order to gather information for this category the MMO's public register, the MMO's ArcGIS Online tool Marine Information System (MIS) and the Planning Inspectorate's National Infrastructure Planning registry were used to identify third party plans, programmes and projects for potential cumulative impacts. The plans and projects within the ZoI that have potential cumulative effects are shown in Table 23.

5.3 Temporal – spatial overlap

At the time of writing this report, the schedule for project wide CEA (within project activities and 3rd party projects) is as shown in Table 24. In Table 25, the list of potential impacts (inter-relationship and 3rd party) per temporal combination of activities is presented.

Table 22: Building and using components of individual features of the proposed development.

Feature	Building Components	Using Components
BLF	Terrestrial pilling vehicle (beach/nearshore) and vehicle traffic	Pile Scour
		Effect of deck piles
	Insertion of marine piles	BLF in use
	Vessel / jack-up barge anchoring	Nearshore dredging on the BLF approach
		BLF infilling rates
		BLF barge docking
Cooling Water Intakes and Outfalls	Intake and outfall drilling	Scour around intake structures
	Intake and outfall dredging	Scour around outfall structures
	Vessel anchoring	Scour protection around intake and outfall structures
Nearshore Outfalls	Dredging	Scour protection around FRR and CDO structures
	Vessel anchoring	Scour around FRR and CDO structures
SCDF	Additional sediment on beach	Additional sediment on beach during storms

Table 23: 3rd party plans – programmes – projects within the ZoI for coastal geomorphology

Project, plan or programme	Location	Timeline
Scottish Power Renewables onshore and offshore facilities for East Anglia One North and East Anglia Two (Tier 3), comprising: Onshore: • two substations (total 20-30ha) • one National Grid compound • temporary construction compound Offshore: • cables to connect to offshore windfarm	Onshore facilities: Friston area or Sizewell Gap (in close proximity to existing Galloper substation) – final decision on location in 2019.	Construction commencing 2024 Operational late 2027.
National Grid interconnectors (Tier 3), comprising: Onshore: • converter stations (to 5ha) • cable landfalls of 200m width Offshore: • 'Nautilus' interconnector cables • Eurolink' interconnector cables	Location to be identified by 2020 (Eurolink) and 2022 (Nautilus).	Construction periods of 2023-2024 (Eurolink) and 2025 - 2027 (Nautilus). Both interconnectors operational at end of 2027.
Shoreline Management Plan (SMP) 7 (dated 2010) Lowestoft Ness to Felixstowe Landguard Point	Suffolk SMP2 Sub-cell 3c Policy Development Zone 4. Dunwich Cliffs to Thorpeness.	Full list of the SMPs for the ZoI can be found in Table 4.
Royal Society for the Protection of Birds (RSPB) Minsmere coastal change strategy.	Minsmere frontage (four named units within the SMP).	Managed realignment of shoreline over 0-100 years, although large scale realignment not anticipated for 50-100 years.

Table 24: Schedule for 3rd party projects, plans and within the project activities within the ZoI (note: St 1 / St 2 indicate first and second semesters – 6 month periods - of each year)

			2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036
						Peak Early Years					Peak MDS Construction					Operation	
	Year		2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036
Third Party	Scottish Power East Anglia One North and Two					Construction						Operational					
	National Grid (Nautilus)						Construction					Operational					
	National Grid (Eurolink)				Construction							Operational					
	SMP 7 (2010)																
	RSPB Minsmere coastal change strategy																
	Temporal Group		1	2	3 / 4	5	6 / 7	8 / 9	10				11				
			2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036
		ACTIVITY COMPONENTS	Year 0	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9	Year 10	Year 11	Year 12	Year 13	Year 14
			St 1 / St 2	St 1 / St 2	St 1 / St 2	St 1 / St 2	St 1 / St 2	St 1 / St 2	St 1 / St 2	St 1 / St 2	St 1 / St 2	St 1 / St 2	St 1 / St 2	St 1 / St 2	St 1 / St 2	St 1 / St 2	St 1 / St 2
Building Components	BLF	Terrestrial Pilling Vehicle (Beach/Nearshore) and vehicle traffic for Temporary rock jetty installation and dismantling.															
		Insertion of marine piles: pile footprints.															
		Vessel / jack-up barge anchoring: interaction with the seabed.															
	Cooling Water Intakes and Outfalls	Intake and outfall drilling plumes: seabed sedimentation.															
		Intake and outfall dredging: seabed sedimentation.															
		Vessel anchoring intakes/outfalls: interaction with the seabed.															
	Nearshore Cooling Water Infrastructure	Nearshore cooling water infrastructure dredging plumes: seabed sedimentation.		CDO installation			Years 4.75 - 5.25 FRR1 installation		Years 6.5 - 7.0 FRR2 installation								
		Vessel anchoring CDO/FRRs: interaction with the seabed.															
	SCDF	Construction of SCDF (NE section and main section) - additional sediment placed on beach		Years 1.0 - 1.25 for SCDF in front of NE section	Years 2.0 - 2.25 for SCDF in front of main section												
Using Components	BLF	BLF pile Scour															
		Effect of BLF Jetty Piles on hydrodynamics															
		Effect of BLF in use (reprofiled bed + piles) on hydrodynamics															
		Nearshore reprofiling of the BLF approach: capital dredging plumes: seabed sedimentation															
		Nearshore reprofiling of the BLF approach: maintenance dredging plumes: seabed sedimentation								Maintenance Dredging (after Year 10) once every 5-10 years thereafter (duration <4 weeks)							
		Dredged areas for BLF approach and subsequent infilling															
		Effect of docked barge at the BLF (with reprofiled bed + piles) on hydrodynamics															
	Cooling Water Intakes and Outfalls	Scour around intake structures															
		Scour around outfall structures										EPR1 (c/w intake 1 & outfall) testing/in use from Year 5.25 onwards					
		Jet scour at outfall structures										EPR2 (c/w intake 1 & outfall) testing/in use from Year 7 onwards					
		Scour protection around intake and outfall structures															
	Nearshore outfalls	Scour protection around CDO structure		CDO													
		Scour protection around FRR1 structure					FRR1										
		Scour protection around FRR2 structure							FRR2				EPR1 (FRR1) testing/in use from Year 5.25 onwards				
		Scour around CDO structure		CDO									EPR2 (FRR2) testing/in use from Year 7 onwards				
		Scour around FRR1 structure					FRR1										
		Scour around FRR2 structure							FRR2								
	sCDF	Operation of SCDF (NE section) - additional sediment on beach during storms		NE section in use from year 1.25													
		Operation of SCDF (main section) - additional sediment on beach during storms			Main section in use from year 2.5												

Table 25: Inter-relationship and cumulative impacts

Year	Inter-relationship impacts	Cumulative impacts
Year 2022.	All building components for the BLF (as given in **Section 5.2.1**), along with BLF pile scour and BLF piles on the seabed.	Not applicable.
Year 2023.	All activity components for building the BLF construction phase, along with Combined Drainage Outfall (CDO) head installation and operation, building and using the SCDF in the north east section, BLF pile scour and effects of BLF piles on the bed.	From the construction of cable corridors for the Eurolink interconnector.
Year 2024, first semester.	All activity components for building the BLF, along with building the SCDF in front of the main HCDF section (south of the BLF), any activity component for using the BLF, CDO and north east section of the SCDF, including capital and maintenance dredging of BLF approach.	From the construction of cable corridors for the Eurolink interconnector.
Year 2024, second semester.	All activity components for building the BLF, along with building the SCDF in front of the main section, any activity component for using the BLF, CDO and SCDF, including maintenance dredging of BLF approach.	From the construction of cable corridors for the Eurolink interconnector and Scottish Power East Anglia One North and Two.
Year 2025.	All activity components for building the cooling water intakes and outfalls (except for tunnelling, which is subterranean and therefore not a marine activity), any activity component for using the BLF, CDO and SCDF, scour/scour protection at the intakes and outfalls, including maintenance dredging of BLF approach.	From the construction of the Nautilus interconnector and of the cable corridors for Scottish Power East Anglia One North and Two.
Year 2026, first semester.	All activity components for building the cooling water intakes and outfalls, any activity component using the BLF/SCDF/CDO, scour/scour protection at the intakes and outfalls.	From the construction of the Nautilus interconnector and of the cable corridors for the Scottish Power East Anglia One North and Two.
Year 2026, second semester.	All activity components for building the cooling water intakes and outfalls, any activity for building Fish Recovery and Return unit 1 (FRR1), any activity component for using the BLF/SCDF/CDO/FRR1, scour/scour protection at the intakes and outfalls, including maintenance dredging of BLF approach.	From the construction of the Nautilus interconnector and of the cable corridors for the Scottish Power East Anglia One North and Two.
Year 2027, first semester.	All activity components for building the cooling water intakes and outfalls, any activity component for using the BLF/SCDF/CDO, including maintenance dredging of BLF approach, any activity for building FRR1, any activity component for using EPR1 and FRR1.	From the construction of the Nautilus interconnector and of the cable corridors for the Scottish Power East Anglia One North and Two.
Year 2027, second semester.	All activity components for building the nearshore cooling water intakes and outfalls, any activity component for using the BLF/SCDF/CDO, including maintenance dredging of BLF approach, any activity component for using EPR1 and FRR1.	From the construction of the Nautilus interconnector and the operation of the Scottish Power East Anglia One North and Two.
Year 2028, first semester.	All activity components for building the nearshore cooling water intakes and outfalls, any activity component for using the BLF/SCDF/CDO, including maintenance dredging of BLF approach, any activity component for using EPR1 and FRR1.	From the operation of Nautilus and Eurolink interconnectors and the operation of the Scottish Power East Anglia One North and Two.
Year 2028, second semester.	All activity components for building the cooling water intakes and outfalls, any activity for building FRR2, any activity component for using the BLF/SCDF/CDO, including maintenance dredging of BLF approach, any activity component for using EPR1 and FRR1.	From the operation of Nautilus and Eurolink interconnectors and the operation of the Scottish Power East Anglia One North and Two.
Year 2029 and onwards.	Any activity component for using the BLF/SCDF/CDO, including maintenance dredging of BLF approach, any activity component for using EPR1, EPR2, FRR1 and FRR2.	From the operation of the Nautilus and the Eurolink interconnectors and the operation of the Scottish Power East Anglia One North and Two.

5.3.1 Limitations for the temporal and spatial overlap

Limitations affecting the identification of temporal and spatial overlaps are listed below. The steps taken to address each limitation, ensuring that a reasonable and conservative approach is used for the CEA, are also detailed.

- **Third party plan extents and timescales:** for the impact assessment, the SMPs short-term and medium-term epochs are considered to cover the construction and the operational phases of Sizewell C. For the spatial overlap of the SMPs, all the policies will be considered according to the spatial extent of each sub-section.
- **Third party project extents**: complete details of the construction and operation activities of the 3rd party projects were not available at the time of writing and thus each project's red line boundary was used to represent its spatial footprint for establishing spatial overlap – this is conservative and is very likely to lead to intersections that would not occur. For example, in the case of the interconnectors we assume that their coastal boundary will be the same as the East Anglia One North and Two projects.
- **Within project activity length:** activities are considered to occur continuously inside of their scheduled window, even though in most cases they will only use a fraction of the time**,** therefore the schedule shown in Table 24 conservatively indicates longer durations than activities would actually need. For example, the construction of the BLF is given a continuous three-year CEA construction window (Table 24), when the expected duration of works would be only six months. The approach minimises the chance of a change in project schedule invalidating the CEA.
- **Within project effect length**: the schedule conservatively extends the duration associated with each activity to account for residual effects.
- **Within project effect magnitude**: where effects were expected to be very small, they were still considered. For example, sedimentation thicknesses greater than 20 mm from plumes are very conservatively considered significant for coastal geomorphology and for consistency with CEA analysis done for Marine Ecology and Water Quality.
- **Within project activity extents**: spatial buffers delineating the extent of an activity (and associated impact) are uncertain and so are defined conservatively. For example:
    - a 100 m buffer is used for anchoring at the nearshore and offshore intakes and outfalls.
    - a 50 m buffer is used for anchoring and vehicle impacts for the BLF building phase.
    - a 10 m buffer is used for the construction zone for building the SCDF.
    - the boundary between the north-east and main sections of the SCDF is arbitrarily drawn.
    - no spatial footprint is assigned for elevated Suspended Sediment Concentration (SSC) or sedimentation from sediment plumes generated during the insertion of BLF piles as these are considered to be very small.

In order to visualise the spatial and temporal overlap of 3rd party plans and inter-relationship activities, the Time Slider tool in ArcGIS was used. It should be noted that individual effect layers do not necessarily represent an effect to the geomorphic receptors. For example, 5% change of bed shear stress (see Figure 65 and Figure 66) does not necessarily imply an effect on geomorphology, but it is included as it may combine with another activity to make a larger effect. Areas with spatial and temporal overlap are assessed according to the risk of impact on the coastal geomorphology receptors in the Section 20.11 of the ES.

A mapped example demonstrating one time-step in the cumulative environmental assessment is given in Figure 65 and Figure 66. This example checks for the spatial overlap of the temporal group for the 1st semester of 2027. An initial conclusion of this map is that the plume from the dredging / drilling of the offshore intakes and outfalls does not spatially overlap with the dredging and drilling for the CDO and FRRs or with the spatial footprints of the East Anglia One North and Two and the interconnectors Eurolink / Nautilus.

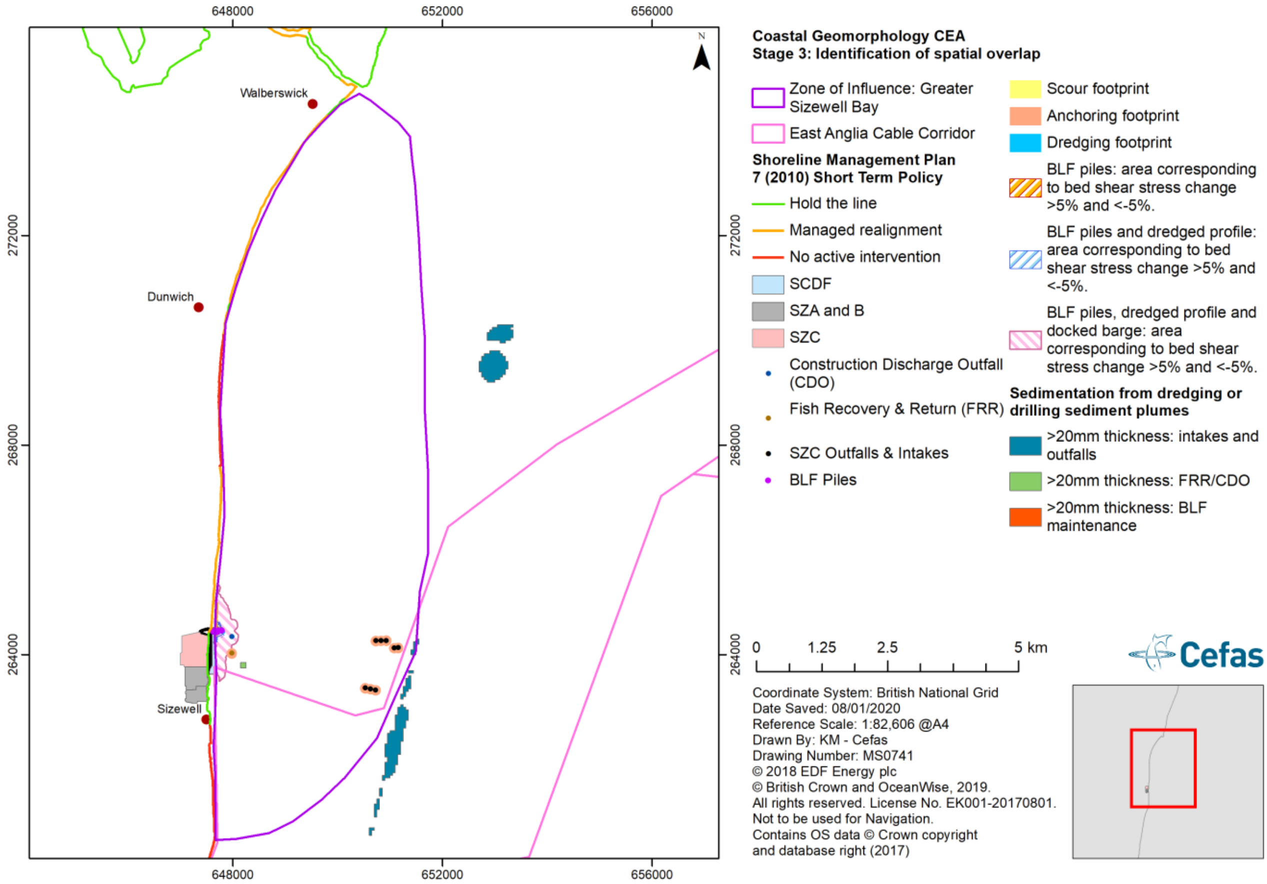

Figure 65: Map of temporal and spatial overlapping activities within the project and 3rd party projects – plans in Semester 1, 2027.

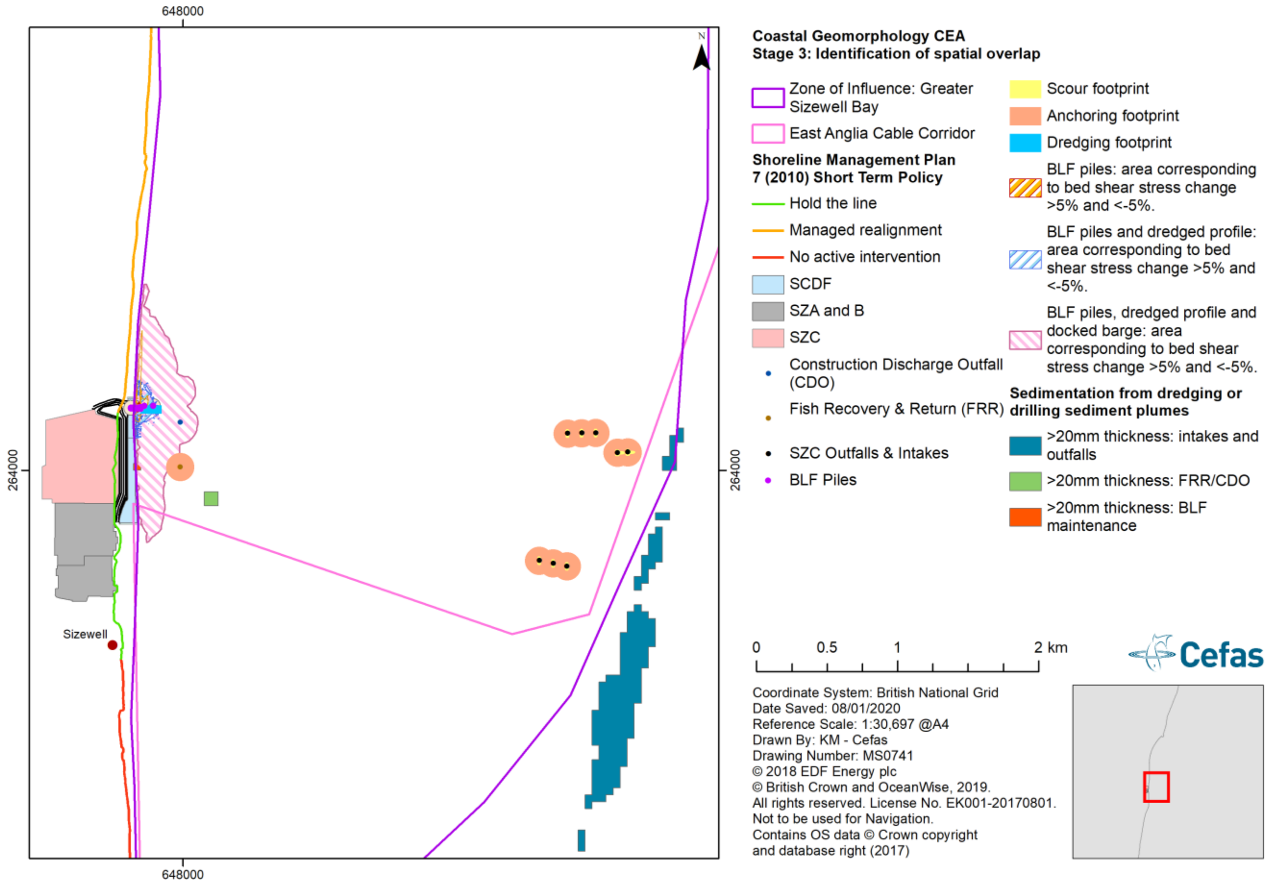

Figure 66: Map of temporal and spatial overlapping activities within the project and 3rd party projects (close up of Sizewell C station) – plans in Semester 1, 2027.

6 High level monitoring and mitigation

Detailed monitoring and mitigation plans would be developed in accordance with any conditions attached to an approved DCO and the associated Deemed Marine Licence (DML). The monitoring and mitigation plans follow the DCO/DML because the predicted impacts and their significance need to first be determined and agreed, as was the case for HPC. If approved, the DCO would contain requirements that form the basis of the monitoring and mitigation plans – activities affecting the coast will not be able to commence until these plans are approved by the MMO, following consultation and agreement with relevant stakeholders . This section provides an indication (high-level description) of the likely monitoring and mitigation expected to be undertaken during the construction and operation of Sizewell C. Monitoring and mitigation are not considered for activities that are assessed in the Environmental Statement to have minor or negligible effects. It also does not consider the HCDF, which would be a terrestrial feature during construction and the early – middle Sizewell C operation phases, however, HCDF effects on the future coastal and marine environment are considered in Section 7, along with future monitoring and mitigation considerations.

The broad details provided in this section are the foundations for monitoring plans, but they would be subject to change based on DCO requirements, final engineering design and final environmental evidence on effects (as contained in the DCO application).

6.1 Offshore cooling water infrastructure

Although scour from offshore cooling water infrastructure is predicted to have negligible effects on coastal geomorphology receptors, it is standard practice to monitor the scour around structures mounted on the seabed to check that the predictions are correct as part of routine asset integrity inspections. Scour may also affect marine ecology receptors. Additionally, monitoring methods would detect any changes caused in construction, such as depressions from jack-up barge legs.

High-resolution swath bathymetric surveys would be conducted over the predicted scour footprint (including a buffer to ensure the entire scour footprint is fully mapped). A pre-construction survey conducted one to three months prior to the commencement of works would be followed by two post-construction surveys, three and six months after works completion. The timing of these surveys would allow scour to develop to an equilibrium state (three months), and confirmation with the follow-up check (six months).

The equations are designed to show a worst-case. No mitigation is proposed since the scour would not affect geomorphic receptors. Scour protection would be considered as part of the detailed designs of these seismically qualified infrastructure elements, which may reduce scour depth but may also cause a greater impact (for ecology) due to the change in substrate.

6.2 Nearshore outfalls (CDO and FRRs)

Due to their proximity to the longshore bar and shoreline receptors, scour and additional monitoring of Sizewell C's nearshore outfalls would be undertaken. The additional monitoring would be conducted because there is a low chance that the CDO and FRRs could alter the shape of the bar and shoreline receptors in a similar fashion to that observed at Sizewell B's outfall[14]. The monitoring is considered precautionary because the Sizewell C structures are smaller (in physical size and discharges over 100 times

[14] The Sizewell B outfall appears to prevent the longshore bar from shoreward migration (whereas elsewhere shoreward migration has been observed over the last 25 years), which may be the cause of shoreline advance in its lee that began 10 years after Sizewell B operation commenced.

smaller) and less likely to affect the outer longshore bar due to their location on its seaward flank (whereas Sizewell B's outfall is large and inshore of the outer bar). The monitoring would also detect any construction-related changes such as depressions from jack-up barge legs.

Several methods could be considered for monitoring the shoreline and longshore bars, including:

- Video and/or radar for shoreline position; these methods provide high frequency data ideal for early warning detection of impacts (daily – weekly).
- Video and/or radar for barline detection using the intensity maxima from the surf zone; this method provides high frequency data when waves are breaking on bars, but none when waves are absent.
- Ground, drone or LiDAR surveys for beach topography; these complementary methods cannot be conducted continuously, but do allow a full assessment of elevation and substrate changes in the intertidal and supra-tidal beach contours and volumes.
- Shallow water bathymetry from vessel mounted multibeam or interferometric systems, or drone/radar-based methods for computing water depth from wave parameters and wave theory.

A pre-construction bathymetric survey would be conducted one to three months prior to the commencement of works. This would then be followed by three post-construction surveys at three-, six- and twelve-months following completion. The extra survey (compared to scour monitoring) would be warranted because of the potential effects to the longshore bar and shoreline receptors. The timing of these surveys would ensure a year of data had been collected, including observations over a winter period. If there is a long separation between construction of the CDO (expected early in the construction phase) and the FRRs, this monitoring may need to be undertaken twice, following the CDO installation and then again following the FRR installations.

Longer term monitoring of the shoreline and bar receptors would be conducted but with decreasing frequency (for manually intensive methods) if the monitoring evidence showed no or minor effects.

6.3 Impacts of BLF piles (BLF not in use) on the inner bar and shoreline receptors

The BLF piles are transmissive and not expected to block sediment transport, however localised scour is predicted. Over very short distances (tens of metres) this could have a minor effect on the shoreline position and / or the inner longshore bar for short durations (i.e., during and following storms).

The methods that could be used to monitor the shoreline and inner bar are the same as those listed in Section 6.2.

A pre-construction bathymetric/topographic survey would be conducted one to three months prior to the commencement of marine BLF deck works. The effect of the presence of the BLF deck piles alone would require a period of six months when the facility is not in use. Once this period has occurred, the effects of the BLF piles (scour) would be assessed using bathymetric monitoring conducted three and six months into the phase of no-BLF-use.

No mitigation is planned, as the effects would be very localised and would have no significant effect on coastal geomorphology receptors.

6.4 BLF in-use impacts (BLF piles and dredge reprofiling) on the bar and shoreline receptors

During BLF use, the effects to the local seabed would be due to the presence of the deck piles and any reprofiling necessary for navigation and docking. The BLF deck approach and the grounding pocket for barge docking would be reprofiled using a plough dredger (Figure 31). There would be an operational requirement to monitor for clearance, navigational safety and impact detection. The latter would require a wider footprint than the dredged area to ensure any changes to the adjacent longshore bar and shoreline receptors would be detected.

The methods that could be used to monitor the shoreline and longshore bars are the same as those listed in Section 6.2.

Surveys for impact detection would initially be undertaken before and after each dredge, but the monitoring schedule would be changed (subject to approval) if the effects were localised in extent and having no significant effect on receptors (as predicted). As the effects are predicted to be localised and no sediment is removed from the system, no mitigation is expected.

6.5 Soft Coastal Defence Feature

The SCDF is a sacrificial, embedded mitigation feature of Sizewell C. During large storms and high-water levels, the feature is expected to erode along its seaward margin, episodically releasing shingle to the beach face. An analogy for the release of SCDF sediments was the erosion of the current sacrificial 5-m shingle bund[15] during Storm Emma (March 2018): 1 m^3/m of the bund was eroded, which if spread evenly over the beach below the level of the Highest Astronomical Tide (HAT) would equate to a 3.5 cm thick deposit. The exact cross-shore pattern of erosion and accretion cannot be predicted and is expected to be spatially variable (see Section 2.3.6), however some beach sections have shown classic cut and fill patterns (BEEMS Scientific Position Paper SPP094).

As erosion can be patchy, the monitoring would need to be spatially continuous. This would be achieved using a combination of methods:

Method (1). An early warning system that would sample frequently and detect change due to individual storm events. This monitoring system would utilise real-time tide and wave data with radar/video cameras to detect changes in the edge of the vegetated surface of the SCDF and the shoreline position. The volume lost from the SCDF following each storm would be estimated by removing the eroded section of the SCDF (as defined by the edge of vegetation) from a 3D digital model of the SCDF and beach surface. Likewise, the volume of the beach would be empirically estimated using the relationship between the beach width (on each section) and volume, which would be determined using method (2) below.

Method (2). A more accurate volumetric method would be used periodically and for rare events, as triggered by method (1) and wave/tide monitoring. The volumetric survey would be conducted using drones, LiDAR and/or a very dense ground survey. These methods are spatially continuous and therefore allow the beach volume and the SCDF volume to be more accurately estimated along the coast. The material released from the SCDF, and its timing, would then be monitored through time to document the volumetric contribution of the SCDF to the coastal system. Regular monthly volumetric surveys would be conducted during construction phases that directly affect the beach, and quarterly thereafter. This volumetric monitoring would decrease in frequency following construction, based on the evidence gathered. Any changes to the monitoring frequency would require approval by the MMO.

[15] The 5-m shingle bund is a feature constructed as part of the Sizewell B development. It is effectively the same type of feature as the SCDF, albeit smaller and only occupying a single ridge.

7 Future shoreline baseline, monitoring, pre-emptive mitigation and post-mitigation impacts

7.1 Introduction

The hard Coastal Defence Feature (HCDF) has been deliberately set back from the coast, but shoreline recession is likely to expose it and cause additional impacts not considered in the Section 4 evidence base. This section is the evidence base for a shifted future shoreline baseline, its associated future impacts and initial monitoring and mitigation guidance to inform the monitoring and mitigation plan that would be developed as a requirement of any approved DCO/DML. It contains the evidence upon which Section 20.14 of the Coastal Geomorphology and Hydrodynamics ES chapter – *Future shoreline baseline, pre-emptive mitigation and potential post-mitigation impacts* – is based.

The rationale behind the definition and projection of a likely future shoreline baseline during the operational phase of Sizewell C is set out in BEEMS Technical Report TR403. Its objectives were to determine:

- whether the shoreline is likely to erode and expose the HCDF (a scenario without Additional Mitigation (also referred to as Secondary Mitigation));
- a plausible future shoreline baseline (without Sizewell C); and
- a plausible future shoreline with Sizewell C, highlighting the likely effects.

The future baseline scenario without Sizewell C features a shoreline that has eroded to the planned position of the HCDF, as marked in Figure 28, Figure 30 and Figure 71. It was determined using an Expert Geomorphological Assessment (EGA) (Section 7.2) and includes a plausible time window for its occurrence of 2053 – 2087 (Section 7.3). During this timeframe, the basic driving coastal processes are not expected to change substantially (i.e., there would be no regime shift). The future shoreline over the same timescale with Sizewell C and the HCDF was also determined using EGA (Section 7.4). The main difference between the two future shorelines is the introduction of additional beach shingle from the eroding SCDF in the 'with Sizewell C' scenario. The impact of this additional shingle would be to reduce erosion rates near the SDCF and delay the natural onset of breaching and roll back. The potential impacts that would arise if the HCDF were exposed following depletion of the SCDF are also discussed in Section 7.4.

An exposed HCDF could disrupt, and eventually block, shingle transport, leading to potential downdrift erosion (see Section 7.4.2). Hence, in order to avoid this eventuality:

- There would be a need for Additional Mitigation (beach management) to prevent exposure of the HCDF and disruption to longshore sediment transport. The potential mitigation methods are described in Section 7.5;
- The trigger to initiate mitigation action is described in Section 7.6, along with a flow chart illustrating the conditions under which mitigation would cease; and;
- Section 7.7 considers two endmember geomorphic configurations and the potential effects that may arise as a result of mitigation cessation and an exposed HCDF. It is not possible to determine the impacts and their significance at this time – that will be done using the future evidence base – and so the potential impacts described are generalised, not for impact assessment and intended to be evolved with coastal change over the next century or more.

Although EGA shows HCDF exposure is very likely, there are some circumstances in which it may not occur, which would avoid the need for any pre-emptive mitigation, maintain the longshore shingle transport corridor, avoid downdrift starvation and avoid a potential significant impact:

- Despite shoreline recession, a perched beach may be naturally sustained at the foot of the HCDF, especially where there are protrusions in the HCDF.
- Increasing sediment supply from local and regional sources may lead to a wide beach, especially if the sheltering effect of Sizewell Bank is maintained by its growth with sea level rise.
- Although there are no predications for a change in the directional wave climate, an increase in SE storm energy would slow or stop retreat on the Sizewell C frontage. Historical evidence also points to sediment accumulation and shoreline advance under a dominant NE wave climate.

7.2 Expert Geomorphological Assessment for future shoreline baseline and hypothetical HCDF impacts in the absence of Additional Mitigation

Shoreline change is driven by several factors whose importance and interaction several decades into the future cannot be accurately predicted (Nichols *et al*., 2012), either separately or in combination. Moreover, there is no current computational modelling platform able to accurately integrate the numerous environmental processes that drive shoreline change (especially for mixed gravel/sand beaches), and there is no published evidence that shoreline change models can be reliably applied over the required multi-decadal timescale. For example, the shoreline change modelling originally carried out to investigate the impacts of an early-design solid BLF block within the active beach[16] (BEEMS Technical Report TR329) was based on the assumption that alongshore variations in modelled longshore transport result in net loss or gain of material and, thus, shoreline advance (accretion) or retreat (erosion). This is a simplification of the processes driving shoreline change and, for future predictions, relies upon unattainable *a priori* knowledge of the:

- driving wave climate (wave angle and energy (storms, storm sequences, seasonal to decadal patterns) at all points along the coast);
- antecedent beach morphology and spatial variability within the gravel-sand beach matrix;
- cross-shore fluxes of sand that are controlled (trapping and release) by the spatially (in three dimensions) and temporally varying ratios of the mixed shingle/sand matrix, and;
- numerous other unpredictable factors, including coincidence of storm surges with high wave energy and high tides, the rate of sea level rise, when and where breaching would occur, changes in shingle supply through exposure of the Dunwich Cliffs, the rate of increase in sand supply and geomorphic feed-back loops and interactions.

The necessary alternative is to undertake an Expert Geomorphological Assessment (EGA) that takes into account all available strands of evidence, acknowledging the limitations and uncertainties that each has (including the timescale over which the evidence would remain valid), to arrive at a plausible future shoreline baseline against which plausible worst-case impacts could be identified. In order to develop a robust assessment for Sizewell, a group-EGA was undertaken as described in the following sections to develop the plausible future shoreline baseline. For additional detail, the reader is referred to BEEMS Technical Report TR403.

7.2.1 Method

Seven Expert Geomorphologists[17], internal and external to Cefas, were convened to assess the physical and scientific evidence for shoreline change processes and to derive a plausible future shoreline baseline using the EGA approach. A two-stage method was developed. Stage 1 was to extrapolate the present shoreline movement trends (using a statistically validated linear trend from bi-annual measurements during 1992-

[16] This was an early design feature that is no longer part of the Sizewell C engineering designs

[17] Dr Steven Wallbridge, Dr Helene Burningham, Emeritus Professor Chris Vincent, Dr Tony Dolphin, Dr Ralph Brayne, Professor Ken Pye and Dr David Brew.

2018) 50-years into the future. The 50-year (2070) timeframe represents a plausible timeframe sufficient to expose the HCDF, based on the shoreline trends. The Stage 1 projections along the coast were then used by the EGA (Stage 2) to agree the anticipated changes in environmental driving forces and the likely shoreline response, and to derive future shorelines without and with Sizewell C that broadly capture how the development could most significantly alter natural processes (the ‘worst-case impact’).

7.2.2 Outcomes of group Expert Geomorphological Assessment (EGA)

The group EGA identified several pivotal factors that would control the future shoreline evolution. Based on the present state of knowledge and evidence, it reached a consensus on the anticipated future states of these factors which would maximise the impact of the proposed Sizewell C (Table 26). The combination of these factors would then establish the development of a shoreline evolution model, from which the effects caused by the proposed Sizewell C could then be estimated. It was broadly considered that the worst-case impacts would arise when the HCDF first begins to affect coastal processes – further into the future, as the degree of coastal change increases, the influence of Sizewell C on those changes was considered likely to decline.

Table 26: Summary of the EGA consensus regarding factors affecting the modification of the Stage 1 linear extrapolation and the EGA future shoreline projection (Stage 2).

No.	Agreed Principles
1	To adopt a future projection based on “reasonably foreseeable” conditions.
2	Sea level rise in the year 2070 would be 0.52 m relative to 1990 levels (UKCP18, see Section 2.4.1 and Palmer *et al*, 2018).
3	Extrapolation of the observed 1991-2018 SLR trend accounts for 68% of the UKCP18 SLR prediction at 2070, which implies that the observed shoreline response already includes a significant element of shoreline response due to SLR. Accordingly, an additional sea level rise of 0.17 m (the UKCP18 prediction) is considered in determining the shoreline response at 2070.
4	The offshore wave climate remains unchanged (UKCP18 indicates small reductions in mean and annual maximum significant wave height)
5	The inshore wave climate remains unchanged.
6	Minsmere Outfall remains physically in place until the sluice is no longer a functional element of Minsmere Levels drainage (due to SLR).
7	No shoreline accretion, and shoreline sinuosity remains similar to that at present.
8	No change in the ‘Hold the Line’ status for Blyth river jetties, as per the SMP across all three epochs.

Based on the information presented in Table 26, the following key elements were determined in driving the future shoreline:

- A projection based on the ‘reasonably foreseeable’ conditions was considered the most appropriate method of reaching consensus as ‘extreme events’ that could occur have a low (or poorly-determined) chance of occurrence and geomorphic systems tend to be shaped by more frequent moderate events (Wolman and Miller, 1960), with the exception of cataclysmic change. The difference between ‘with Sizewell C’ and ‘no-Sizewell C’ scenarios was considered likely to be much smaller for a radically-altered coastline than for the EGA-derived projection.

- The consensus view was that the ‘natural’ future shoreline was likely to be no more sinuous than it is today and that net accretion relative to the present was very unlikely under rising sea levels, even at locations with currently accreting trends (e.g., Minsmere Cliffs, around Minsmere Outfall and adjacent to the Sizewell B Outfall). Simply extrapolating the rate of sea level rise observed during 1992-2018 to 2070 would account for around 65% of the projected rise of 0.54 m under the RCP 8.5 UKCP18 projection; Figure 21 and Palmer *et al*, 2018). Therefore, extrapolating the shoreline change trends defined over this period to 2070 implicitly contains the shoreline response to this fraction of the projected sea level rise. The effect of the additional 35% (0.19 m) was calculated to be a further 9.5 – 19 m of horizontal shoreline retreat (BEEMS Technical Report TR403). Higher rates of sea level rise pose a greater threat to designated sites and the environmental processes which would be affected by the proposed Sizewell C and relevant rates of SLR were considered to engender the potential worst-case impacts.
- Present beach/barrier levels along the majority of the shoreline are too high for extensive overtopping or breaching to occur, even during very large storm surges (except for on the Northern Barrier). Therefore, erosion of the shingle barrier would be by scarping until, in places, the profile becomes low enough to be overtopped, after which episodic overtopping or breaching and roll-back would begin. Permanent inlets are unlikely to form due to the small potential tidal prism and the high entrainment threshold of the gravel beach sediment. Inlets, as a potential interruption to longshore transport pathways, would also relatively reduce the significance of the Sizewell C development and so are not considered to contribute to the ‘worst-case impact’. Any breaches would be temporary, closing up shortly after breaching, which is typical behaviour at neighbouring breach sites on the Suffolk coast (e.g., Covehithe Broad, Easton Broad and the Dunwich-Walberswick coast). The transition to roll-back within the 50-year period is expected on the Northern Barrier (where overtopping has already occurred and erosion is presently up to 2 m per year) and is probable on the Southern Barrier near Sizewell C site for the no Sizewell C scenario (present net rate of retreat 0.5 – 1.4 m per year).
- Net longshore sand transport along the nearshore bars is southward toward Thorpeness. Rates between Sizewell Hall and Thorpeness are particularly low, which may lead to accumulation, acting counter to shoreline retreat under rising sea levels (i.e., stable shorelines). There is no evidence of net cross-shore shingle transport, as beach volumes remain relatively stable on an annual timescale (BEEMS Technical Report TR223 Edition 3) and beach shingle is almost entirely absent in the nearshore subtidal. Sediment supply (sand and shingle) is not expected to constrain shoreline development (e.g., due to cliff erosion and supply to the north of the study region; Brooks *et a*l., 2012).
- UKCP18 climate projections suggest decreases in the seasonal mean and extreme waves in the North Sea (Palmer *et al*, 2018 and as noted in Section 2.4.2).
- The Sizewell-Dunwich Bank affects the inshore wave climate, especially when waves are sufficiently large that they break on the bank (BEEMS Technical Reports TR058 and TR319); bathymetric surveys undertaken between 2007 and 2017 showed that Sizewell Bank has remained in a relatively stable position, likely to be due to the erosion-resistant Coralline Crag, which outcrops as sub-tidal ridges beneath the bank and provides an anchoring effect.
- In comparison, the Dunwich Bank has no inherited stabilising hard geology (i.e., no headland or underpinning crag) and showed greater variability over the same period: migration landward of the landward flank of the bank by 50 m and by 200 – 400 m on the seaward flank, whilst substantial lowering (1-2 m) occurred across approximately 10% of its area. Reductions in Dunwich Bank are not considered to be a worst-case scenario for Sizewell C as they would eventually lead to cliff erosion and increased sediment supply, minimising the chance or degree of exposure of the HCDF (or the amount of mitigation required to prevent this).

- The work (physics) required to migrate the entire bank mass a significant distance would require long time periods (to integrate enough storm energy); however, the inner and outer longshore bars are smaller and shallower, and are consequently relatively mobile features that would change their positions relatively quickly in response to sea level rise. The inferred combined effect is that the nearshore propagation of waves would remain similar to the present as sea level rises. The EGA therefore assumed no significant change in the nearshore profile and hydrodynamics at Sizewell over the assessment period.
- The EGA identified continued net southward transport under a similar hydrodynamic regime to the present as the likely cause of worst-case impacts from the proposed Sizewell C. Due to the shape of the HCDF and the prior existence of both Sizewell A and Sizewell B platforms, the highly unlikely reversal of the principal sediment transport direction (and associated geomorphological changes) would lead to less differentiation between the 'no-Sizewell C' and 'with-Sizewell C' future, relative to the (more probable) continuation of present patterns.
- The reinforced Minsmere Outfall is considered to have had a significant role in anchoring the shoreline immediately adjacent to the outfall structure by trapping shingle moving north and south during storms, resulting in the formation of a promontory and accretion observed over c. 500 m of frontage (between 265915N and 266386N; Minsmere Outfall is located at 266120N). The outfall increasingly holds the shoreline away from its 'preferred' north-south drift alignment, defining and reinforcing the small areas of recent (30-year) trends of shoreline retreat on its northern and southern sides. The EA are presently obligated to maintain the drainage function of the sluice until it is no longer effective[18], which, practically, may in turn involve maintenance of the outfall across the future baseline assessment period. Hence, the EGA considered that shoreline change at and around the outfall, would continue at its present rate or greater, due to the continued presence of the outfall. This in-combination with the changes that would arise from the proposed Sizewell C, would generate the 'worst case impacts'.

7.3 Future shoreline baseline (without Sizewell C)

The EGA projected shorelines (mean sea level (MSL)) without Sizewell C are shown Figure 67 for the time-horizon when the shoreline would reach the position of a hypothetical (in the no Sizewell C scenario) HCDF. Assessment of uncertainty in the methodology, and of relevant assumptions, suggests that this shoreline would most likely reach the proposed HCDF location during the period 2053 – 2082.

The present-day stability and accretion from Dunwich to around 1.5 km north of the Minsmere Outfall led the EGA to project minimal shoreline change in this area (although change and cliff erosion could occur, this would not produce a worst case scenario for HCDF exposure/impacts). South of this area to just north of the outfall (the Northern Barrier), rapid erosion was projected to continue at its present rates, the barrier line to be eroded by more than 60 m and the beach to enter roll-back within 30 years. The position of the shoreline at the outfall would be maintained, but the promontory would become more pronounced due to the recession on either side. The eroding sub-bay between Minsmere Outfall and Sizewell C (the Southern Barrier) would continue to deepen at the present rate, hence the projected shoreline would retreat by up to 100 m. This is approximately the width of the existing barrier, implying that this sub-bay may also just enter into a phase of roll-back after a 50 year period. The southern limit of the retreat would be marked by the existing high ground at the north-east corner of the proposed Sizewell C development site, known as the Sizewell Bent Hills. Further south, the present low rates of change are expected to be maintained. The present day beach salient formed at the Sizewell B Outfall is likely to be maintained until the station ceases to operate, after which the beach is expected to 'relax', eroding locally until the salient has disappeared (as per the Sizewell A salient following cessation of operation; see Section 2.5). The relatively low rates of shoreline change presently

[18] As the Minsmere Sluice drains under gravity, future sea level rise will render the drainage function of the sluice inoperative, after which it would not be maintained.

being experienced between Sizewell A and the Thorpeness promontory are not projected to change as the majority of plausible environmental changes appeared more likely to increase sediment retention in this area, rather than decrease it.

As there is no definitive and official statement on the future preservation of the Minsmere Outfall, the EGA considered the possible decay or removal of the structure. It was considered likely that loss of the anchoring structure would lead to retreat of the current promontory that has developed around the structure, while increased longshore sediment movement would also reduce the present rates of erosion on adjacent lengths of the shoreline. However, it was also considered unlikely that the structure would be entirely exposed and removed during the period up until the shoreline reaches the HCDF footprint. Likewise, the possible future erosion of the Minsmere – Dunwich Cliffs would also increase sediment supply and availability and slow erosion rates (see Section 2.3.3). Were either or both of these to occur, shoreline retreat rates would reduce and lengthen the time-interval before the shoreline would reach the hypothetical (in the no Sizewell C scenario) HCDF position. Consequently, the case considered (retention of the sluice outfall and no Minsmere cliff erosion) amounts to a worst-case scenario in terms of the timeline for the shoreline reaching the HCDF footprint.

7.4 Future shoreline with Sizewell C (2053 – 2087)

To assess the impact of the station's coastal defences (hard and soft)[19], the EGA also generated an equivalent shoreline with Sizewell C in place, to represent the shoreline when the marine environment would begin to interact directly with the HCDF[20] – this scenario, which includes the SCDF but does not include any Additional Mitigation, is used in Section 7.5 to justify the need for mitigation in the form of beach/sediment management. The broad form of this future shoreline, and the coastal processes acting upon it, would be essentially similar to the present (with the exception of sea level rise) – that is, the same tidal regime and wave climate, infrequent overtopping (though frequency increasing with sea level rise), no breaching of the southern barrier and no new hard points. Initial exposure of the HCDF could potentially cause erosion of the Minsmere to Walberswick Heaths and Marshes SAC and Minsmere to Walberswick SPA, and would introduce elements and processes not naturally present (the effects of a subsequently-exposed HCDF at this time are briefly considered in Section 7.4.2).

This section concludes that, as a result of a period of potential erosion to the SAC/SPA, Additional Mitigation would be warranted to prevent the HCDF exposure and thereby retain a shingle beach frontage and longshore sediment transport continuity to minimise the impact of the HCDF on longshore transport and erosion (Section 7.5).

7.4.1 Shoreline position just before HCDF exposure

The sole area of difference between the shorelines with and without Sizewell C would be a 1 km stretch extending north from the Sizewell C frontage along the Southern Barrier (Figure 67 and Figure 68). Under the action of the bimodal wave climate, the SCDF sediment would be slowly redistributed both north (and so

[19] As the other Sizewell C marine structures would not have a Likely Significant Effect or Adverse Effect on Integrity (AEoI) on European sites, only the effects of the SCDF and HCDF are considered. Once decommissioned, BLF deck piles and all intake and outfall structures would be removed to below the seabed, as is standard practice in the North Sea.

[20] The future environmental impact of the HCDF can then be assessed based on the difference between these separate shorelines, incorporating potential mitigation strategies.

reduce the rate of natural retreat relative to that which would occur without the station) and south (where little change is presently occurring). Expected events at Sizewell B are considered likely to reinforce this pattern:

- The defensive 5 m (ODN) berm and the SCDF would be a future source of sediment, which if eroded would stabilise or reduce erosion rates across the Sizewell C frontage.
- Alongshore release of beach shingle currently residing within the Sizewell B beach salient is expected following cessation of Sizewell B operations, which would contribute to broad shoreline stability[21].
- The expected relaxation of the shoreline when Sizewell B enters its decommissioning phase (which was also observed when Sizewell A stopped operating), may also reduce erosion pressure immediately north of the HCDF, due to gross transport during SE events and trapping of some material to the immediate north of the HCDF.

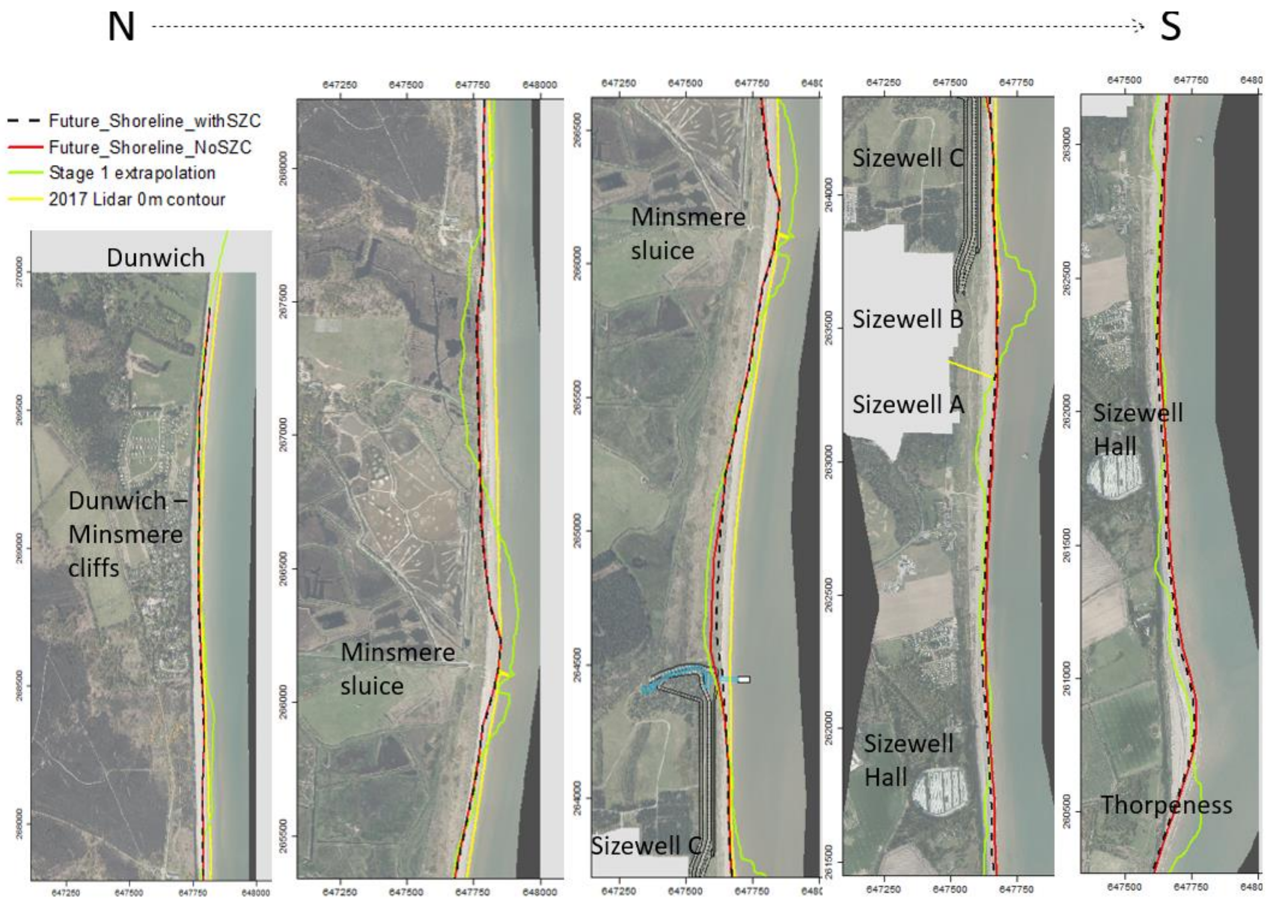

Figure 67. EGA projected future shorelines from Dunwich to Thorpeness compared to the linear extrapolation of present shoreline change rates. The extent of the expected impact of Sizewell C, a reduction

[21] The formation of the Sizewell B salient is thought likely to be linked to the deviation of the nearshore bars around the site of the operational cooling water outfall. Presently, it fluctuates slightly in volume and alignment, but data collected since 2011 (presented in BEEMS Technical Report TR223) suggest that is no longer growing and is not an ongoing sink for longshore shingle.

in erosion, is limited to a 1 km stretch between Sizewell C and Minsmere sluice outfall. From left to right, panels run north to south. The black dashed line is the EGA future shoreline with Sizewell C.

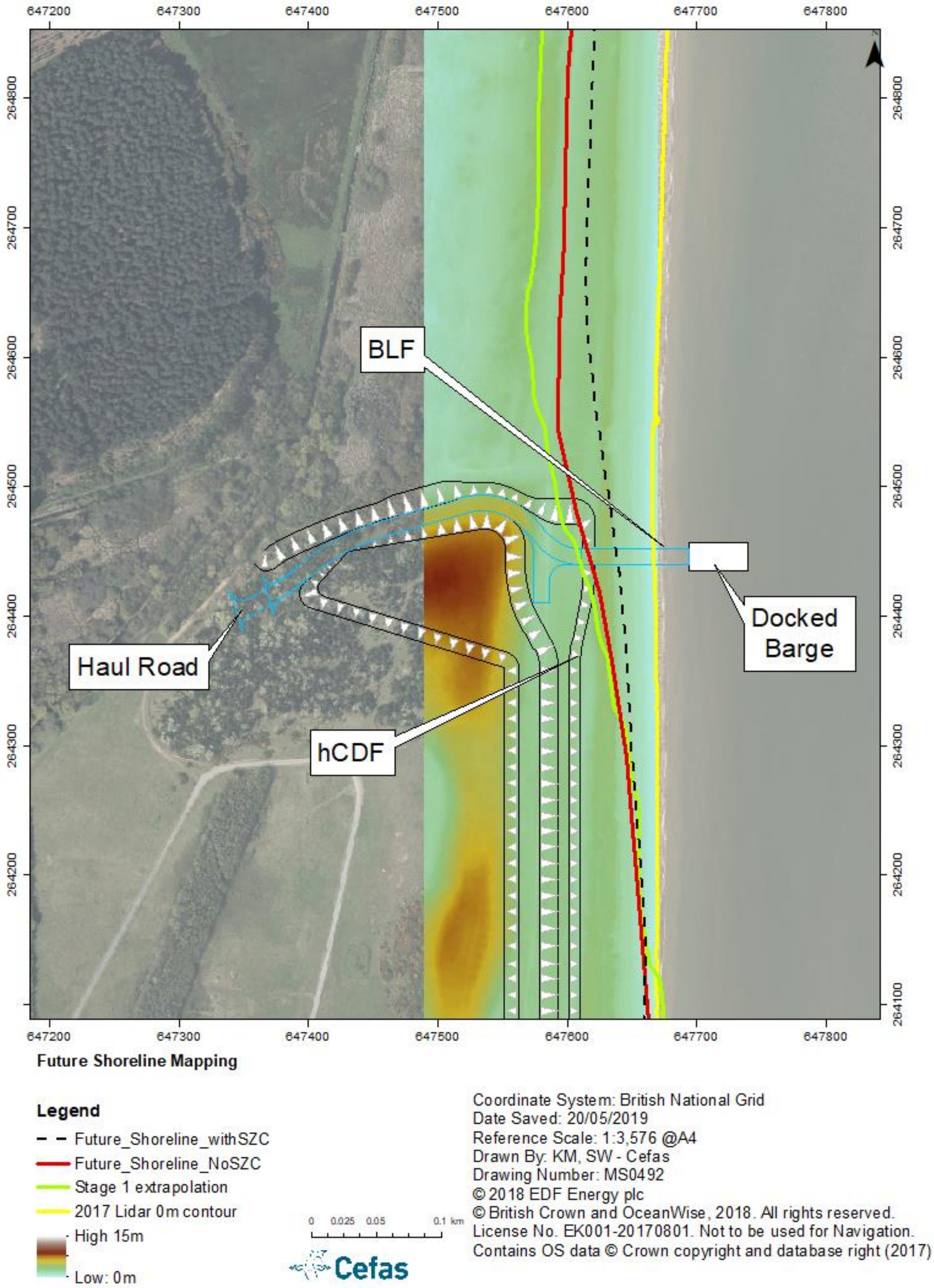

Figure 68. Projected shorelines with and without Sizewell C, showing the expected exposure of the HCDF (white hatching indicates sloped surfaces) and constraints on the shoreline position just north of the development site. The existing 'mound' of high ground at this location (the Sizewell Bent Hills) would have a similar bounding effect on the beach roll-back without Sizewell C. The black dashed line is the EGA future shoreline with Sizewell C.

The shoreline retreat between the northern half of the Sizewell C frontage to a few hundred metres north of Sizewell C would be reduced by several tens of metres over a number of decades. As well as slowing erosion rates, the presence of Sizewell C's coastal defence features could lead to restoration of the formerly destroyed supra-tidal 'annual vegetation of drift lines' habitat (Minsmere to Walberswick Heaths and Marshes SAC) and potential nesting sites for little tern (*Sterna albifrons*) (Minsmere to Walberswick SPA) just north of Sizewell C. It would also mean that the shoreline would not retreat back to the SSSI crossing over this time scale.

Individual sediment contribution events from the SCDF to the active beach would be relatively small and episodic. Evidence from Storm Emma and the weakening polar vortex ('The Beast from the East') in March 2018 showed substantive reprofiling of the intertidal and supra-tidal beach in some areas, but limited erosion of the barrier itself – the 1 m^3 per metre of beach width eroded from Sizewell C's barrier toe (above 3 m ODN; see BEEMS Scientific Position Paper SPP094) equates to approximately 3 cm of raised beach elevation were the material spread evenly over the active beachface. Changes to the local shoreline, as well as the locations and volumes of sediment added to the system by the sacrificial SCDF, would be monitored (see Section 6.5).

7.4.2 Exposure of the HCDF – cross-shore and alongshore effects

If the coast were allowed to recede without further intervention, the beaches along the Sizewell C frontage could gradually reduce in width until the denuded beach exposed the HCDF[22], either permanently or intermittently. Two effects would then be likely: (1) scour at the exposed face of the HCDF due to the interaction between waves, currents and the HCDF, which would cause localised beach lowering and hinder sediment deposition; and (2) patterns of localised erosion and accretion caused by the HCDF's disruption to longshore sediment transport. These effects are discussed in turn below.

7.4.2.1 *Cross-shore effects*

Provisional uncalibrated cross-shore modelling (using XBeach 1D) of a narrow, volumetrically depleted, beach profile (i.e., as would be expected shortly before initial exposure of the HCDF) was conducted to indicate the scale of beach lowering that could occur. The storm event in late February and early March 2018, which was a combination of Storm Emma and a weakening polar vortex known as 'The Beast from the East', generated wave heights of over 2.1 m for 65 hours (peak of 4.1 m) at the Sizewell wave buoy (BEEMS Technical Report TR486 and Scientific Position Paper SPP094), and was used to force the model. The morphological start condition was a narrow, low-volume beach profile, representing a future depleted beach/SCDF prior to HCDF exposure – it was determined by truncating and translating landward a typical Sizewell C profile (see Figure 69, heavy dashed line). The HCDF was modelled as a solid, non-erodible, vertical wall of infinite depth. The observations of scour and beach lowering in front of this structure are conservative as the front face of the HCDF would actually be sloping (non-vertical), porous, rock armour with less turbulence and erodibility than the modelled case. The modelled storm acting on the depleted profile caused temporary exposure of the HCDF and maximum lowering of 2.25 m, followed by a partial recovery of beach levels (Figure 69). The beach levels at the HCDF were very similar at the start and end of the storm, suggesting intermittent exposure is a possibility.

Building on the monitoring and mitigation described in Section 6.5, further modelling (including calibration and examination of different storm conditions) would be undertaken to determine a threshold beach volume designed to avoid exposure of the HCDF under high energy storm conditions – this would be used in

[22] It is possible that the shingle beach would not permanently, or temporarily, expose the HCDF – shingle beaches often co-exist with hard defences. Here and at this approximate time scale (2053 – 2087), exposure is used as the worst-case scenario. With no mitigation and on a longer timescale, exposure is inevitable due to rising sea levels.

monitoring plans to specify when any Additional Mitigation should be applied (see Section 7.5). The modelling work presented here and in BEEMS Technical Report TR486 provided initial conservative indications of scour, but is not being used in the design of the HCDF foundations – the final design will be determined by EDF's engineering design team and assessed as part of the Nuclear Safety Case by the Office for Nuclear Regulation – it is not relevant to, nor duplicated in, the Environmental Statement or this synthesis.

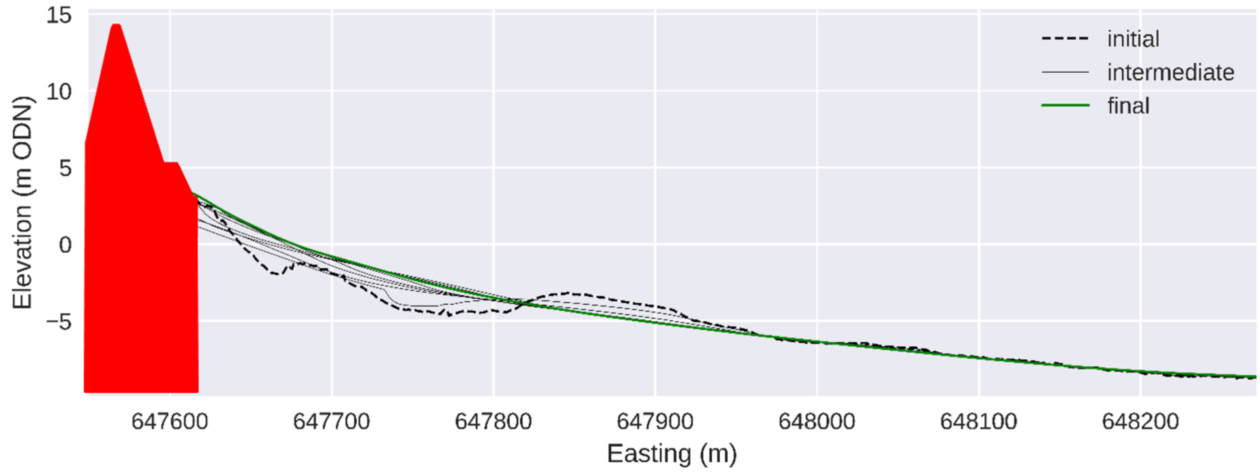

Figure 69: Profile response of a beach with an almost depleted SCDF. The starting beach profile is represented by the black dash lined, the green solid line the end profile and the grey lines the intermediate profiles every 24 hours (BEEMS Technical Report TR486).

7.4.2.2 *Alongshore effects*

The alongshore effects of an exposed HCDF would be localised downdrift erosion and updrift accretion. As Sizewell's longshore drift climate is bi-directional, and there are no UKCP18 climate predictions of a directional shift in wave climate, short-duration episodes of erosion/accretion would be expected on either side of an exposed HCDF in accordance with changing storm direction. Although conceptually similar to the Minsmere Outfall, which also disrupts shingle movement, the effects of the HCDF would be more subtle due to its curved shape and minimal easterly protrusion for sediment moving south to north under southerly storms. That is, the HCDF would have no likely significant effect on shingle supply to the Southern Barrier and beach.

Exposure of the HCDF could initially cause some very localised erosion at the NE corner, probably within the Sizewell C site boundary. However, once the shoreline is more recessed than (i.e., landward of) the NE corner of the HCDF, sediment would be trapped[23] on its north face causing net accretion/stability and barrier building to the north, and localised starvation and narrowing to the south along the Sizewell C frontage. Trapping of sediment following initial exposure is a negative feedback loop that would promote accretion to the north of Sizewell C, counterbalancing the initial exposure and stabilising the area. Changes in the shape of the shoreline as it becomes naturally more recessed north of Sizewell C, the additional sediments provided by the SCDF and beach maintenance practices, could result in restoration of the former supra-tidal 'annual vegetated drift lines' habitat, destroyed in 2011 (i.e., net environmental gain) (Appendix A). It would

[23] Were recession rates north of Sizewell C to be maintained (at least in a relative sense), that shoreline would eventually hold a more landward/easterly position than the HCDF. The consequent curvature of the shoreline from this point to the Sizewell C frontage would act to locally retain the sediments that are already present barrier in the barrier, and to accumulate additional sediments.

also mean that the shoreline would not retreat back to the SSSI crossing over this time scale. The low net shingle transport rates and gross-transports indicate a down drift starvation extent in the order of hundreds of metres, which is not sufficient to affect the Leiston – Aldeburgh SSSI.

In contrast, SE storms and, in particular, occasional winters dominated by several SE events (which tend to occur once or twice per decade), would have limited potential to erode the accumulating sediments to the north of the HCDF, due to its low-profile obstruction to northerly longshore transport. The join between the southern section of the HCDF and Sizewell B's curved 10 m defence, approximately 750 m south of the Minsmere SAC/SPA frontage, would form a sediment filled, hard-backed, 150 m-long embayment in which the curved HCDF protrudes 20 m seaward of Sizewell B's defence. In the unlikely event that all of the beach material was lost, the hard protrusion would have a relatively gentle gradient of 25°, which is small relative to angular changes that normally trap sediments (e.g., groynes, Minsmere Outfall, at 90°). A smaller 11° gradient would be apparent on the south side of the BLF landing area (again, if all of all of the beach sediment were stripped). Although these are subtle shifts with limited ability to block shingle transport, their potential to cause some small magnitude erosion on the Minsmere SAC/SPA frontage has led to the proposal for Additional Mitigation (if required) to maintain the beach and shingle transport corridor (Section 7.5).

7.5 Additional Mitigation – beach maintenance

The coastal configuration at the 2053 – 2087 time horizon would have a similar basic form to that of today, however overtopping/breaching on the Northern Barrier (north of Minsmere Outfall) would have become more frequent, signalling a transition there from barrier scarping to roll-back. On the Southern Barrier (immediately north of Sizewell C) in comparison, the probability of overtopping/breaching events would be lower due to trapping of shingle against the northern side of the HCDF after either:

- its initial exposure or
- as the shoreline retreats to a more landward position, relative to the HCDF (see Figure 70).

Accumulating shingle would inhibit erosion and potentially prevent breaching altogether (Figure 70). Natual overtopping would be expected as this process is required in barrier building (i.e., shingle deposition on the barrier crest).

During the phase just before the shoreline moves landward of the HCDF, there would be some potential for unmitigated beach recession to expose the NE corner of the HCDF. Exposure would affect (reduce or block) longshore shingle transport on the Sizewell C frontage, thereby locally altering the rate and/or direction of shoreline change, which could include short-term, low-magnitude erosion of the Minsmere SAC/SPA frontage under SE storms (in the absence of Additional Mitigation; Section 7.4.2). To avoid exposure of the HCDF, blockage to the shingle transport corridor and a potential erosion impact north and south of Sizewell C at this stage, Additional Mitigation would be undertaken.

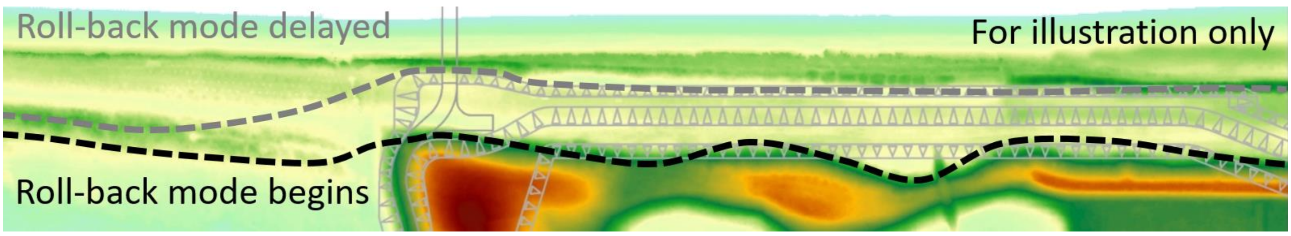

Figure 70: Schematic of a shoreline without Sizewell C (black dashed line) in which the Southern Barrier just north of the Bent Hills (dark yellow and red topographic highs; pale yellow and blue are low elevations) has begun to breach and roll back. This is compared to the shoreline with the Sizewell C HCDF (grey dashed line

and thin grey solid line and triangles indicating slopes, respectively) at the same time-horizon, which would delay or prevent breaching and roll back.

7.5.1 Beach maintenance methods

The proposed Additional Mitigation is beach maintenance. Its purpose would be to maintain a shingle beach in front of the HCDF, thereby maintaining the shingle transport corridor and preventing sediment starvation either side of a potentially exposed feature. Maintenance of a continuous shingle beach would minimise or eliminate any disruption to longshore transport.

The method, location and volumes for each mitigation action would depend on the circumstances at the time – the future monitoring evidence base would be used to identify areas of potential exposure and mitigate them if a significant impact is predicted (see Section 7.6). Although depleted beach locations, and the best methods to mitigate them, cannot be predicted *a priori*, only a few scenarios are possible, and so guidelines can be formed regarding the types of beach depletion and their matching mitigation. These will be fully detailed in the beach monitoring and management plan to be developed in 2020 as an expected DCO/DML requirement. The mitigation methods involve either moving beach sediment (bypassing or recycling) or introducing new material (recharge).

Beach recycling would be used if sediment accumulated at one location but threatened to expose the HCDF at another (Figure 71, top panel). This technique has been used elsewhere in Suffolk (e.g., the mixed sand/shingle South Beach at Lowestoft) and by EDF Energy and the Environment Agency on the UK's largest coastal shingle landform at Dungeness (which, like the Sizewell – Minsmere area, has several conservation designations). Beach recycling involves no additional sediments, just sediment movement from a borrow (orange in Figure 71) to a deposit area (green) – for example, a salient in the lee of outfalls, were one to form. The borrow and deposition sites can be up or downdrift of one another. The effect on the shorelines would be accretion or a reduction in erosion rates local to those deposition sites, and the sediment would slowly disperse with time. Should this situation arise (i.e., accumulation and depletion) beach recycling would be applied. The intention is not to extract sediment from the designated Minsmere sites, however if a case for this did arise it would be subject to any assessments and approvals relevant at that time; as the future environment naturally changes, some designated habitats/features (as described by Natural England's condition assessment reports and DCO/DML monitoring reports from the beach monitoring and mitigation plan) may also naturally change in quality or disappear, potentially allowing such an activity.

Bypassing would be used if the HCDF caused persistent updrift sediment accretion and downdrift erosion (Figure 71, middle panel). The potential accumulation sites for bypassing are north of the northern HCDF face (under phases of southerly transport), immediately south of the BLF landing area and in the embayment between the HCDF and the Sizewell B defences (under phases of northerly transport). Given the natural bi-directionality in longshore transport, consideration would be given to the persistence of erosion/accretion patterns so as to avoid unnecessary disturbance and mitigation activity (i.e., mitigation would not be applied if the beach was considered likely to recover naturally without causing any significant impacts, which is a commonly applied coastal management practice). The effect of bypassing is to manually restore longshore sediment supply and alter the shoreline position local to the extraction and deposition sites. As with beach recycling, the intention is not to extract sediment from the designated Minsmere sites, however if a case for this did arise it would be subject to any assessments and approvals relevant at that time.

If sections of the Sizewell C frontage were to become depleted in the absence of an obvious or accessible zone of accumulating sediment, beach recharge would be applied (Figure 71, bottom panel). The effect of introducing extra sediment would be to initially decrease, halt or reverse the erosion rate, as well as maintain continuity in the beach and longshore transport system. Over time, the introduced material would slowly disperse. As the length of recharged coast would be short (100 – 200 m), the slow dispersion of additional sediment is unlikely to be distinguished from natural behaviour along the neighbouring coasts. The sediment would be expected to move slowly south in net terms (except where it became trapped on the northern side of the HCDF).

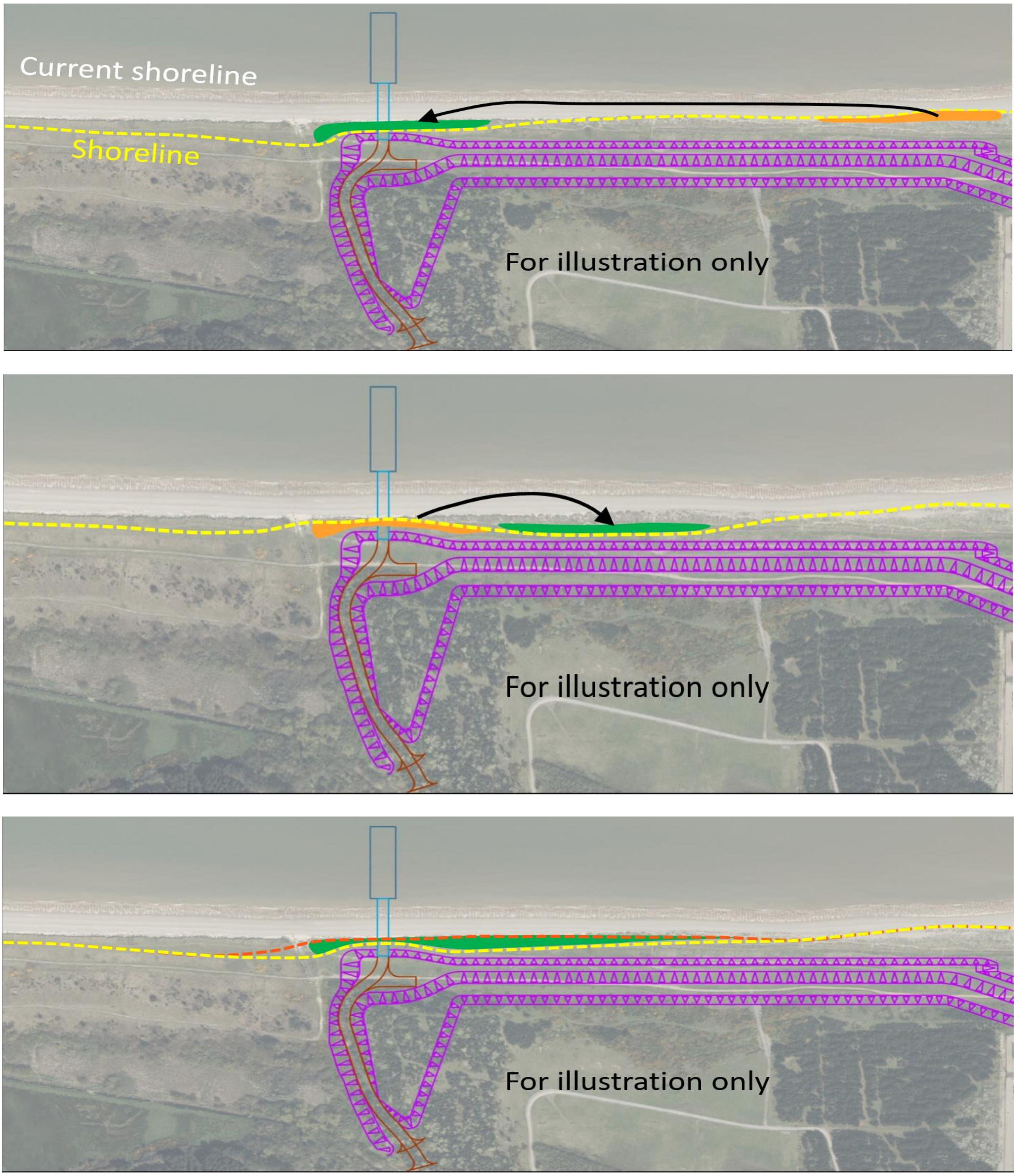

Figure 71: Schematics showing examples of depleted beach sections and the likely mitigation method: beach recycling (top), sediment bypassing (middle) and beach recharge (bottom). The examples assume a net southerly (left to right) longshore drift, but the same principles can be applied in the unlikely event of any persistent reversal in the net transport direction.

As it is not possible to predict depleted areas in advance, it is also not possible to predict the required volumes of beach grade sediment to be moved or supplied, as this would depend on the beach condition (volume, extent) warranting intervention. Instead, a robust monitoring plan would gather the evidence needed to identify areas of beach depletion and propose the details (type, location, volume) of each individual mitigation.

The proposed mitigation methods are viable, initially at least, because:

- the shoreline retreat rates are not especially fast;
- retreating areas typically have a relatively small spatial extents (i.e., spatially localised erosion/accretion patterns following individual events is a strong characteristic of this coast);
- shingle has a high entrainment threshold (i.e., low mobility, so maintenance activities would have a moderately high resilience); and
- shingle moves relatively slowly and has a very low net displacement.

Hence the environment around Sizewell C is considered suitable for bypassing, recycling and, if needed, recharge. Note that beach recycling is a practice that has been employed in the UK at sites with both Hold The Line (e.g., South Beach, Lowestoft) and Managed Realignment (e.g., Slapton Sands) SMPs.

7.5.2 What are the potential impacts of beach maintenance practices on designated sites?

The beach maintenance / sediment management approaches described above would not have an adverse effect on designated supra-tidal shingle habitats (annual vegetated shingle and potential little tern nesting sites) as:

- they would not cause erosion;
- they would cause some localised short-term beach accretion, limited in extent by the relatively small volumes being moved or introduced;
- sediment would not be extracted from statutory designated sites (in the cases of bypassing or beach recycling) unless accumulating sediments were a direct effect of Sizewell C (mitigation or presence of the HCDF) and approval was given following demonstration that designated features would not be affected; and
- sediment would not be deposited on the supra-tidal beach within statutory designated sites unless approval was given following demonstration that designated features would not be affected.

Beach maintenance would result in some localised short-term beach accretion, limited in extent by the relatively small volumes being moved or introduced. Were added or moved shingle to subsequently deposit on the supra-tidal, it could increase its elevation and width, and potentially restore lost features such as the annual vegetated drift lines habitat (net ecological gain). This situation would most likely occur for beach recharge mitigation, as it would add new shingle to the system.

Deposited material would move under natural coastal processes within the active beach, behaving in the same fashion as the rest of the beach material. Therefore, deposited sediments are no different from the material already present. Effort would be made to spread deposited material appropriately, so that unnatural mounds or shapes would not result, which would allow the beach to function naturally. Any beach maintenance activity directly on the designated frontage would require assessment and approvals from Natural England. Notwithstanding approvals, sediment extraction from the active beach face (not the supra-tidal zone) could still produce a net ecological gain, if only some of the excess sediments (for example, accumulating sediments against the HCDF) were removed.

Beach maintenance, and the trapping capacity of the HCDF, would also prevent the shoreline from retreating to the SSSI crossing.

The Leiston – Aldeburgh SSSI is too distant to be affected by beach management activity at Sizewell C, as shown by modelled longshore transport and measured shingle movement (BEEMS Technical Reports TR329 and TR420).

The monitoring plans for supra-tidal substrate and vegetation (Section 6.2), could also contribute to Natural England's condition assessment reports.

7.6 Beach maintenance triggers – initiation and cessation

The Additional Mitigation activity of beach maintenance would start when declining beach volumes threaten HCDF exposure that would subsequently result in a likely significant effect[24]. That is, a threshold beach volume per unit length of coastline, together with identification of a potential significant effect without mitigation, would constitute the trigger to start mitigation.

The volumetric threshold required to withstand an agreed storm condition, and thereby avoid detrimental HCDF exposure, would be determined using a storm erosion model, developed as part of the beach monitoring and mitigation plan in 2020. It would be a requirement of the DCO and DML[25] as set out in the monitoring plan. The shoreline and beach volume methods outlined in Section 6.5 would be used to establish if the trigger threshold had been reached, and to determine which sections require mitigation, the type of mitigation to be employed and, for bypassing and recycling, the locations of any borrow areas (where accumulation occurred).

Monitoring would continue in order to assess mitigation performance and detect any future mitigation needs. The same mitigation trigger (beach volume and impact assessment) would also be used to trigger any subsequent mitigation activity.

Figure 72 illustrates the basic steps for determining whether a mitigation action would be required, as well as the conditions under which mitigation would cease. These steps would be based on future monitoring evidence and assessment for significant effects of a potentially exposed HCDF.

[24] A significant effect is defined using the EIA methods found in Volume 1, Chapter 3, Appendix 6P and Volume 2, Chapter 20, Section 20.3.

[25] The details of the monitoring plan are not required (or developed) as part of the Environmental Statement evidence base following the procedure used to discharge the DCO/DML conditions at Hinkley Point C.

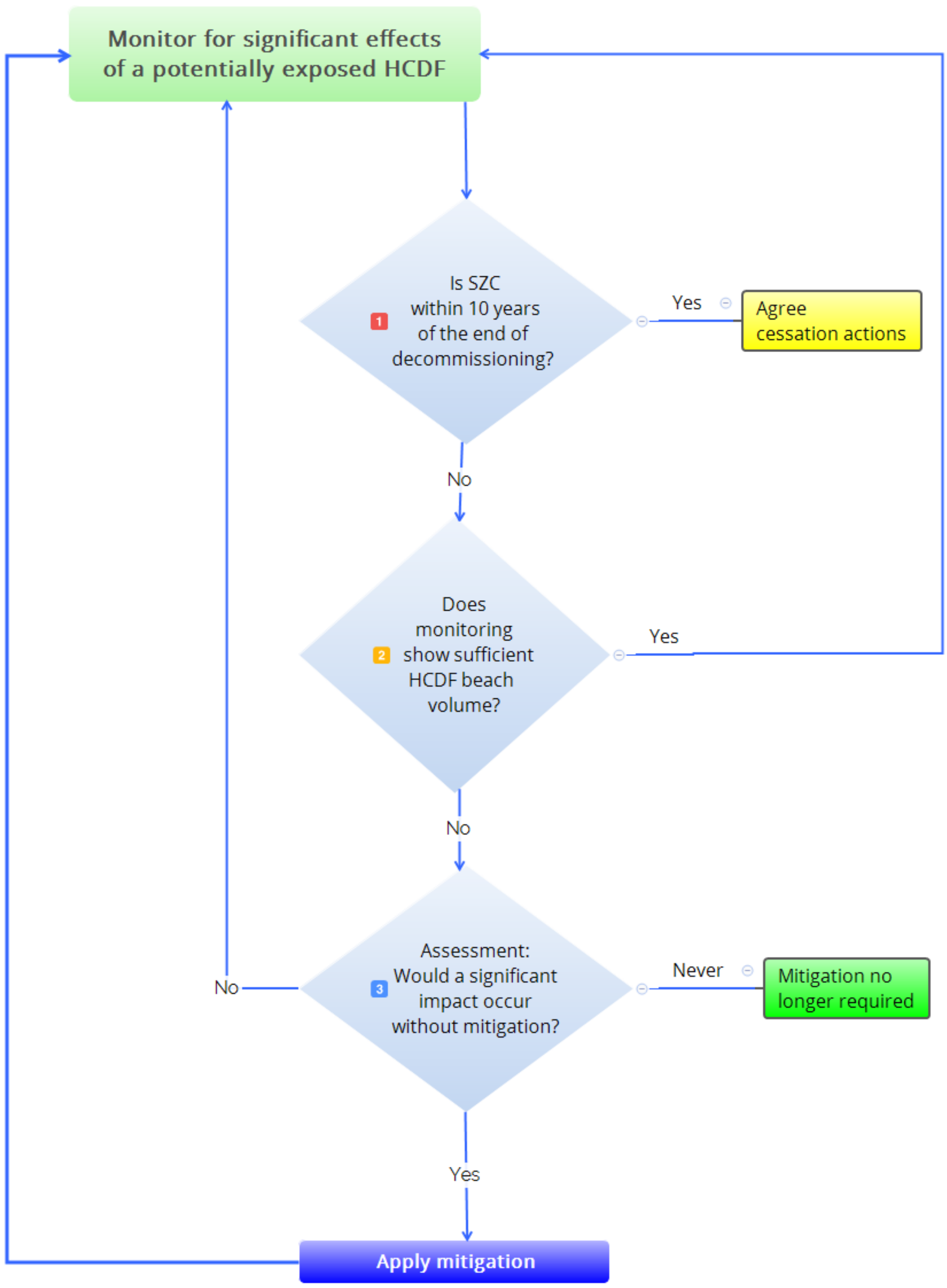

Figure 72: Steps to determine whether mitigation should occur, whether it is no longer required or whether compensation needs to be considered.

The numbered steps in Figure 72 are detailed below:

1. **Is Sizewell C within 10 years of the end of decommissioning?** Decommissioning is the scheduled end of project trigger for cessation of monitoring. Before the monitoring and mitigation could cease, the scientific evidence of any likely significant impact would be provided to the marine regulator and, were there a significant impact, cessation actions would be agreed. Mitigation would not occur beyond the end of Sizewell C decommissioning, as the operating company would cease to exist at this time. It is only possible to consider cessation actions at the decommissioning time horizon because the impacts of an exposed HCDF, and the locations and extents of designated sites and features, cannot be established until that time. If Sizewell C has not been decommissioned, then step 2 would be considered.

2. **Does the monitoring evidence show sufficient HCDF beach volume?** The monitoring evidence would be used to track beach volumes. The early warning system described in Section 6.5 would be used to trigger topographic surveys in the event that a critical beach volume may have been reached. The early warning system would run continuously and so would detect beach volumes lower than the threshold earlier than regular topographic surveys. For accuracy reasons however, only topographic surveys can be used to determine if beach volumes have fallen below the threshold. The threshold would be set in the monitoring plan for a given length of coast. If the beach volume fell below the threshold for any HCDF coastal section, an impact assessment would be required.

3. **Would a likely significant effect occur without mitigation?** The impact assessment that answers this question would use the EIA methodology described in Section 20.3 of this chapter. It would deliver one of three answers:

   i. No. If the location and volume of a depleted beach section does not cause a significant effect, mitigation would not be justified. However, depending on the circumstances a temporary increase in topographic survey frequency may be required.

   ii. Never. If it were shown that an exposed HCDF would have no significant effect on designated sites, then the monitoring frequency could be decreased and, eventually, monitoring and mitigation could cease (once the evidenced case for this was compelling). There are several ways in which this situation could arise:

      - Evidence showing that HCDF exposure would not generate any significant effects.
      - Natural retention of a shingle beach in front of the HCDF, thereby maintaining the transport corridor without intervention. Mitigation would not be required if the shingle beach were naturally self-sustaining.
      - Re-designation of statutory designated site boundaries, and/or the features within them, as a result of natural changes in the coastal environment, specifically supra-tidal shingle. The shifting of designated features may mean they no longer intersect with the impact areas, or no longer exist, resulting in no significant impact.

   iii. Yes. If a significant impact were expected to occur in the absence of mitigation, then step 3 would be considered.

As the performance of mitigation cannot be foreseen, it would be an important component of the beach monitoring and mitigation plan reporting. The monitoring and reporting frequency will be determined in the beach monitoring and mitigation plan, which will require approval from the MMO under the DML. Monitoring will also track and document the natural behaviour of the adjacent coastline.

Regular scientific monitoring and mitigation reports would be submitted to the MMO, including reports on the need to mitigate and mitigation performance, and these would require MMO approval (in consultation with regulatory stakeholders as required).

Other forms of evidence may also be used in decision making, especially if impacts cannot be readily distinguished from natural behaviour. For example, when new material (the eroded SCDF and any beach recharge mitigation) volumetrically exceed that made unavailable due to the presence of the HCDF (see Figure 73), the Sizewell C development would become a net contributor to coastal sediment supply.

This volumetric principle relates only to the SCDF and beach recharge and is effectively a sediment budgeting consideration. The schematic in Figure 73 highlights sediment made unavailable for future erosion due to the presence of the HCDF ($V_L \approx 130{,}000\ m^3$), sediment that would be available without the SCDF (V_C) and additional sediment made available for erosion from the SCDF ($V_{SCDF} \approx 120{,}000 m^3$). The balance between V_L and V_{SCDF} is expected to be a small volumetric deficit. However, the net sediment contributions of Sizewell C from the SCDF and beach recharge are expected to exceed V_L ($V_{SCDF} + V_{recharge} > V_L$) and so may offset any residual effects apparent at the end of decommissioning or at an earlier time should mitigation fail to be effective.

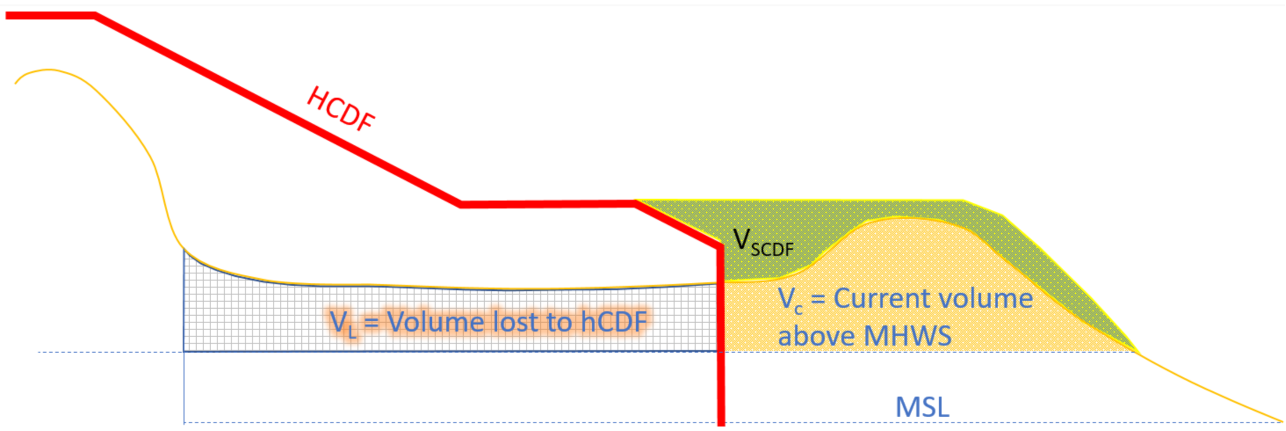

Figure 73: Cross-sectional schematic showing the lost and gained sediment for future beach erosion due to the HCDF.

7.7 Potential post-mitigation effects

The flow chart in Figure 72 shows that mitigation will cease by the end of decommissioning or earlier if the mitigation is no longer required. The exact timescale for cessation is unknown but long. It would begin after:

- **The terrestrial HCDF period (no marine impacts and no Additional Mitigation).** This period is expected to be until 2053 – 2087 and features coastal processes and parameters (other than sea level) similar to the present; and
- **The beach maintenance (Additional Mitigation) period.** The duration of this period cannot accurately be determined at this time (see Section 7.6) but would be expected to last for several years to decades. During this period beach maintenance activities would maintain the continuous shingle beach feature, which would allow sediment (shingle and sand) to flow around the HCDF and thereby prevent a sediment transport blockage.

This means that the post-mitigation period could at any time up to the end of decommissioning[26]. Were any residual significant effects to remain at this time, they would need to be identified, assessed and compensated. However, the detail required to undertake that assessment cannot be known until much closer to that time, when the nature of the HCDF exposure, the broad geomorphic setting and the locations of designated sites and features, are all known. Despite this, it is possible to conceptually determine the geomorphic setting and the potential effects that could occur. In doing so, a broad context is given whose understanding can then be evolved as monitoring evidence is gathered over many decades on the evolving geomorphology and the habitats it supports.

Section 7.7.1 considers the key factors controlling long-term coastal evolution at Sizewell, along with their direction of travel and certainty. These factors are then used to develop two geomorphic endmembers of a possible spectrum in Section 7.7.2, and the impacts that an exposed HCDF could have on them.

The potential effects in Section 7.7.2 are not for assessment

Rather they are to be evolved and updated with the changing coast and eventually used in a final assessment at the end of mitigation to determine the detail of any significant effects.

7.7.1 Factors controlling long-term coastal evolution at Sizewell

With rising sea levels expected for centuries to come, cessation of beach maintenance is likely to lead to exposure of sections, or all, of the HCDF (temporarily or permanently) and impacts not apparent prior to exposure. Any impacts arising are likely to differ from those described in Section 7.4.2 due to the longer post-mitigation time-horizon and the associated increased risk of a regime shift with new coastal processes and geomorphic features (breaching, roll-back and other exposed engineering features) – by definition this would also be reflected in changes to the location, extent or existence of designated sites and features (extents or loss of features).

Initial exposure of the HCDF could cause localised erosion of a few tens of metres on the Sizewell C frontage (to the north of the HCDF), but this would be followed by shingle trapping as the NE corner of the HCDF protrudes into the longshore shingle transport pathway (as explained in Section 7.4.2.2). The shingle beach may re-establish itself as a result of trapping on a permanent basis, or it may be intermittently exposed. Intermittent exposure would result is periods of disruption to longshore supply, followed by catch up as slugs of material are released from the accumulating shingle north of the HCDF.

The key factors controlling long-term coastal evolution at Sizewell are considered in Table 27. Some of the factors controlling the longer-term evolution of the coast are not expected to change significantly:

- wave climate directionality,
- storm surge,
- tidal elevations and currents, and
- the beach sediment size range (though the size distribution may change).

In contrast, other factors are expected to change and are moderately well-known, at least in terms of their direction of travel:

- mean sea level will continue to rise (at a rate higher than present),
- sediment supply will increase due to sea level rise and increasing exposed cliff length, and

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[26] Mitigation would not take place beyond decommissioning (see Section 7.6).

- engineered features – the tip of the Coney Hill cross-bank (also known as the North Wall) and the Sizewell A and Sizewell B hard defences – would become exposed.

Annual and maximum wave heights are forecast to decline in the UKCP18 projections for Sizewell, against a background of no compelling trends in storminess (as determined by maximum gust speeds). It is worth noting that UKCP18's forecasts for decreasing wave height are specific to the GSB.

Table 27: Factors controlling coastal configuration at the post-mitigation timescale (c. 2080 – 2120). The direction of each factor, and the associated certainty are shown.

Controlling factor	Direction of change	Certainty of change direction
Hydrodynamic – sea level rise	Rising	High (UKCP18 RCP8.5 indicates 0.56 to 1.12 m of sea level rise)
Hydrodynamic – annual mean significant wave height	Minor decrease (-3.3% for 2081 – 2100 using RCP8.5; Lowe *et al*., 2018)	Moderate but the climate of enclosed seas can be affected by local weather (Section 2.4.2)
Hydrodynamic – annual maximum significant wave height	Decrease (12.3% for 2081 – 2100 using RCP8.5; Lowe *et al*., 2018)	Moderate but the climate of enclosed seas can be affected by local weather (Section 2.4.2)
Hydrodynamic – wave direction	No evidence available to determine a potential trend (UKCP18)	Not applicable
Hydrodynamic – storm surge	No significant change is expected in the atmospheric contribution to storm surge (UKCP18)	High
Hydrodynamic – tides	Minor (-2% for 3 m SLR; BEEMS SAR036)	Sign uncertainty at this very high SLR but change is small
Sediment supply – mass	Rising as sea level exposes and erodes a greater length of cliff line	High
Sediment supply – particle size	Stable/similar	High. Eroding source cliffs are composed of sand and shingle
Altered geomorphology – breached barriers	Rising in frequency and area	Moderate / high (due to sea level rise, but could be countered by rising sediment supply)
Altered geomorphology – Sizewell-Dunwich Bank	Dynamic. Stable or rising volume. Stable Sizewell Bank, variable Dunwich Bank	Moderate – rising sand supply with similar or improved (less disruptions) supply pathway
Engineering – North of Sizewell C (Walberswick jetties, Coney Hill cross-bank, Minsmere Outfall)	Unknown. No formalised plans for Walberswick jetties; Coney Hill cross-bank likely to be exposed; Minsmere Outfall likely to have decayed or been removed.	Not applicable
Engineering – Sizewell C's HCDF	Increasing effect – increased disruption to longshore transport until an equilibrium shoreline formed (which may or may not block transport)	Moderately high, although a narrow shingle beach may persist
Engineering – Sizewell B salient	Absent – sediments dispersed, and coast straightened	Very high
Engineering – South (Sizewell A and Sizewell B hard defences)	Increasing – eventual exposure of these structures	High

Barrier overtopping and breaching frequency is to likely rise on the Northern Barrier (north of the Minsmere Outfall) due to sea level rise, however increased sediment supply, and in particular shingle from the Minsmere – Dunwich Cliffs, may temper the breaching/overtopping frequency compared to that expected under sea level rise alone. The breaching potential at the southern end of the Southern Barrier (without Sizewell C) would be affected by similar processes, however it would experience a time-lag in the arrival of new shingle (from the Minsmere – Dunwich Cliffs) compared to the Northern Barrier, which is closer to the source.

Although sand supply to the Sizewell – Dunwich Bank is expected to rise, the bank form, elevation and extent cannot be accurately predicted. Historical data over multi-decadal/centurial time periods show that the southern part of the bank (Sizewell Bank) is very stable and this is unlikely to change, as supported by several strands of evidence (Section 2.3.4):

- erosion resistant Corallina Crag;
- persistent tidal circulation caused by the Thorpeness headland and crag ridges; and
- the offshore funnelling of sands by the sub-tidal outcropping crag.

In contrast, Dunwich Bank has been highly variable in its elevation and extent over decadal time periods and this is likely to continue; any phases in which the bank elevation and extent are reduced could lead to increased wave exposure of the Minsmere – Dunwich Cliffs and erosion; over time, sea level rise may exacerbate this issue as bank lowering and rising sea levels could combine to increase wave energy at the shore. Although Dunwich Bank is currently in a phase of lowering, cliff erosion rates remain low. Were erosion to be re-activated by the reduced Dunwich Bank, new shingle and sand would be released into the GSB. Any erosion is likely to be slower than the pre-1925 period, due to the bidirectional wave climate, low longshore transport rates and a negative feedback where eroded sediments would initially widen local beaches and offer periods of protection. Thus, over the next 100 years, stability at Sizewell Bank and variability at Dunwich Bank is plausible/likely, with episodes of new sediment supply due to erosion of the Minsmere – Dunwich Cliffs being likely.

There are no official published plans describing the very long-term future of the Minsmere Outfall, Coney Hill cross-bank and the Walberswick jetties:

- The Walberswick jetties would be maintained according to their SMP status of Hold-The-Line over all three epochs. Although there is no official position on a change to this SMP (i.e., removal of structures or introduction of new ones), any shortening or removal would reduce the longshore transport barrier and maintain or increase the rate of sediment supply into the GSB.
- The Coney Hill cross-bank is a clay structure and would have limited resistance to erosion were it exposed. There are no current plans to protect this feature once it is exposed.
- The Minsmere Outfall is expected to be maintained as a functional drainage outlet until natural regime change occurs on the Minsmere Reserve due to higher sea levels and frequent saline intrusion, which would render the drainage function ineffective. During the intervening period, the concrete outfall pipe could be partially dismantled or left to naturally decay. The buried sections of the outfall pipe may have a longer life expectancy due to their protection within the immobile beach, which would imply a gradual decay. Removal or decay would restore, or partially restore, the shingle transport corridor, possibly increase erosion of the Central Barrier and increase the rate of net southerly transport.

The broad evolution of the Minsmere – Sizewell shoreline is initially likely to be similar to that of today, but with deeper embayments on the Northern and Southern Barriers, if the Minsmere Outfall was still disrupting longshore shingle transport. However, toward the end of the station life, the most likely scenario is that the outfall pipe would no longer be present or effective as a longshore transport disruptor and the shoreline, particularly at the then former outfall, could rapidly recede under a coastal catch-up scenario (e.g., Dolphin et al., 2012). Without further intervention (maintenance or engineering), spatial patterns in recession rate would be locally influenced by the barrier volume and height, with higher rates expected on the Northern Barrier where the volume and height are lower, and lower rates to the immediate south (for 1500 m) on the Central Barrier where the elevation and volume are substantially greater. Coastal catch-up erosion near the Minsmere Outfall location would release shingle previously trapped by it, and from the barrier itself, into the active beach system. Travelling south under net longshore transport, these sediments could reduce erosion rates on the Southern Barrier and supply extra material to the HCDF trap and power stations frontage

7.7.2 Generalised post-mitigation shorelines and impacts

Of the factors discussed (Table 27), those most likely to promote different geomorphic configurations in future are:

- Sediment supply – stable or rising;
- Dunwich Bank – either
    - a higher bank promoting little or no erosion of Minsmere – Dunwich Cliffs; or
    - a lower bank with higher cliff erosion; and
- Sizewell A and B hard defences – either exposed (and so a barrier to longshore transport), or not exposed.

Combining these variable factors with the more predictable ones (in terms of direction of travel), suggests two endmember tendencies – (i) similar sediment supply with little/no Dunwich Cliff erosion due to a large Dunwich Bank and (ii) gradual increases in regional and local sediment supply (due to exposed Benacre – Easton Cliffs (see Section 2.4.3) and a lowered Dunwich Bank exposing Dunwich Cliffs). As the Benacre – Easton Cliffs are distant from the GSB, it would take time for increased sand supply to reach the GSB, however this is a gradual and ongoing process that is already occurring. Sand and shingle from the erosion of the Minsmere – Dunwich Cliffs would have a more immediate effect, especially for the closer Northern Barrier. These two tendencies – similar and increasing sediment supply – are used to derive generalised shoreline positions, examine the effects on coastal engineering (including Sizewell C) and consider potential effects on supra-tidal shingle.

7.7.2.1 *Case 1: similar sediment supply after cessation of beach maintenance (mitigation)*

The driving parameters for this case are increased sea levels but little to no erosion of the Minsmere – Dunwich Cliffs (due to a high Dunwich Bank), resulting in maintenance of the current level of sediment supply (primarily regional supply of sand). The Minsmere Outfall is assumed to be absent or causing minimal disruption to longshore transport. The broad trend would be erosive, but with local increases in shingle availability due to erosion of the Central Barrier. See the red line in Figure 74 for an illustration to accompany the following text.

In this scenario, scarping north of the former Minsmere Outfall would be replaced by roll back. The Coney Hill cross-bank clay structure would be prone to erosion and offer little disruption to longshore transport. The red shoreline shown on Figure 74 represents the most eroded form of the two end members.

The legacy of the Minsmere Outfall (i.e., its stabilising effect whilst still present in the beach) and the high elevation/volume barrier to its south, would mean that barrier scarping (not roll back) would occur/accelerate on the 1500-m-long Central Barrier frontage. On the Southern Barrier, for a kilometre north of the HCDF, the lower elevation/volume barrier would be prone to overtopping and gradual roll back, however it would receive shingle eroding from the Central Barrier and the former sluice promontory.

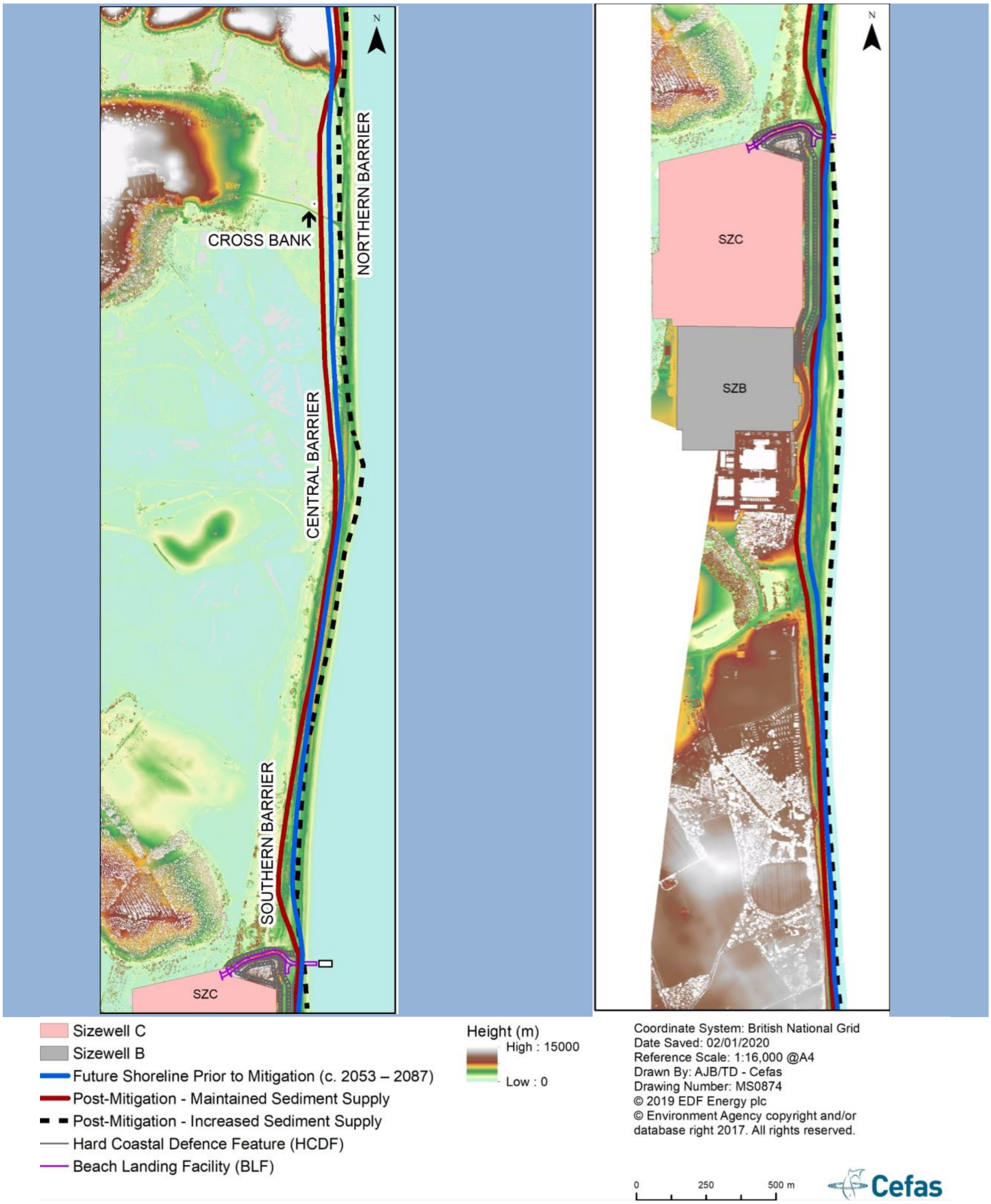

Figure 74: Future shoreline configuration after mitigation has ceased for maintained and increasing sediment supply scenarios.

Although the Southern Barrier has a similar elevation to Northern Barrier (that would be in roll-back mode at this time), its volume and width are around twice the size, suggesting greater resilience and less likely overtopping and breaching.

The HCDF would act as a partial or total barrier to longshore shingle transport. As a result, shingle (including that released from the former sluice promontory) would build up to the north over time, slowing erosion and/or stabilising (in net terms) the shoreline there. Over time this process could halt any roll back to the immediate north and is likely to prevent exposure of the SSSI crossing to wave attack. The Sizewell C development is in the lee of Sizewell A and Sizewell B with respect to northerly shingle transport (southerly storms), and, as discussed in Section 7.4.2.2, offers only a minor obstruction to northward longshore transport. When combined with accumulating sediment north of the HCDF, it is unlikely that Sizewell C would have an adverse impact on the Minsmere frontage and supra-tidal shingle habitats (should they still exist). A more likely cause of reduced supply to the Minsmere frontage would be exposure of the Sizewell A defences, which is not an impact due to Sizewell C.

Assuming a natural beach was not retained in front of the HCDF post-mitigation, a partial or total blockage would lead to alternating event-based starvation in the downdrift direction. To the north, the HCDF impact is likely to be minimal because it does not offer a significant barrier to northward moving shingle and because, in net terms, the frontage north of the HCDF would be slowly building up a reservoir of shingle. At Sizewell C and to the immediate south, beaches would be narrow and partially or wholly absent as a result of general slow recession due to sea level rise, coastal squeeze (which applies to the hard defences of all three power stations) and finally due to a reduction in shingle supply during individual events (i.e., gross transport). Whilst coastal squeeze would occur against the HCDF, a similar effect could be encountered in the no-Sizewell C case when the shoreline retreated to the Bent Hills. Note that the Sizewell C frontage does not have an annual vegetated shingle habitat as the supra-tidal shingle is narrow, constrained between the intertidal beach and the occasionally scarping barrier.

The balanced net shingle transport would mean the extent of starvation to the south would be defined by the typical range of gross transport events - which is less than a kilometre (BEEMS Technical Report TR420). As the power station's (SizewellA, Sizewell ZB, and Sizewell C) frontage is approximately 1500 m long, starvation during individual events would primarily be contained to that frontage. However, if the blockage were permanent, several decades of additional starvation could lead to an erosion impact on the Leiston – Aldeburgh SSSI frontage (were it still to be designated). It is worth noting however that the timescale involved is around 100 – 150 years in the future, by which time the same impact is very likely to have occurred as a result of exposed Sizewell B defences (in the absence of Sizewell C). Therefore, Sizewell C is not considered to affect the Leiston – Aldeburgh SSSI in a fashion that would not have occurred as a result of another development (Sizewell B), though it may exacerbate it.

7.7.2.2 *Case 2: increased sediment supply after cessation of beach maintenance (mitigation)*

The driving parameters for this case are increased sea levels with a gradually rising regional sediment supply through time (due to larger regional (Kessingland – Easton cliff erosion) and local shingle and sand supply (due to erosion of Minsmere – Dunwich cliffs under a lower Dunwich Bank). The Minsmere Outfall is assumed to be absent or causing minimal disruption to longshore transport. The broad trend would be patches of lesser erosion via scarping and roll back due to the higher counter-balancing sediment supply (compared to the Section 7.7.2.1 case). Higher spatial and temporal variability is also likely to result in accordance with erosive episodes on the Minsmere – Dunwich cliff frontage. See the blue line in Figure 74 for an illustration to accompany the following text.

The basic patterns would be similar to those described in Section 7.7.2.1, but with lower rates of retreat. The combination of high rates of supply (regional and local) would locally slow erosion everywhere and, when combined with the HCDF trapping effect, would significantly reduce the likelihood of roll-back and breaching north of the HCDF. Permanent exposure of the HCDF itself would be less likely, with high sediment supply maintaining a narrow beach along the power station's frontage. The presence of shingle beaches would allow shingle transmission, making adverse effects on the Leiston – Aldeburgh SSSI substantially less likely compared to Case 1. The effect of the HCDF on the supra-tidal shingle habitats of the Minsmere SAC/SPA would be largely positive, through its shingle trapping and stabilising effect.

Consequently, any long-term impacts due to HCDF exposure would be lesser under a higher sediment supply (e.g., eroding Minsmere – Dunwich Cliffs) scenario.

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Appendix A Natural England condition survey for Unit 113

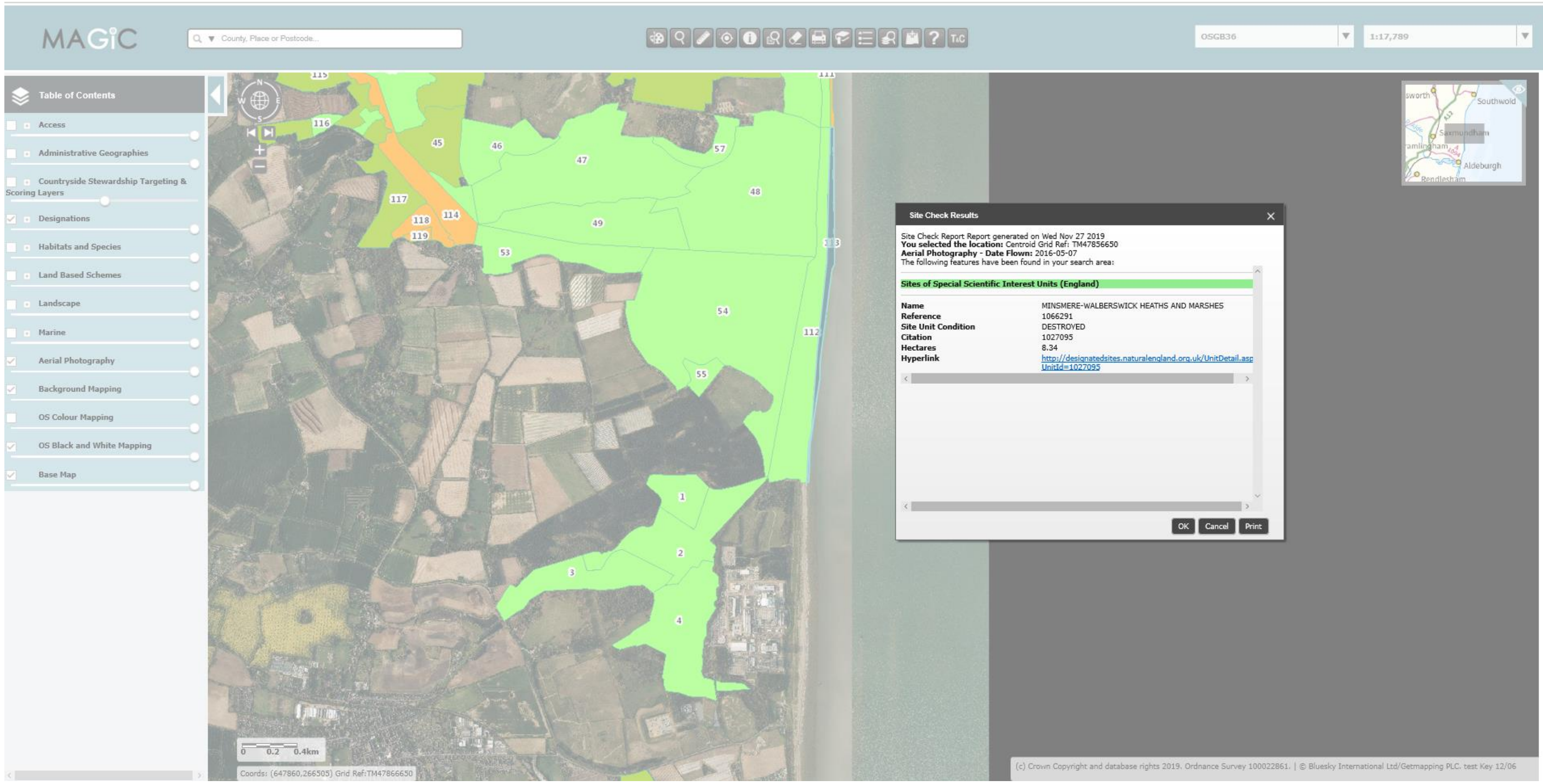

Figure A. 1: Location map for Unit 113 – Minsmere – Walberswick Heaths and Marshes SSSI – Minsmere haven shingle (source: DEFRA MAGIC).

NATURAL ENGLAND

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Minsmere-Walberswick Heaths and Marshes SSSI - MINSMERE HAVEN SHINGLE (PSRT UNIT 56 PRE-FEB 02) (113)

Staff member responsible:	Edward Boyle		
Unit Id:	1027095		
Unit area (ha):	8.3419		
Unit Status:	Live	**Gridref:**	TM 478 659
Main habitat:	SUPRALITTORAL SEDIMENT		
Condition and Comments (click for history):	Destroyed	**Assessed by:**	HAY, (EMMA)
Last assessed:	17/08/2011	**Last assessment field visit:**	17/08/2011
ISA Survey:	View Surveys		
Date of last site check:		**Last CSM assessment:**	17/08/2011
Estimated year unit will go Favourable:		**Confidence in estimate:**	
Reason for adverse condition:	COASTAL - COASTAL SQUEEZE,PUBLIC ACCESS/DISTURBANCE - PUBLIC ACCESS/DISTURBANCE,		
Comment:	This unit has been lost through coastal erosion. Aerial photography shows that the beach was previously much wider. Overlaying of the SSSI unit on the 2007 aerial photograph showed most of the unit was gone apart from its northern end. The ISA site visit confirmed that all the unit had gone or if deposition had started to occur again no shingle habitat was present. It is estimated that the coast to the north at Dunwich is eroding at 0.75m pa, EA and EDF monitor the coastline. The remaining beach (Unit 112) shows evidence of ridge formation and is shelved but not as steeply as other parts of the coast such as Thorpeness and Aldeburgh but more so than Dunwich Beach. It may be that Unit 113 has rolled back into Unit 112. A sea wall divides Minsmere Scrapes (unit 48) from the beach (unit 112) and supports dune communities. It is likely that if the shingle is trying to rollback back this is inhibitated by the seawall and therefore coastal squeeze could be an issue for unit 112. The beach (unit 112) is trampled throughout, heavily at access points such as the sluice. Annual shingle vegetation was evidence but appeared to be a single species of Atriplex (Atriplex prostrata). The target should be reconsidered as other shingle sites have shown a low diversity of species (notably Orfordness, in discussions with Sue Rees). Perennial shingle vegetation was present including Rumex crispus, Crambe maritime and Glaucium flavum, all abundant or frequent. Bitter stonecrop and sea sandwort present. A Little tern fence had been erected on Unit 112 but they had failed to breed this year (and in recent years). This ISA concludes that the unit has been destroyed through natural processes and the unit should be removed from the site accordingly, unless it is deemed that accretion might occur and the unit returns (but with no vegetation feature!?).		
Number of adverse condition reasons:	2		

Figure A. 2: Information on unit 113 as provided by Natural England (source: DEFRA MAGIC).

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Minsmere-Walberswick Heaths and Marshes SSSI - MINSMERE HAVEN SHINGLE (PSRT UNIT 56 PRE-FEB 02)
Condition History

Assessed	Condition	AssessedBy	CSM assessment date	VisitedBy
17/08/2011	Destroyed	HAY, (EMMA)	17/08/2011	HAY, (EMMA)
01/12/2010	Partially destroyed	JACKSON, (JOHN)	01/12/2010	BURROWS, (ADAM)
29/11/2010	Partially destroyed	JACKSON, (JOHN)	19/08/2004	BURROWS, (ADAM)
24/11/2010	Partially destroyed	JACKSON, (JOHN)	19/08/2004	BURROWS, (ADAM)
25/08/2004	Unfavourable - Declining	COOPER, (GLEN)	19/08/2004	COOPER, (GLEN)
06/08/2002	Favourable	SMITH, (DUNCAN)	06/08/2002	SMITH, (DUNCAN)
12/12/2000	Favourable	SMITH, (DUNCAN)	12/12/2000	SMITH, (DUNCAN)

Figure A. 3: Condition history of unit 113 (source: DEFRA MAGIC).