Geomorphological Assessment of SMP2 Coastal Management Area ORF15.1: Martello Tower to Orford Ness, Suffolk

Report prepared for the Alde & Ore Association

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Kenneth Pye Associates Ltd.

Scientific Research, Consultancy and Investigations

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Summary

This report was commissioned by the Alde & Ore Association to inform discussions relating to a possible change in SMP2 policy for Management Area ORF 15.1 (Martello Tower to Orford Ness). The current preferred management policy for ORF 15.1 is Hold the Line (HTL) in SMP2 Epoch 1(up to 2025) and No Active Intervention (NAI) in Epoch 2 (2025 - 2055) and Epoch 3 (2055 - 2105). The SMP2 noted that this is an interim policy pending an agreed Management and Investment Plan for the Alde and Ore area. The Final Draft Alde and Ore Estuary Plan (AOEP, 2016) seeks to make improvements to the river walls such that they will withstand a 1 in 200 year tidal surge in the year 2050, and there is concern that a breach in the shingle barrier between Aldeburgh and Orford Ness might compromise this objective.

Several approaches have been used in this study: (1) analysis of Environment Agency (EA) topographic and bathymetric profile data, LiDAR data, aerial photographs and wave data, (2) site visits and a RTK GPS survey on 6 February 2016, (3) assessment of barrier stability using the Bradbury (2000) and Obrahai *et al*. (2008) models, and (4) assessment of the likelihood of subsequent inlet formation and inlet stability, including regime modelling and analysis of results from previous alongshore sediment transport modelling.

It is concluded that overwashing of the present artificial shingle ridge is unlikely along most of its length but erosional cut back of the seaward face of the unprotected section of the barrier opposite the northern end of Lantern Marshes is very likely during sustained stormy periods, especially when waves are directed from an ENE direction. If cut-back occurs to the point where the beach scarp intersects the back slope of the ridge, overwashing will occur and the ridge is likely to experience catastrophic failure at the point where it is narrowest. Under surge tide conditions waves would cross the flattened barrier and impact on the river wall along the west side of the northern Lantern Marshes, forming a wide gravel spread. Under a policy of NAI further storm overwashing would lead to breaching of the river wall, allowing shingle loss into the river and wave impact on the Sudbourne Marshes river wall.

Although the eastern bank of the river lies very close to the maintained shingle ridge in this area, development of a permanent breach, leading to capture of the upper Alde, is considered unlikely in the short term (<20 years) but could occur by 2055 if the undefended shore immediately south of the existing rock armour erodes by c. 30 - 40 m as a result of successive storms. If a breach is cut down through the underlying marsh sediments to mean sea level the tidal velocities through the breach would be high enough to cause further deepening and keep the breach open. The formation of a permanent inlet would have very significant consequences for the whole estuary, including a reduction in the width, depth and cross-sectional area of the existing estuary entrance, greater sedimentation in the lower estuary, and an increase in high water levels and current speeds in the upper estuary which would increase flood risk, lead to intertidal erosion and impact on the river walls at

Sudbourne Marshes. Further hydrodynamic and sediment transport modelling is required to quantify the potential impacts in more detail.

Given the risks associated with a policy of NAI for unit ORF 15.1 in SMP2 Epoch 2 (2025 - 2055), there is a strong case for a change in management policy while longer term options are considered. HTL might be achieved by extending the present rock armour revetment southwards, accompanied by maintenance of beach levels through shingle nourishment if sediment volumes fall below critical levels. The shingle embankment should also be realigned landwards slightly and given a lower angle seaward slope to reduce cliffing during storms. Additional shingle should be imported and placed at the back of the barrier to increase the barrier width and sediment volume. Under a NAI scenario it very unlikely that a 'stable' ridge with sufficient height and width to prevent frequent overwashing would form through natural rollover. Under a policy of managed realignment (MR) a high maintained ridge could be moved backwards in stages but this would require frequent import of shingle to maintain the barrier, and the neck of land between the sea and river would be progressively narrowed, increasing the risk of inlet development over time.

It is recommended that a programme of simultaneous data capture is undertaken within the estuary and in the nearshore zone between Slaughden and the northern end of Sudbourne Beach to provide the data necessary for calibration/ validation in further modelling investigations, and to facilitate data-driven assessments.

The programme of data collection should include re-deployment of an AWAC off Sudbourne Beach and simultaneous deployment of instrumentation to quantify changes in wave parameters as they approach the beach (e.g. using an Argus camera or X-Band radar).

When nearshore / offshore swath bathymetric data for the Slaughden - Sudbourne Beach area (planned as part of the EA 2016-2021 monitoring programme) become available, additional modelling of wave run-up and barrier response should be undertaken using a model such as XBeach-G (McCall *et al*., 2013, 2014; Masselink *et al.*, 2014).

In addition, in order to provide better information about synoptic variations in water levels and depth-related currents along the length of the estuary, a series of instruments should be deployed at up to seven stations along the estuary. These data would allow the development of a refined 2D / 3D model of the estuary which could then be used to assess the impact of a potential breach at Sudbourne Beach or elsewhere, and of other changes within the estuary (e.g. raising of the river walls, managed or unmanaged realignment). Modelling of wave penetration into the estuary under different potential breach scenarios, and implications for overtopping of the river walls and intertidal erosion along Sudbourne Marshes, is also required.

In order to maximise the utility of additional data collection and assist the planning of further modelling, it is recommended that a meeting / workshop should be organized to integrate the requirements / views of interested parties.

1.0 Report scope and purpose

This report was commissioned by the Alde & Ore Association (A&OA) to inform discussions relating to a possible change in SMP2 Policy for Management Area ORF 15.1 (Martello Tower to Orford Ness). Management Area 15 forms part of SMP Policy Zone PDZ 5 and is divided into two sub- policy units, ORF 15.1 (Sudbourne Beach) and ORF 15.2 (Orford Ness). The boundary between ORF 15.1 and ORF 15.2 lies at Chainage 49, opposite the northern part of Lantern Marshes and the Blackstakes Reach of the River Alde, while the northern boundary of ORF 15.1 lies just to the south of the Martello Tower at Chainage 46 (Royal Haskoning 2010; Figure 1). The current preferred management policy for ORF 15.1 is Hold the Line (HTL) in SMP2 Epoch 1(up to 2025) and No Active Intervention (NAI) in Epoch 2 (2025 - 2055) and Epoch 3 (2055 - 2105). The SMP2 noted that this is an interim policy pending an agreed Management and Investment Plan for the Alde and Ore area.

The Final Draft Alde and Ore Estuary Plan (AOEP, 2016) seeks to make improvements to the river walls such that they will withstand a 1 in 200 year tidal surge in the year 2050, and there is concern that a breach in the shingle barrier between Aldeburgh and Orford Ness might compromise this objective. Concern has also been expressed that the creation of a permanent tidal opening in this area would have severe adverse effects on the navigability of the River and an unquantifiable negative impact on habitats and the local economies (Zins & Marson, 2004). For these reasons, the AOEP is seeking a change in the management policy for ORF 15.1 in Epoch 2 from NAI to HTL. This would be consistent with the preferred policy for management unit ALB 14.4, Aldeburgh Town (Chainage 45) to Martello Tower (Chainage 46.5).

This report presents an assessment of the following:

(1) the current geomorphological condition of the shingle beach and barrier system within units ORF 15.1and ORF 15.2

(2) historical to recent changes in the form and position of the barrier system

(3) the risk that the barrier will be breached or over-washed under present management (HTL) and possible NAI after 2025

(4) the risk that a breach might develop into a self-sustaining tidal inlet

(5) the possible consequences of such a change for the hydrodynamic and sediment transport regimes of Alde - Ore estuary as a whole, and consequently on estuary morphology, habitats and flood risk.

2.0 Methods

Several approaches have been used in the preparation of this report:

(1) data analysis (data sets include Environment Agency topographic beach profile and subtidal profile extension data, LiDAR surveys between 1999 and 2015, aerial photographs and inshore wave data for Slaughden South)

(2) a Real Time Kinetic Global Positioning System (RTK GPS) survey on 6 February 2016 and a further site inspection visit on 19 May 2016

(3) assessment of wave overtopping risk and barrier stability using the Bradbury (2000) and Obrahai *et al*. (2008) models

(4) inlet stability assessment, including the results of previous regime modelling, alongshore sediment transport modelling, hydrodynamic modelling and morphological modelling (Halcrow, 2005; Hartley & Pontee, 2008; Pye & Blott (2014a, 2015a,b) and Pye *et al.* (2015)

(5) Expert geomorphological assessment (EGA) (Pye & van der Wal, 2000) which incorporates the results of (1), (2), (3) and (4), previous studies of the Alde-Ore estuary (Pye & Blott, 2005, 2007, 2015a) and barrier breaching on the Suffolk coast (Pye & Blott, 2006, 2009), and wider ranging reviews of coastal processes, sediment transport and morphological evolution in the area (Steers, 1926; Carr, 1969, 1970, 1973; Green & McGregor, 1988, 1990, 2012; McGregor & Green, 1990; Randall & Fuller, 2001; May 2003; Halcrow, 2005; Royal Haskoning, 2009a,b; HR Wallingford, 2016).

Beach and barrier sediment volumes were determined from quantitative analysis of LIDAR digital elevation models (DEMs) for each available survey epoch in order to identify patterns of temporal change and to identify sections of the barrier which are most vulnerable to breaching. For this purpose the barrier between Fort Green and Orford Ness was divided into 136 ‘cells’, within which the sediment volume of the barrier above +1.6 m ODN, and the volume of the beach between -0.08 m and +1.6 m ODN, were calculated. The boundaries of the cells are shown in Figure 2. Values of the barrier inertia criterion B_i and the risk of overwashing within each cell were determined for selected combinations of surge tide and storm wave conditions using the predictive formulae proposed by Bradbury (2000) and Obhrai *et al*. (2008). Tide and wave conditions for the assessment were chosen following an analysis of water level data for the Environment Agency (EA) tide gauge at Orford Quay and the EA Sudbourne Beach Acoustic Wave and Current (AWAC) recorder which was operational between 2006 and 2009 (see Appendix 4).The environmental conditions responsible for historical periods of barrier erosion and overwashing have been valuated and an assessment made of the conditions which would be required to form a ‘permanent’ breach under management policies of HTL and NAI. The potential implications of such a breach on water levels and current speeds within the estuary are considered in the light of previous hydrodynamic modelling results (Black & Veatch, 2004; JBA 2011; Pye & Blott, 2014a; Pye *et al*., 2015). However, further 2D and 3D hydrodynamic and sediment transport modelling of the estuary is required to fully assess these potential impacts.

3.0 Data analysis

3.1 History of breaching, overwashing and coastal management

Historical maps, charts and historical records indicate that the coast between Aldeburgh and Orfordness has been eroding episodically for hundreds of years (Carr, 1969, 1971, 1973; Pye & Blott, 2005, 2007). During the past 120 years the position of the shoreline in front of Aldeburgh Town has shown little net change, but the shingle ridge between Aldeburgh and the northern side of Orfordness has experienced landward recession along much of its length and has been overwashed on a number of occasions, including during storm surges in 1897, 1911, 1938, 1949, 1953, 1963 and 1978.

Maps published by the Ordnance Survey in the later 19th and early 20th centuries, and early photographs taken during the 1920s and 1930s, show a relatively wide beach at Aldeburgh and a mobile shingle beach and barrier system with washover fans extending southwards towards Orfordness (Figures A1.1 and A1.5 in Appendix 1). In 1890 Slaughden consisted of a cluster of about 20 buildings, including the Mariner's Inn which was abandoned after a surge tide in 1911 and finally demolished in 1920. The outer wall around the Martello tower and much of the gun battery in front of it were intact. Further south, along Sudbourne Beach, the ridge crest and upper beach were used as a Rifle Range. An isolated cottage (Sheep cottage) and associated sheep folds were located next to two fields on reclaimed saltmarsh behind the shingle barrier. A track followed the crest of the barrier along Sudbourne Beach towards Orfordness, but the barrier had a mainly natural profile with large areas of bare shingle extending up to 100m landward of the mean high water spring (MHWS) tide line.

The beach at Aldeburgh and Slaughden experienced significant erosion and narrowing in the 1920s and 1930s. Following storms in February 1938, including a significant surge tide on 12 February, recommendations were made for the construction of a groyne system but no action was taken before the outbreak of World War II. During the War the beach around Aldeburgh was extensively fortified with tank traps and barbed wire. Aerial photographs taken by the RAF in 1945 show a wide, bare gravel ridge extending between Aldeburgh and the north side of Orford Ness (Figure A1.2 in Appendix 1), with washover fans located within Aldeburgh Town Marshes, at Slaughden and along Sudbourne Beach which appear to have been created or reactivated during the 1938 storm surge. Construction of a system of wooden groynes with sheet piling foundations was started in 1947, but the lack of a rear wall meant that erosion continued between the groyne bays and the beach continued to narrow (Dobbie & Partners, 1985, 1986; Babtie Dobbie, 1991a,b; Pontee, 2005). A further storm surge on 8 January 1949 overtopped the shingle bank south of Fort Green and activated two large washover fans, one extending into Aldeburgh Town Marshes and the other reaching the eastern bank of the River Alde between the Aldeburgh Sailing Club and the Martello tower (Figure A1.2 in Appendix 2). Other, smaller, washover lobes were formed or reactivated between the Martello tower and Orfordness. Following this event further groynes were installed and a section of concrete sea wall built. A higher magnitude storm surge event on 31 January – 1 February 1953 caused the sea to break through the new defences south of Fort Green, reactivating and extending the existing washover fans. Water also flowed seawards across the barrier for a short time at the beginning of the ebb tide but the River Alde did not break through. The flattened sections of the shingle ridge were quickly bull-dozed back from the landward side to create a new, thinner ridge located approximately 45 m landward of its previous position (Steers, 1953).The groyne system and concrete sea wall were subsequently repaired and improved in 1954-55.

Following a further breach in 1963, in 1965-66 the groynes fronting the Brudenell Hotel were rebuilt, almost all of the groynes were raised, and approximately 250,000 m^3 of shingle were transported into the Aldeburgh - Slaughden area from the northern part of Orfordness by the East Suffolk and Norfolk River Authority (ESNRA) (Kinsey, 1981; Taylor & Marsden, 1983; Pontee, 2005). Along the King's Marshes frontage the shingle was bulldozed back towards the shore to raise the defences by the East Suffolk and Norfolk Rivers Authority (ESNRA) under contract to the Ministry of Defence (Fuller & Randall, 1988). Further erosion of the beach and shingle ridge south of Fort Green occurred in the 1970s, and parts of the unprotected shingle bank were flattened during a storm surge in January 1978 (Steers *et al.*, 1979; Pontee, 2005). By 1983 most of the recharge sediment placed in the mid-1960s had disappeared (Taylor & Marsden, 1983; Figure A1.3 & A1.8 in Appendix 2). By the mid-1980s the groynes fronting the sea wall were in a dilapidated condition and failing to retain shingle. Storms during the winter of 1988 exposed the toe of the sea wall and abrasion by moving shingle created holes in the sheet piling foundations, allowing erosion of the fill behind (NRA, 1991).

At this time the formation of a breach was considered unacceptable due to the costs of managing the breach, strengthening the flood embankments around the estuary, the consequences of likely sedimentation due to formation of a shingle bar across the existing mouth of the estuary at North Weir Point, and the adverse and largely unknown ecological consequences (Dobbie & Partners, 1985, 1986; HR Wallingford, 1986, 1988; Babtie Dobbie, 1991; Pontee, 2005). Following the transfer of coastal management responsibility from Anglian Water to the National Rivers Authority in 1989, major improvements were made to the defences at Slaughden in 1991-92. The northern part of the seawall was repaired and additional protection provided for the most vulnerable sections in the form of a rock armour apron. The southernmost 200m of the sea wall south of the Martello tower, including a terminal groyne, was demolished and replaced with rock armour along a new (set-back) alignment. New hardwood groynes were built in front of the sea wall, with rock armour placed in front of the sea wall in some groyne bays around the Martello tower, and 75,000 m^3 of marine shingle from Orfordness was imported to fill the groyne bays. Most of the old groynes between the Martello tower and Fort Green were left in place.

A programme of maintenance beach nourishment at Slaughden, using material taken from the northern part of Orfordness, was initially proposed for a 15 year period in 1986 (Babtie Dobbie, 1986). It was initially envisaged that beach nourishment would be required every 5 years, but by 1992 re-cycling of approximately 10,000 m^3 was being undertaken almost annually. Despite concerns raised by English Nature, and later Natural England, that removal of shingle was damaging the vegetated shingle at the donor site opposite Lantern Marshes, the effects have been considered to be relatively minor (Halcrow, 2002a,b) and the practice of shingle recycling has continued to the present. During the 1990s shingle recycling operations were largely carried out in the spring each year, but in more recent years the operations have principally been undertaken before the start of the winter to provide better beach protection against winter storms, and to avoid disturbance to nesting birds and other wildlife on Orfordness.

The recycling operations have assisted in maintaining beach levels but sediment has continued to be lost from some groyne bays, notably during periods of heavy seas from the east north-east. Severe erosion of the undefended shingle ridge has also occurred during such conditions, notably during the winters of 1989-90, 1990-91, 1992-93 and 1996-97, 2005-06, 2006-07, 2012-13 and 2013-14. During March 2013 a large quantity of shingle was lost from the groyne bays opposite the Aldeburgh Yacht Club, north of the Martello tower, and further sediment losses occurred during the stormy winter of 2013-14, locally exposing the toe of the sea defences. The beach levels in this area did not subsequently recover naturally, and waves were able to break directly against the concrete sea wall, resulting in wave-splash overtopping which led to local failures on the landward side of the shingle embankment in early 2016. A scheme to provide 4,500 tonnes of rock armour protection, infilled with stockpiled shingle, along a 200 m long section of the defences adjoining the northern end of the existing rock armour protection in front of the Martello tower was begun in May 2016. The period of ENE waves in March 2013, followed by further storm waves during the winter of 2013-14, also caused serious erosion of the beach and barrier face to the south of the defended frontage, beyond the limit of existing rock armour, and in places the barrier crest was reduced to only 1.5 m. However, beach recovery occurred before emergency intervention was undertaken.

3.2 Environment Agency beach and nearshore monitoring

Strategic topographic monitoring of beach profiles at approximately 1 km intervals along the Anglian coast, including the Aldeburgh - Orfordness frontage, was initiated by the National Rivers Authority Anglian Region in 1991 and has subsequently been continued since 1996 by the EA Anglian Region. Strategic beach surveys are undertaken twice a year, usually in spring and autumn. Additional ‘scheme specific’ monitoring profiles have also been surveyed in some locations, including Slaughden). Single beam echo sounder profiles have been conducted as offshore extensions of the beach profiles in 1992, 1997, 2003, 2007, 2008, and 2013, although not all profile lines were surveyed in each campaign. Multibeam swath bathymetry surveys of the Aldeburgh Ridge and the Thorpeness to Fort Green nearshore areas were undertaken in September 2013 and July 2014, respectively. A similar survey of the Fort Green to Orfordness nearshore area has not been undertaken but is planned as part of the 2016-21 monitoring programme. Vertical aerial photography and LiDAR surveys have been commissioned at intervals since 1991, although not all provide complete spatial coverage. A full analysis of the monitoring data is beyond the scope of this report but selected beach and nearshore profile data are presented in Appendix 2. Additional profile data extracted from the LiDAR surveys, and obtained from a KPAL ground survey in February 2016, are presented in Appendix 3. Comparison of the LiDAR survey with ground survey data allowed determination of the errors associated with the LiDAR surveys. Based on this analysis (Figure 3), the earlier LiDAR data were adjusted by appropriate correction factors prior to profile extraction and calculation of sediment volumes from the DEMS. A previous overview of coastal trends up to 2011 was published by the EA Anglian Region Shoreline Management Group (EA, 2011) and changes at selected profiles along the Slaughden and Sudbourne frontage have been previously discussed by Pye & Blott (2015b).

The crest height of the artificially maintained section of the barrier behind the sea wall between Fort Green and south of the Martello tower, determined from the January 2015 LiDAR survey, varies from just below to just above 5.0 m ODN. Beyond the end of the sea wall the height of the maintained barrier crest (haul road) increases to c. 5.5 - 6.0 m ODN in cells 24 to 33, before falling back to c. 5.0 m ODN between cells 36 and 50. Most of this section of the barrier is not protected by groynes or rock armour. To the south of cell 50 the maintained crest height generally exceeds 6.0 m ODN and the barrier width is much greater (Figure 4). The crest height of the maintained barrier south of the defended frontage has always been low (c. 5.0 m OD), even though this section of the barrier lies closest to the river. However, the volume of sediment in the barrier above HAT level (1.6 m ODN) in this area increased between 1999 and 2009 due to seaward progradation of the barrier, a trend which extended along the whole frontage between cell 38 and cell 110 (Figures 5 & 6).

Figure 7 shows the alongshore variation in beach sediment volume (below 1.6 m ODN) at the time of the January 2015 LiDAR survey. Beach volumes were particularly low between Aldeburgh Yacht Club and the Martello tower, reflecting loss of sediment from these groyne bays during the period 2010 -2013, and its subsequent failure to return. South of the Martello tower the volume of beach above -0.8 m ODN showed a general declining trend towards cell 100, broken only by a small increase near the wall separating Lantern Marshes north and Lantern Marshes south (Figure 7). By the time of this survey significant beach recovery had taken place along this frontage following major sediment losses during 2013 (Figure 8).

The EA bathymetric data for the profile extensions presented in Appendix 2 clearly show that the Aldeburgh Ridge, which has an approximate SSW - NNE orientation, extended northwards, increased in crest height and moved shorewards between 1992 and 2014. Major differences exist compared with the bathymetry represented in the Seazone Trudepth data set, based substantially on surveys in the 1980s, which was used in the sediment transport modelling undertaken by Halcrow (2005) and HR Wallingford (2016). In 2014 the northern end of the bank was located opposite profiles S1A5A and S1A15 and the crest height of the bank opposite upper Lantern Marshes was much higher, approaching - 3.5 m ODN at profile S1A7A but decreasing northwards. The growth of the bank since the 1980s would be expected to have a significant effect on wave heights and approach directions along the Slaughden and Sudbourne Beach shoreline, and on resulting alongshore sediment transport rates. Northward extension of the bank may have reduced the effectiveness of southeasterly waves and northward alongshore sediment transport and/ or caused greater focusing of northeasterly wave energy on the Sudbourne Beach frontage , but this requires further investigation by additional modelling when the collection of swath bathymetry data for the inshore area is completed.

4.0 Barrier stability modelling and assessment

4.1 Nature of the governing processes

Natural shingle barriers experience periodic overwashing during storms, and the crest height of a natural shingle ridge is determined principally by the most extreme combination of wave and tide conditions experienced during the formation of the ridge. Sediment availability, sediment size, sorting and packing arrangement, which influence water infiltration into the barrier and wave energy dissipation, also exert an influence on shingle ridge height (Pye, 2001). If significant amounts of water overwash the barrier crest there is a risk of flooding on the landward side. However, this tends to be limited in extent and whole-scale flooding of a large back-barrier area normally only takes place if the barrier crest is lowered significantly by wave action during a major surge tide, or the barrier is otherwise breached.

The risk of overwashing of a coastal barrier is strongly influenced by the 'freeboard' that the barrier maintains against wave run up during storms. Positive freeboard exists where the maximum run up limit elevation (R_{max}) is less than the maximum height of the barrier crest (B_h), while negative freeboard exists where $R_{max} > B_h$ (Orford *et al.*, 2003). If $R_{max} > B_h$ sediment is likely to be transported from the upper beach face towards the barrier crest, which may be slightly overtopped before the water percolates into the ridge and deposits the sediment, gradually raising the crest level to a point where $R_{max} = B_h$ and a new temporary equilibrium is attained through a negative feedback process. This situation is similar to the 'Type 1 Overwashing' recognized by Nicholls (1985) which occurs without a reduction in crest height. However, if R_{max} greatly exceeds B_h sediment is likely to be transported over the crest and down the backslope of the barrier. During the early stages of a storm tide periodic 'sluicing' overwash may occur, becoming more continuous as the still water level rises and/

or waves become more energetic (Orford *et al*., 1991). Scour and mobilization of sediment on the crest may reduce B_h, increasing the amount of overwash through positive feedback. This process, which equates to the 'Type 2 Overwashing' recognized by Nicholls (1985), forms washover fans on the landward side of the barrier.

In natural systems, B_h usually shows a degree of alongshore variation, and overwashing is likely to be initiated at low points ('throats') in the barrier crest. Depending on nearshore wave conditions, alongshore variations in wave height (and sometimes period) may also occur (e.g. due to edge waves which create alongshore variations in breaker height), with overwash taking place preferentially at locations where the wave energy and runup are greatest. In circumstances where washover throats are closely spaced the washover fans may coalesce to form an almost continuous washover apron along the back of the barrier (Stripling *et al*., 2008). If the barrier crest is lowered sufficiently below the prevailing still water level, waves may pass right across the barrier without fully breaking. In such circumstances gravel can be transported several hundreds of metres in the landward direction before being deposited, and the waves may have sufficient energy to cause breaching or overtopping of earth embankments behind. If the land / water level behind the barrier is lower than the still water level on the seaward side, the tide may pour over the flattened section of barrier into the back-barrier area, causing scouring and formation of an incised channel. Further deepening of this channel (or multiple channels) may occur on the ebb tide, or subsequent flood and ebb tides, as water flows out and in through the breach(es). These processes were illustrated during and after the breaching of the Dunwich - Walberswick barrier between 2005 and 2007 (Pye & Blott, 2009a).

Other processes may also contribute to movement of sediment on a shingle barrier. On barriers which are protected by concrete structures or rock armour, water thrown over the barrier crest after wave breaking may run down the landward side of the barrier, causing scouring. This process occurred recently in 2015 and 2016 at Slaughden and caused collapse of part of the landward side of the artificial shingle embankment in front of the Aldeburgh Yacht Club. In the case of coarse grained gravel barriers which have no impermeable central 'core', sustained high water levels on the seaward side which are not matched by similar water levels on the landward side can generate a pressure gradient which forces water to flow through the barrier, causing scour at a 'spring line' on the landward side. This may undermine the base of the landward slope and lead to large scale slumping; in extreme cases an entire section of shingle bank may fail due to progressive erosion on the landward side, even without overwashing.

When R_{max} is $< B_h$, sediment eroded from the upper beach slope by storm waves is moved seawards by wave backwash, lowering the upper beach level and causing a scarp to form on the seaward side of the barrier (a process often referred to a 'crest cut-back'). Landward recession of this scarp from the seaward side can, in the case of narrow barriers, completely erode through the crest into the backslope, reaching a point where overwashing becomes possible. Funnelling of wave-induced flow through the 'throat' causes it to widen and deepen, and sometimes leading to a sudden 'breakthrough'. In rare instances, high water levels on the landward side of a barrier, caused by river floods in estuaries or high lagoon water levels, accompanied by waves generated by winds from the land, may cause seaward directed breaching at low points in the crest of a barrier, or there may be a combination of landward and seaward directed breaching (Cope, 2004, 2006). In view of the above, barrier width and total sediment volume, not just crest height or 'freeboard', are important controls on breaching risk.

Only limited observational data exist relating to wave conditions and shingle barrier response during storms. Nearshore wave and run-up data obtained near Worthing have recently been used to develop a new predictive tool which utilises wave spectral data (HR Wallingford, 2014), but the model does not include information about beach sediment grading and packing, or the level of beach water table, both of which exert a significant influence on wave infiltration and energy dissipation. Sedimentological studies have shown that most shingle and mixed sand-shingle beaches display significant variations in sediment size distribution and packing, cross-shore, alongshore, with depth and over time (Pye, 2001). However, the available models of shingle beach and barrier behaviour only contain simple representations of sediment properties, such as the D_{50} particle size (e.g. Powell, 1990; McCall *et al*., 2013, 2014; Masselink *et al.*, 2014).

Longer-term changes in the morphological state of a barrier are essentially controlled by the sediment budget of the fronting beach and nearshore zone. Where the average beach sediment budget is positive, a barrier will grow to an equilibrium crest height and then a new accretionary ridge will begin to form on the seaward side. If the average beach sediment budget is negative, the barrier will show a tendency for overwash and landward retreat, with or without a reduction in sediment volume. Whether the barrier rolls back, maintaining essentially constant form and sediment volume, or becomes lower and flatter, possibly with an accompanying reduction in sediment volume, depends on the magnitude of the local sediment budget deficit, on the frequency and magnitude of storms which cause overwashing and/ or breaching, and on the frequency of conditions which encourage sealing of breaches and barrier re-building. Closely-spaced (in time) major storm surge events, or high rates of sea level rise, can cause the upper part of a barrier to move landwards rapidly, leaving a remnant of the barrier on the shoreface, a process referred to as 'overstepping' (Orford & Carter, 1995; Forbes *et al*., 1991).

Hartley & Pontee (2007) reported a breach analysis of the Aldeburgh to Orford Ness frontage under a NAI scenario. They considered three levels of breach with increasing likelihood of permanence: a level 1 breach with a basal level around MHWS tides, a level 2 breach with a basal level around MLW, and a level 3 breach with a basal level at or below MLWS which is subject to tidal flows on a daily basis. Level 1 and level 2 breaches may be temporary, being subject either to infilling and closure, or they may evolve through deepening and widening into a more permanent level 3 tidal inlet. Two locations were identified where development of a permanent breach was considered possible under a NAI scenario, at Slaughden and at the northern end of Lantern Marshes. Slaughden was considered the most likely location for a breach in SMP2 Epoch 3 (50 - 100 years into the future) due to failure of the defences, if not maintained. Northern Lantern Marshes was considered to be a less likely location primarily because at the time of the analysis in 2005 the beach and barrier in this area had a healthy sediment volume and had experienced an accretionary trend in the preceding decade. This situation has changed since 2005.

4.2 Barrier inertia parameter

Building on initial work by Powell (1990) and Bradbury and Powell (1992), Bradbury (1998, 2000) carried out a barrier stability investigation with specific reference to Hurst Castle Spit in Hampshire. Physical model tests were undertaken for a range of conditions and compared with field data. A barrier inertia parameter, (B_i), was developed (Bradbury, 1998):

(Eqn. 1) $$B_i = R_c B_a / H_s^3$$

where R_c = crest freeboard (m), level of crest relative to a defined still water level (SWL), where SWL = astronomical tide plus surge, but excluding waves
B_a = supra-tidal barrier cross sectional area (m^2) above a defined SWL
H_s = significant wave height (highest one-third of wave heights, m, close to the beach)

Overwashing is predicted when the critical barrier inertia threshold is exceeded, i.e. if

(Eqn. 2) $$R_c B_a / H_s^3 < 0.0005 (H_s/L_m)^{-2.545}$$

where L_m = wave length of mean wave period, T_m, is obtained from the equation (Bradbury *et al.*, 2005):

(Eqn. 3) $$L_m = (g.T_m^2)/2\pi$$

where g = acceleration due to gravity (9.81 m s^{-2})

The upper confidence bound critical value for overwashing is given by (Bradbury *et al.*, 2005):

(Eqn.4) $$R_c B_a / H_s^3 < 0.0006 (H_s/L_m)^{-2.5375}$$

As noted above, the model was developed for the shingle barrier at Hurst Castle Spit and Bradbury *et al.* (2005) reported that application of the model outside the valid range of H_s/L_m would result in under prediction of overwashing. Later physical modelling results (Obhrai *et al.*, 2008) confirmed that extrapolation of the original empirical model is not valid and a modified predictive curve was proposed which is valid for the range $0.01 < H_s/L_m < 0.06$:

(Eqn. 5) $$R_c B_a / H_s^3 < -153.1(H_s/L_m) + 10.9$$

Hartley & Pontee (2008) calculated values of the Bradbury barrier inertia parameter for each of the EA strategic beach monitoring profiles using data for 1991 -2005, taking an average profile for the sections with no hard defences, and hindcast wave data for the period 1993-2005 previously used by Halcrow (2005). They also used the equation of Bradbury and Powell (1992) to estimate the theoretical geometry which would allow the barrier to evolve naturally through rollback once overwashed:

(Eqn. 6) $$C_F = C_H / (H_{sb}^2 L_m)^{1/3}$$

where C_F is the critical freeboard parameter
C_H is the freeboard (m)
H_{sb} is the shallow water breaking wave height (m)
L_m is the shallow water wave length (m) with mean period T_m

and if C_F >0.7 no crest lowering will occur
if C_F = 0.1 to 0.7 the barrier may respond by crest lowering or crest accretion, depending on the crest geometry
if C_F <0.1 and the crest width is <20 m inundation of the barrier will occur, lowering the crest below the storm peak SWL.

Based on an analysis of LiDAR data, Hartley & Pontee (2008) concluded that the greatest threat to the estuary would arise from the formation of a breach near the northern end of Lantern Marshes where the River Alde comes closest to the sea and the land level behind the barrier is low. However, the Bradbury analysis indicated that in this area there is an annual probability of the crest being overwashed of 0.33% in year 0 and 0.33 to 5% in year 100, whereas at Slaughden and south of the Martello tower the risk of defence failure increased from 0.33% in Years 0 - 20 to up to 100% by year 50.

However, as noted above, the barrier width and sediment volume at the northern end of Sudbourne Beach have decreased significantly since 2009 due to erosion, and the risk analysis undertaken by Hartley & Pontee (2008) therefore needs updating.

As part of the present assessment, values for the barrier inertia parameter B_i and critical values for over-washing have been calculated using the formulae proposed by Bradbury (2000) and Obhrai *et al*. (2008) for each of the 136 barrier cells between Fort Green and Orford Ness shown in Figure 2, although the calculated values have limited meaning for the defended frontage north of cell 40 and the very wide, multiple-ridge section of the barrier south of profile 50. The calculations were made for inshore wave conditions of H_s = 3.0 m and 5.0 m, mean wave periods of 3.5, 5.5, 7.5 and 9.5 s, and still water levels of 2.5m, 3.0 m and 3.5 m ODN. These combinations of conditions were selected following an examination of wave data for the AWAC located off Sudbourne Beach (sometimes referred to as

Slaughden South) between October 2006 and October 2009 (Table 4 and figures in Appendix 4).

Using the Bradbury (2000) formula and validity criterion, the results for the H_s = 3.0 m and SWL = 3.0 m scenario showed that the undefended single ridge between cell 40 and cell 50 would be likely to experience overwashing only with 9.5 s period waves (Table 5). In the case of H_s = 5.0 m, application of Eqn. 3 (section 4.2) indicates that the formulae is outside the valid range of H_s/L_m of 0.015 to 0.032 suggested by Bradbury *et al.* (2005) and no threshold can therefore be specified (Figure 9; Table 5). Using the modified formula and validity range H_s/L_m of 01 to 0.06 suggested by Obhrai *et al*. (2008), overwashing on the undefended single ridge section (cells 40-50) is predicted for wave conditions with H_s =3.0 m and H_s = 5.0 m accompanied by mean wave periods of 9.5 s and 7.5 s, but not for a wave period of 5.5 s (Figure 10; Table 5). It should be noted that the thresholds for overwashing predicted by the Bradbury and Obhrai *et al*. formulae are highly sensitive to wave length / period, and this parameter can change (reduce) rapidly as waves approach the shoreline, slow down and become more closely spaced due to bed friction. Estimates of the wave run-up and washover risk based on measured or modelled wave period in the nearshore zone (e.g. 200 m from the shore) are, in most cases, likely to over-estimate the risk.

Where wave run-up does not reach the barrier crest, seaward movement of sediment and upper beach scarping are likely to result. Over the longer term, if the eroded sediment is moved alongshore this may result in net reduction in the sediment volume of the barrier. Figure 11 presents a schematic model showing the likely future evolution of the artificially maintained barrier at profiles KP15 and SO44, under a policy of NAI (profile locations are shown on Figure 12). If the beach sediment budget in this area remains negative, storms will cause recession of the seaward barrier face to a point where it removes the ridge crest, intersects the backslope, and overwashing becomes possible. Subsequent storms will transport much of the remaining barrier sediment landwards, covering the marsh and infilling channels in the back-barrier area. Waves passing over the flattened barrier will impact on the river wall, resulting in its progressive erosion; at that point overwashed shingle will be lost into the river. In this location equilibrium barrier rollover is probably impossible, and in the medium term there would be a high risk of a significant breach.

Figures 8 and 13 illustrate how the sediment volume of the narrow section of barrier south opposite the northern end of Lantern Marshes has decreased markedly since the beginning of 2010, and notably since March 2013, following a period of significant sediment accretion in the years up to late 2009. It is presently uncertain to what extent this trend may again reverse in the short term. As noted earlier in this report, the possibility that the likelihood and magnitude of future alongshore sediment transport reversals may have been reduced by the northward growth of the Aldeburgh Ridge, and by other changes in nearshore bathymetry, requires further investigation. However, whether or not short term beach recovery takes place, this section of the barrier is likely to continue to be a significant weak point in the medium and longer term.

Analysis of data recorded at a series of EA AWAC sensors deployed at a number of stations along the Suffolk coast between October 2006 and October 2009, and comparison with a longer term record (2006 to present) for the Southwold Approach buoy, suggests that the change from accretion to erosion at Lantern Marshes north after 2010 was due to a change in net alongshore transport direction and wave power. The Suffolk coast experiences a bi-directional wave regime, with waves approaching the shore from directions centred on ENE and SE. The relative magnitude of wave from the two directions, and total wave energy varies from north to south down the coast (Figures A4.1 & A4.2 in Appendix 4). There is also a reduction in the frequency of higher energy waves (H_s >1.5 m) southwards from Covehithe towards Slaughden South (Sudbourne), and an even sharper reduction south of Orfordness towards Felixstowe. At Slaughden South waves >1.5m were recorded mainly from the ENE between 2006 and 2009, but waves with H_s <1.5 m occurred for a higher proportion of the time and the resultant (average) potential alongshore transport direction, based on wave power, was almost perpendicular to the shore during this period (Figure A4.1). The relationship between wave height and wave power is exponential (Figure A4.3), but potential alongshore transport flux is dependent on wave frequency (duration) as well as wave height. The wave regime off Suffolk shows a high degree of temporal variability, with alternating groups of months or years characterised by dominance of waves from the ENE or SE. Total wave power, and hence sediment transport potential, also shows high variability from month to month, season to season, year to year and sometimes decade to decade. Consequently, net potential alongshore sediment transport shows periodic reversals on sub-annual to decadal timescales, with the instantaneous drift direction at any particular point on the coast determined by nearshore wave approach direction and the local shoreline orientation. Figure A4.4 shows the variation in mean monthly wave power between October 2006 and October 2009. Figure A4.5 shows the calculated net northerly or net southerly alongshore transport calculated for all waves along the sections of shoreline closest to each AWAC.

The Slaughden South (Sudbourne) AWAC recorded marked net northerly potential transport during the winter of 2006 – 07, and to a lesser extent during the winter of 2007-08, but for the remainder of this period up to October 2009 the net transport directions alternated and remained approximately in balance (Figure A4.5). Larger waves (H_s >1.5 m) from the NE and ENE were primarily responsible net southerly transport in the period October 2007 – October 2009 (Figure A4.6). Smaller (H_s = 1.0-1.5 m) but sustained waves from the SE were mainly responsible for the northerly transport during the winter of 2006-07 and part of the winter 2007-08 (Figure A4.6).

Comparison of the AWAC wave records with a longer term (October 2006 to present) record for the Southwold Approach waverider buoy shows good agreement for the period October 2006 – October 2009, and it is therefore reasonable to assume that changes in wave characteristics recorded since October 2009 at the Southwold buoy are also broadly representative of conditions at Slaughden South (although the total wave power and alongshore transport potential are lower at the Slaughden South inshore location than at the Southwold buoy, which is located further offshore and is more exposed). Calculated values of the alongshore component of wave power for the Southwold buoy show that the period of slight net northerly potential transport between 2006 and 2009 was followed by a period of strong net southerly potential transport which lasted almost until the end of 2013 (Figure A4.7). A short period with very strong net southerly potential transport, associated with large waves from the ENE, occurred during March 2013 and resulted in a sharp fall in beach levels / sediment volumes and barrier face erosion at the northern end of Sudbourne Beach. Slight net southerly drift conditions continued throughout 2013, and beach levels were low at the time of the surge tide on 5-6 December 2013. This event, which gave rise to a resultant SWL of approximately 3.05 -3.10 m ODN at Slaughden, was not accompanied by very high waves and a complete breach of the barrier did not occur. During late December 2013 and 2014 a period of southeasterly waves gave rise to strong northerly potential sediment transport and beach levels at the northern end of Sudbourne Beach recovered. Although there have been temporary reversals, net northerly transport conditions have dominated since that time and there has been some further beach recovery along the undefended Sudbourne Beach frontage. Sediment has not, however, reached the groyne bays to the north of the Martello tower where beach levels have remained low.

5.0 Inlet formation and stability assessment

If a shallow channel is formed during or shortly after a breach, it may subsequently infill and the barrier may regain its lateral continuity, or the channel may evolve into a more permanent inlet. Hartley & Pontee (2008) distinguished three types of breach of increasing relative permanence:

- Level 1 breach with a base around the level of MHWS tides (1.4-1.5 ODN in the northern Lantern Marshes area), which may either (i) infill through a combination of alongshore drift and the formation of accretionary ridges around successive high water marks on the foreshore, or (ii) deepen to form a Level 2 breach

- Level 2 breach with a base around the level of MLW (0–0.2 m ODN) which may initially be more permanent than a Level 1 breach but may still either infill or evolve into a permanent inlet

- a permanent inlet (considered here as Level 3) with a base at or below the level of MLWS tides (below -0.7 m ODN) which experiences tidal flows on a daily basis.

Key questions are:

- what factors control the initial level to which a breach is cut (i.e. whether a level 1, 2 or 3 breach is initially formed)?

- what factors control the stability of the new inlet and determine if it evolves from Level 1 to Level 2, or from Level 2 to Level 3, or vice versa?

- in the event of inlet healing would the reconstructed ridge have the same dimensions and sediment volume as the pre-existing ridge, or would there be a permanent reduction in barrier inertia with the former inlet acting as a continuing location of potential weakness (and hence create a greater risk of future overwashing and/ or breaching)?

5.1 Tidal channel formation

Overwashing and flattening of a shingle barrier during a storm surge frequently does not result in the formation of a permanent (Level 3) tidal channel at the breach location(s). Observations following breaching events on the Walberswick – Dunwich barrier (Pye & Blott, 2009a) and elsewhere have shown that a flattened barrier may maintain a constant level along much of its length, broken by one or more shallow channels which typically are cut down no deeper than mid tide level (MTL). In the case of breaches are formed in unconsolidated sands and gravels, scouring can be very rapid, especially where the tidal range and volume of escaping tidal water are large. In cases where barriers overlie compacted mudflat or saltmarsh sediments there is greater resistance to scour and deepening is usually much slower. Rapid formation of a deep, wide channel is most likely to occur if the newly formed scour channel 'captures' a pre-existing river or estuarine channel, thereby diverting a large volume of flow through the new breach. Although there have been many barrier flattening and overwashing events on the UK coast in the past 250 years, only a relatively small number have resulted in the creation of permanent inlets , and in very few cases has the main tidal channel of an estuary been diverted.

In the case of Sudbourne Beach, the narrowest part of the maintained barrier coincides with the area at the northern end of Lantern Marshes where the distance between the backslope of the shingle embankment and the eastern river wall is at a minimum (Figure 12). At EA monitoring profile S044 (Figure 11) the landward toe of the embankment lies only about 40 m from the river wall, and the intervening area of recently reactivated saltmarsh is heavily dissected and has a maximum surface level of approximately c. 1.3 m ODN, approximately equivalent to MHWS at Slaughden Quay, Table 1). This compares with an estimated 1 in 200 year water level on the open coast at Slaughden of 3.25 m ODN (Table 3).

Owing to the time difference between high water on the open coast and in the estuary, there can be difference in still water levels across the Slaughden – Sudbourne barrier of up to 30 cm during a surge tide. If the shingle bank failed in this location, having been narrowed by wave erosion to a point where bursting could occur due to the combined effect of hydrostatic pressure differences, wave impact and overwashing, water would flow towards the river, transporting shingle which would initially pile up against the Lantern Marshes river wall (Figure 11). Had the ridge been breached completely in March or December 2013, only a relatively small volume of shingle from the barrier remnant would have been available to be spread across the marsh surface. The washover lobes would have probably have had a maximum elevation of approximately 2 m ODN on the seaward side, sloping towards the river wall. The condition of the wall is presently such that it would be reasonably effective as a shingle retaining structure, but it can be expected to become less effective over time under a policy of NAI (no maintenance). The maximum SWL attained during the 5-6 December 2013 storm surge along the Slaughden – Sudbourne is estimated to have been 3.1–3.2 m ODN. However, it is unlikely that the flow of flood tidal water and movement of waves across the flattened barrier would have been sufficiently large or strong to cause significant incision below the base of the barrier, which in this location rests on a foundation of reclaimed saltmarsh mud and the remains of earth embankments which are relatively erosion resistant. These deposits, which have been compacted beneath the weight of the barrier, outcrop on the seaward side where they form a ledge around MHWN level (photograph A5.9 in Appendix 5). Had barrier failure occurred in March or December 2013, the maximum depth of water above the flattened barrier crest is likely to have been of the order of 1.1 - 1.2 m. On the ebb tide a reverse water slope would have developed between the estuary and the sea, but the return flow would have been restricted to the top of the tide. As soon as the water level fell below the crest of the flattened barrier (c. 2 m ODN) all of the ebb the flow would have been diverted down the main estuary. Bed scour would therefore not have been of sufficient magnitude or duration to create a Level 3 breach, and it is unlikely that a level 2 breach would have formed. In order to form a Level 3 breach in the northern Lantern Marshes area the entire barrier structure, including marsh foundation, would need to move landwards by at least 40 m. It is very unlikely that erosion of cohesive sediment on this scale could be accomplished during a single high magnitude storm event, but the process could be achieved through sustained erosion over a period of 15 to 20 years.

Hartley & Pontee (2008) estimated the size of a potential breach at Slaughden (which they considered to be the location at highest risk of a breach) using the empirical relationship proposed by Jarrett (1976), modified from O'Brien (1969):

(Eqn. 7) $$A_E = cP_n$$

where A_E is the equilibrium cross-sectional area (m^2)
P is the spring tidal prism (m^3)
c is an empirical scale and shape coefficient
$_n$ is an empirical exponent (c and n are determined from regression analysis of data for relevant field sites)

They examined this relationship for the existing (2005) mouth of the Alde-Ore estuary, assuming an estuary spring tidal prism of 19.6 x 10^6 m^3 suggested by Black & Veatch (2006), a simplified rectangular cross-section at the mouth with width of 235 m, depth below mean sea level of 2.06 m, and area of 484.1 m^2. Based on the assumptions that the cross-sectional area of the present estuary entrance is in equilibrium with the spring tidal prism, that half of the tidal prism would pass through a new breach at Slaughden, and that existing alongshore transport rates would remain unchanged, they estimated the likely width of a new breach to be 122.31 m if scouring occurred to a depth of 2.06 m below MTL, and 185.27 m if scouring only occurred to MLW level. They noted that the timescale and the breach scenario would dependent on the frequency and severity of storm events and alongshore transport rates, but did not consider the mechanisms by which a Level 1 or Level 2 breach might evolve to Level 3.

The approach adopted by Hartley & Pontee (2008) made a number of questionable assumptions. Analysis of LiDAR DEMs (Pye & Blott, 2014a,b, 2015a,b and this study) has shown that the cross-sectional area of the Alde – Ore estuarine channel is highly variable in space and time, bringing the concept of an equilibrium relationship between estuary mouth cross-sectional area and tidal prism into question. Figure 13 shows the cross-sectional profile of the estuary at North Weir Point based on LiDAR and bathymetric surveys in 2012-13. The cross-sectional area of the channel below mean sea level at that time was 720 m^2, compared with the value of 487 m^2 used by Hartley & Pontee (2008). At the time of the 2012-13 surveys the cross-sectional area of the channel showed major variation up the estuary, being considerably larger at locations near the northern end of Lantern Marshes than at the mouth (Table 6). Given that the MHWS tidal prism of the estuary above Lantern Marshes is less than half that of the estuary as a whole (Table 7), this suggests that the mouth of the Ore is constrained (i.e. too narrow relative to the equilibrium tidal prism suggested by the Jarrett (1976) and similar equations (e.g. Le Conte, 1907; O'Brien, 1931, 1969; Hume & Herdendorf, 1993; Gao & Collins, 1994a,b; Hughes, 2002; Townend, 2005). This may be explained by the tendency for coastal processes to push the southern end of Orford Spit towards the coast, and to form significant flood and ebb-tidal delta shoals just inside and outside the estuary mouth. The response of the estuary to this pressure is to create a relatively deep channel at the entrance (reaching -7 to -8 m ODN in 2012-13) which is characterised by high current speeds and a gravel-dominated bed. The *A/P* ratio determined for the mouth therefore does not provide a good basis on which to calculate the equilibrium morphology of shallower breaches / inlets elsewhere in the estuary which are not subject to the same confining pressures. A more appropriate basis for the equilibrium breach estimation might be provided by using relationships based on the morphology / tidal prism relationships of smaller, unconstrained inlets and saltmarsh creeks, or those associated with storm-induced breaches in earth embankments around formerly reclaimed saltmarshes in the area (cf. Byrne *et al.*, 1980; Goodwin, 1996). However, experience has shown that, under condition of NAI, breaches in embankments around reclaimed marshes do not keep the same dimensions for long, typically tending to widen and coalesce while at the same time the tidal prism of the re-flooded marsh area decreases with time due to sedimentation (Pye & Blott, 2009b).

5.2 Inlet closure due to alongshore sediment transport

The likelihood of a breach being infilled with sediment (healed) is dependent on the balance between opposing forces, namely the current speeds and bed shear stresses associated with tidal flows which flush sediment out of the breach on flood and/or ebb tides (related to tidal prism), and the rate at which sediment is supplied to the mouth of the inlet by alongshore and/ or onshore sediment transport. Hartley & Pontee (2008) used the potential breach stability ratio developed by Bruun & Gerritsen (1960) to assess the likely stability of a breach at Slaughden. This ratio is given by:

(Eqn. 8) $$P/M$$

where P is the tidal prism (m^3)
M is the annual sediment drift rate ($m^3\ a^{-1}$)

If the P/M ratio is >150 sediment flushing is very strong and only a small ebb tidal delta is likely to develop at the inlet entrance, while at the other extreme where P/M <20 flushing is weak, sediment is likely to accumulate within the entrance, and the inlet becomes blocked (Bruun, 1978). A critical value of P/M = 100 is frequently taken to indicate whether an inlet is stable or unstable (i.e. will remain open or will close). Using average sediment transport rates reported by Halcrow (2005), Hartley & Pontee (2008) calculated a P/M ratio of 84 for the entrance to the Alde Ore estuary, indicating a transitional stability condition with a large ebb tidal delta exists but a channel is maintained (Bruun, 1978). They also calculated a P/M ratio of 39 for a breach at Slaughden assuming half the tidal prism of the estuary passes through it; this value indicates poor stability with the entrance partly or periodically blocked by an ebb tidal delta but periodically flushed during storms. However, the P/M ratio values obtained for the estuary entrance at North Weir Point and a potential breach at Slaughden are clearly dependent on the selected values for P and M.

Only limited modelling of alongshore sediment transport in the Aldeburgh area has been undertaken, all of it using outdated bathymetric data and hindcast modelled wave data. Vincent (1979) used daily vector- averaged wind data for a single site (where) over the period 1964-76 and empirical equations to calculate offshore wave heights and wave refraction over the nearshore topography to estimate the angle of incidence of waves on the beach and the ratio of the incident wave's energy per unit crest length to the offshore wave energy per unit crest length. Potential alongshore transport around the East Anglian coast was calculated for a single wave period (6 s) using the CERC (1977) formula. Results were average over 5 km or more of coast. Net potential transport was estimated to be 80 x $10^3\ m^3\ a^{-1}$ in a southerly direction at Aldeburgh, 195 x $10^3\ m^3\ a^{-1}$ in a westerly (onshore direction) at Orfordness, 198 x $10^3\ m^3\ a^{-1}$ in a south-southwesterly direction at Shingle Street. Onyett & Simmonds (1983) adopted a similar approach using wind data for the period 1961-80 and obtained similar results. However, the CERC formula used by Vincent and Onyett & Symmonds is applicable only for sand, whereas the upper beaches between Aldeburgh and Shingle Street are composed largely of gravel, the transport rates for which are likely to be far lower than for sand (Sutherland *et al*., 2002).

Halcrow (2005) used Met Office hindcast wave data for the period 1990 - 2003, based on winds recorded 52 km offshore from Aldeburgh, transformed to eight inshore locations using an in-house wave-refraction model, and the Kamphuis (1991, 2000) equations to compute sediment transport along the Thorpeness to Hollesley frontage. The results indicated average net southerly drift in excess of average net northerly drift along the shore between Aldeburgh and Sudbourne Beach, net northerly transport in excess of southerly transport at Lantern Marshes, and net southerly transport in excess of northerly transport along most of Orford Beach. Net alongshore sediment transport convergence was indicated opposite the southern end of Lantern Marshes. Between 1990 and 1996 net drift was indicated to be southerly in all years except 1994, while between 1996 and 2003 net drift was northerly in all years except 2001and 2003. For the Sudbourne Beach coastal study unit the maximum northerly drift (350,000 m^3) was indicated to occur in the year 2000, while the maximum southerly drift (approximately 370,000 m^3) was indicated to occur in 1996. Maximum net drift (approximately 220,000 m^3) was also indicated in 1996 in a southerly direction. Data for the adjoining coastal study unit (Martello to Sudbourne Beach) showed a similar temporal pattern but the magnitude of indicated transport was only about two-thirds that on Sudbourne Beach (Halcrow 2005). The Kamphuis equations are not well validated for shingle transport (van Rijn, 2014) and the Halcrow modelling results can therefore only be considered to indicate the likely direction and broad magnitudes of transport. Net transport direction and transport rate for coarse gravel on the upper beach can differ to those for sand and fine gravel on the lower beach since transport of medium to coarse gravel is only accomplished by higher energy waves which may have a different directional distribution to that of smaller, more frequent waves (Pye, 2001). Beach drifting on the upper beach close to the time of high water is the principal mechanism of alongshore shingle transport, whereas sand transport is accomplished mainly by wave and tide-generated currents, predominantly on the lower beach and in the shallow sub-tidal zone, over a greater part of the tidal cycle.

HR Wallingford (2016), as part of a Crown Estate sponsored feasibility study for a shingle engine (large scale beach nourishment) at Slaughden, modelled baseline alongshore sediment transport using Met Office hindcast offshore wave data for the period 1980-2014, transformed inshore for 10 nearshore locations between Thorpeness and Orfordness using the SWAN 2D model, Seazone bathymetry, the BEACHPLAN 1D model and the DRCALC model (based on the CERC alongshore sediment transport formula). Results indicated high gross rates of alongshore transport but low net rates of transport owing to the bimodal nature of the wave regime in the area. At Point 105, opposite Aldeburgh Town, the average potential net drift rate was found to be close to zero, while along the Slaughden and Sudbourne beach frontage (Points 106 to 108) net southerly drift was indicated to exceed net northerly drift. Opposite the southern part of Lantern Marshes (Points 109 and 110) net northerly transport was found to exceed net southerly transport, indicating sediment transport convergence at the southern end of Sudbourne Beach. Although this study used outdated bathymetry, the broad pattern of sediment transport indicated by the modelling is consistent with other evidence.

The Halcrow (2005) and HR Wallingford (2016) modelling studies both indicate periodic drift reversals due to intra- and inter-annual variability in wave conditions, maximum transport rates which are up to four times the average rate, and net southerly potential transport along the Slaughden – Sudbourne Beach frontage. However, the magnitude of the actual net transport is subject to considerable uncertainty for several reasons, including:

(1) the modelling has used hindcast wave data for offshore locations, transformed inshore, rather than measured inshore wave data, to drive the sediment transport models; validation of the modelled wave direction and height is very limited

(2) none of the modelling studies have used bathymetry which reflect the present situation; parts of the composite Seazone Trudepth data sets used in the HR Wallingford and Halcrow studies are based substantially on surveys in the 1980s and do not capture more recent and significant changes in the extent, height and position of the Aldeburgh Ridge (see Appendix 2)

(3) the models have used sediment transport equations which have not been fully validated for gravel transport, and simplistic assumptions have been made about the size of sediment being transported - e.g. use only of median (D_{50}) and sometimes D_{90} exceedance particle diameter; while the upper beaches are composed largely of medium to coarse gravel, the lower beaches typically consist of spatially and temporally variable mixtures of, on average, approximately one third gravel and two thirds sand (Pontee, 1995; Pontee *et al*., 1995)

(4) most annual drift calculations have been presented for calendar years (January – December), rather than individual months or summer (April – September) and winter (October – March) periods; this tends to 'split' into two different years periods of high or low drift rates which mainly occur over the winter periods and which are important determinants of beach condition at any particular time; this problem is avoided where monthly values are reported

(5) actual alongshore transport along the Slaughden – Aldeburgh frontage is likely to be much lower than potential transport because sediment is trapped by the groyne system; actual north to south transport along Sudbourne Beach is likely to be reduced to a greater extent (relative to the potential transport) than south to north transport since no groynes are present to the south

(6) field observations and the available monitoring data suggest that onshore-offshore transport under is important in emptying the groyne bays at Slaughden and causing draw-down of Sudbourne Beach, but the magnitudes of these exchanges, and there relationship to subsequent alongshore transport, are not well quantified; onshore transport is favoured by periods of moderate waves with low steepness (small *H/L* ratio), while offshore transport is

favoured by large waves with high steepness (large *H/L* ratio), especially those which approach the shore almost perpendicularly, and which are reflected from the hard defences and the steep face of the maintained shingle barrier (which prevent overwashing and landward transport of sediment).

Several authors have proposed relationships between maximum current velocity (V_{max}) and cross-sectional flow area (A_c) to define inlet stability / instability (e.g. Brown, 1928; Escoffier, 1940; Kraus, 1988). The widely used Escoffier diagram indicates that there is an equilibrium flow velocity and cross-sectional area required to maintain hydraulic stability of an inlet. However, the defining curve is based on empirical relationships obtained mainly for sandy inlets in the United States, and is of uncertain applicability to gravel and mixed sand-gravel dominated inlets. Experimental studies have shown that the threshold friction velocity and bed shear stress required for a unidirectional current to mobilize sediment particles of quartz density (2.65 g cm^3) in water at 20^{o}C increases more steeply as a function of particle size for particles larger than about 1 cm than for smaller particles (Inman, 1949; Neill, 1967; Miller *et al*., 1977). The threshold friction velocity for a 1 cm (fine gravel) particle is approximately 2 cm s^{-1} and that for a 10 cm particle is approximately 4 cm s^{-1}, although the exact threshold value will depend on particle shape, particle density, sediment sorting, particle packing arrangement, bed slope and bed type (solid or granular).While flood or ebb tidal flow within an estuary channel such as the Alde may be considered to be approximately steady and unidirectional, conditions on the open coast are more complex, involving a combination of oscillatory flow due to wave action and variable steady / unsteady flow due to wave-induced and other currents. The critical shear stress required to initiate particle movement is therefore likely to be exceeded more frequently in an inlet entrance on the open coast (e.g. at Sudbourne Beach) than within a sheltered estuarine channel environment. It is likely that there would be a strong tendency for waves to move sediment into a channel in this area and complete flushing would be unlikely to occur as long as the entrance at North Weir Point remained open. Johnson (1973) compared deep water wave power with tidal prism for inlets in the United States and concluded that, as wave power increases, the tidal prism must also increase and exceed a critical value for the inlet to be flushed free of sediment. For sandy inlets, a mean cross-sectional tidal current speed velocity of 1.0 m s^{-1} is usually sufficient to flush away incoming sediment (Bruun, 1978), but in the case of gravel-dominated systems a considerably higher mean velocity may be required (2 – 4 m s^{-1}, depending of sediment size).

Consideration of the morphology of the Slaughden – Sudbourne shingle barrier and adjoining River Alde suggest that any channel formed following a breach would initially be fairly narrow and shallow. Smaller inlets (<150m wide) are more prone to closure than larger inlets (>150 m wide) because they tend to have a smaller depth and are more easily blocked by alongshore or onshore drifted sediment than larger inlets which require a large influx of sediment, or a major reduction in tidal prism (e.g. through land reclamation), to cause instability and closure (Mehta, 1996). In some estuaries the influx of significant quantities of river water helps to flush sediment from the entrance channel (Gao & Collins, 1994). This may be a semi-continuous process or may occur only every few years or decades when high magnitude flood events occur (Cooper, 1993). However, in the case of the Alde-Ore, the maximum river flow is small relative to the tidal prism (Pye & Blott, 2015b) and is unlikely to play any significant role in flushing sediment from a new breach.

Especially in areas with a small tidal range (and hence relatively smaller tidal prism), formation of a bar due to alongshore and/or onshore sediment transport can close off an inlet relatively quickly, although further shallow breaches may form at the same locations during subsequent storm surge events. Experience of breaching of many gravel and mixed sand – gravel barrier on the East Coast of England has shown that natural formation of new permanent tidal inlets is a rare occurrence, and historically establishment of a new permanent channel has required deliberate excavation and installation of mouth groynes to intercept alongshore sediment transport (e.g. at the present mouth of the Blyth estuary).

Natural formation a channel of sufficient depth and cross-sectional area to be stable / self-sustaining at Sudbourne or Slaughden is only likely to be possible if the entire barrier moves landwards by 40–50 m due to erosion and the deep water channel of the Alde is captured, or alternatively the Alde Channel moves seawards by a similar distance. If a stable Level 3 inlet was formed, the existing channel at North Weir Point would be likely to experience a reduction in width and depth due to reduced flow velocities and sediment accumulation, but it would be unlikely to close entirely due to natural processes. Two channels of moderate size could co-exist, effectively making Orford Ness an island, similar to Scolt Head Island in North Norfolk, but with a deeper 'harbour' behind the Orford Island barrier. Such a change in morphology and hydrodynamic regime could have significant implications for surge tide levels and flood risk around the estuary. South of the new breach, tidal waters would enter from two directions and it is likely there would be a significant reduction in average flow velocities, leading to sedimentation. A slight increase in maximum high water levels around Orford might also occur. North of the new breach the estuary would be likely to experience a more steeply rising flood tide and a more rapidly falling ebb tide, resulting in higher flow velocities which could cause localised bed scour and erosion of saltmarsh edges in front of the river wall defences. Maximum water levels would be likely to increase, with potential increased flood risk between Slaughden and Snape. If the breach was >150 m wide, significant wave penetration into the adjoining parts of the estuary could occur, with implications for overtopping of the river walls in the area. However, the magnitudes and spatial distribution of these effects need to be more accurately quantified through additional hydrodynamic and sediment transport modelling.

6.0 Conclusions and recommendations

Prior to the late 1940s the shingle barrier beach system south of Fort Green was able to respond relatively naturally to coastal forcing. This response consisted of net long-term landward movement through a process of barrier over-washing, although the rate of barrier recession varied in space and time, reflecting the incidence of storm events and aperiodic movement of 'slugs' of sediment along the shore in response to inter-annual variations in wave conditions. Long-term net sediment transport was in a southerly direction, but periodic sediment transport reversals were significant in slowing the rate of beach retreat at Slaughden and the northern end of Sudbourne Beach. Since the late 1940s the natural response of the northern end of the barrier system has been prevented as a result of the construction of groynes, sea walls and rock armour revetments. The form of the barrier has also been changed by bulldozing of the shingle to form an artificial embankment, supplemented by nourishment of the beach and parts of the embankment. This 'hold the line' policy has been largely successful in preventing significant overtopping of the barrier during storm surge events, and has reduced (although not eliminated) the tidal flood risk within the estuary.

One effect of the groynes and other defences has been to significantly reduce the natural movement of shingle along the beach, athough sediment exchange within the groyne bays now takes place mainly in an offshore –onshore direction is still possible. Since the average position of the shoreline at Slaughden and the northern end of Sudbourne Beach now lies seaward of its equilibrium position, it is increasingly difficult to maintain adequate sediment volumes within the groyne bays fronting the Aldeburgh Yacht Club and Martello tower, and much of the replenishment material is rapidly moved offshore, beyond the groynes, where it can be moved alongshore or offshore into deeper water.

The unprotected section of Sudbourne Beach which lies just beyond the southern limit of rock armour is still able to respond partly naturally to fluctuations in wave direction and alongshore sediment transport. For much of the period 1997 to 2009 this area experienced sediment accretion, fed largely by sediment moved alongshore from the south and partly from offshore as wave conditions favoured the general onshore movement of sediment. However, from early 2010 onwards a general increase in wave energy, and a relative increase in the influence of waves from the ENE, has favoured net sediment removal from this area and transfer towards an area of long-term net accretion opposite the southern end of Lantern Marshes. The result has been beach erosion and cut-back of the barrier crest at the northern end of Sudbourne Beach. Despite the dominance of ENE waves, little or no sediment has been supplied to this section of beach by alongshore transport from the north due to the presence of the groyne system around the Martello tower. Strong ENE wave condition in March 2013 caused major cut back of the barrier face at the northern end of Sudbourne Beach and came close to breaching the embankment. Erosion continued throughout 2013 and it was fortunate that the surge tide of 5-6 December 2013 was not accompanied by very high waves. Since that time there has been some natural beach recovery and the barrier has been repaired, with some minor landward realignment. However, further periods of ENE waves and beach erosion can be expected in the future, and the risk of barrier breaching will return.

Management options to deal with this problem include:

(1) Do Nothing, and allow the barrier to move landward and re-establish a natural profile

(2) Continue to hold the line through beach and barrier nourishment and reprofiling
(3) Hold the line through placement of additional rock armour or other engineering works
(4) Managed realignment, including artificial re-profiling of the barrier ridge on a different alignment, and potentially involving placement of shingle at the back of the ridge (river side)
(5) a combination of (2), (3) and (4)

The modelling and data analysis discussed in this report indicate that overwashing of an artificial shingle barrier ridge with a maintained crest level of 5.0 to 6.0 m ODN is unlikely, but that cut back of the barrier face is very likely during sustained stormy periods (especially those with waves from the ENE). If cut-back proceeds to the point where the beach scarp intersects the back slope of the ridge overwashing will become possible and the ridge may experience catastrophic failure. Since the quantity of shingle remaining in the residual part of the ridge will be small, only a thin, low amplitude washover fan will be able to form on the northern part of Lantern marshes. Under exceptionally high surge tide conditions (still water levels above 3.0 m ODN), waves will not fully break and will pass over the flattened barrier and will impact on the river wall along the western side of the northern Lantern marshes. This will initially impede transport of shingle into the river, but under a policy of No Active Intervention (NAI) the wall will rapidly degrade, allowing shingle transport and loss into the river during future storms. At that point waves will be able to pass over the barrier into the estuary and impact on the river wall along Sudbourne Marshes. This stage could be reached within 10 years of the introduction of a policy of NAI, depending on the incidence of storm events.

The assessment undertaken suggests that the development of a permanent breach at the northern Lantern Marshes, leading to capture of the upper Alde, is unlikely in the short term (Epoch 1, 0- 20 years), but could occur in the medium term (Epoch 2, 20 - 50 years) if conditions which favour beach sediment accretion in this area do not return and the shoreline is allowed to erode landwards so that the width of the barrier at mean tide level is reduced by c. 30 - 40 m. If a breach is cut down to a level below that of mean sea level the tidal flow through the breach is likely to be sufficiently rapid to cause further deepening and widening. A significant proportion (25–40% of the tidal prism of the estuary could potentially be captured by the new breach, with average flow velocities sufficiently high to keep the breach open, provided alongshore transport rates do not increase significantly. The consequences of establishing a second permanent inlet would be very significant for the estuary as a whole. The existing mouth at North Weir Point would be likely to reduce in width, depth and cross-sectional area due to the reduction in tidal prism passing through it, leading to greater sedimentation in the lower estuary. The tidal regime within the estuary would be altered and impact of storm surges on water levels and current speeds would change, potentially increasing the flood risk between Orford and the head of the estuary at Snape. In the absence of an artificial high shingle ridge defence, and especially with a wide / deep breach, there would greater wave action in the middle part of the estuary, increasing the risk of wave over topping and potential embankment erosion at Sudbourne Marshes. The precise magnitude and consequences of these changes under different scenarios require detailed hydrodynamic and wave overtopping-modelling.

Given the risks for the entire estuary associated with a policy of NAI for coastal management unit ORF 15.1 in SMP2 Epoch 2 (2025 - 2055), including the near-certainty of barrier failure in the short term, it is recommended that the current preferred management policy for ORF 15.1 in SMP2 Epoch 2 is changed to HTL, and that further assessment is undertaken of options for Epoch 3 (50 - 100 years).

It is recommended that further data capture campaigns are undertaken within the estuary and in the nearshore zone between Slaughden and the northern end of Sudbourne Beach. The purpose would be both to provide the data necessary to allow calibration and validation in further numerical modelling investigations, and to facilitate parallel data-driven assessments. A campaign undertaken in late 2014 to obtain data for water levels, current speeds and suspended concentrations within the estuary failed to deliver the necessary data, and the hydrodynamic modelling reported by Pye *et al*. (2015) had to make use of data collected in the 1990s by University College London and in the early 2000s by Gardline (2003). Previous modelling of water levels, flows and sediment transport within the estuary undertaken by HR Wallingford (1999), Black & Veatch (2006) and JBA (2011, 2012) has used relatively simple 2D models of the estuary with limited validation. Moreover, the morphology of the estuary, especially of the intertidal zone, has changed significantly since these modelling studies were undertaken. Additional modelling is required to fully assess the impacts of a potential breach at Sudbourne Beach, or elsewhere, using updated bathymetry, better field calibration / validation data, and more sophisticated 2D / 3D models of the estuary. Modelling of wave penetration into the estuary, and implications for overtopping of the river walls and intertidal erosion along Sudbourne Marshes, is also required.

The programme of further field data collection should include re-deployment of an AWAC opposite the northern end of Sudbourne Beach, together with the simultaneous deployment of a supplementary system to record changes in wave characteristics (notably, height, period / wavelength and steepness) as waves approach the beach (e.g. using an Argus camera or X-Band radar). This information would allow better quantification of alongshore variation in nearshore breaking wave height. Data from the offshore reference station could also be used for a more detailed analysis of wave spectral energy distribution and its relationship to wave run-up.

When nearshore / offshore swath bathymetric data for the Slaughden - Sudbourne Beach area (planned as part of the EA 2016-2021 monitoring programme) become available, additional modelling of wave run-up and barrier response should be undertaken using a model such as XBeach-G (McCall *et al*., 2013, 2014; Masselink *et al.*, 2014).

An AWAC, Aquadopp or similar instrumentation should also be established within the estuary close to the Aldeburgh Yacht Club to provide simultaneous information about water levels and current speeds. Ideally these deployments should be for a period of at least one year. In addition, in order to provide better information about synoptic variations in water levels and depth-related currents along the length of the estuary a series of instruments should be deployed at a number of stations between the estuary mouth and the head. Data should ideally be collected for a full neap -spring tidal cycle, but data collected (in part from boats) on a number of selected tides would also be useful.

In order to maximise the utility of additional data collection and assist the planning of further modelling, it is recommended that a meeting / workshop is organized to integrate the requirements / views of all interested parties.

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Tables

Table 1. Tidal levels (in metres, relative to Ordnance Datum Newlyn, ODN) on the open coast and within the Alde-Ore estuary from: (A) predictions in Admiralty Tide Tables (UKHO, 2015); (G) Gardline (2003) who deployed Aquadopp (acoustic Doppler) meters and Aanderaa (pressure transducer) tide recorders over a period 32 days between 21st August and 23rd September 2003; *The Gardline Alde-Ore Mouth values are considered to be unrepresentative as only nine days of data were obtained. Values in **bold** have been estimated or calculated by extrapolation using the trend at the relevant Standard Port.

	HAT	MHWS	MHWN	MSL	MLWN	MLWS	LAT	CD	MSTR	MNTR	Source
Open Coast, north to south											
Lowestoft	1.4	0.9	0.6	0.16	-0.5	-1.0	-1.4	-1.50	1.9	1.1	A
Southwold	1.6	1.1	0.8	0.25	-0.4	-0.8	**-1.2**	-1.30	1.9	1.2	A
Aldeburgh	1.8	1.1	0.7	0.06	-0.7	-1.3	**-1.8**	-1.60	2.4	1.4	A
Martello Towers	**1.86**	1.56	1.04	nd	-0.25	-0.76	nd	nd	2.31	1.29	G
Orford Ness	1.4	1.2	1.1	nd	-0.8	-1.2	**-1.5**	-1.65	2.3	1.8	A
Orford Haven Bar	1.9	1.5	0.9	0.13	-0.7	-1.3	**-1.7**	-1.66	2.8	1.6	A
Bawdsey	2.0	1.6	1.0	0.09	-0.8	-1.5	**-2.0**	-1.77	3.1	1.8	A
Felixstowe	2.3	1.9	1.2	0.13	-1.0	-1.6	-2.1	-1.95	3.4	2.1	A
Harwich	2.4	2.0	1.4	0.12	-0.9	-1.6	-2.1	-2.02	3.6	2.3	A
Walton-on-the-Naze	2.5	2.0	1.2	0.08	-1.1	-1.8	-2.3	-2.16	3.8	2.3	A
Alde-Ore Estuary, mouth to head											
Orford Haven Bar	1.9	1.5	0.9	0.13	-0.7	-1.3	**-1.7**	-1.66	2.8	1.6	A
Alde-Ore Mouth	**1.78**	1.51	1.04	nd	-0.27	-0.75	nd	nd	2.25	1.31	G
Butley River Entrance	**1.57**	1.36	0.99	nd	-0.22	-0.60	nd	nd	1.97	1.21	G
Gedgrave Marshes	**1.56**	1.35	0.98	nd	-0.25	-0.63	nd	nd	1.98	1.23	G
East Havergate	**1.60**	1.38	1.00	nd	-0.25	-0.63	nd	nd	2.01	1.25	G
Orford Moorings	**1.64**	1.42	1.03	nd	-0.22	-0.61	nd	nd	2.03	1.25	G
Orford Quay	1.5	1.2	0.7	0.20	-0.5	-1.0	**-1.4**	-1.60	2.2	1.2	A
Main Channel	**1.67**	1.44	1.04	nd	-0.26	-0.66	nd	nd	2.10	1.30	G
Aldeburgh Yacht Club	**1.71**	1.48	1.08	nd	-0.23	-0.64	nd	nd	2.12	1.31	G
Slaughden Quay	1.5	1.3	1.0	0.19	-0.6	-1.0	**-1.3**	-1.60	2.3	1.6	A
Aldeburgh Marshes	**1.74**	1.50	1.08	nd	-0.27	-0.69	nd	nd	2.19	1.35	G
Iken Cliffs	1.6	1.3	0.8	0.20	-0.5	-1.0	**-1.4**	-1.60	2.3	1.3	A
Iken Cliffs	**1.72**	1.47	1.03	nd	-0.35	-0.80	nd	nd	2.27	1.38	G
Snape Maltings	**1.57**	**1.29**	**0.84**	**0.30**	**-0.46**	**-0.66**	**-0.80**	**-1.27**	**1.95**	**1.49**	K

Table 2. Estimated tidal levels in different parts of the estuary used to calculate tidal volumes in this study. Levels in the upper (A) and lower (E) estuary are averaged from Gardline (2003) data, and intermediate tidal levels have been calculated using the relationship shown in Figure 6. Levels in areas B, C and D have been calculated using 0.75, 0.50 and 0.25 proportions respectively of the differences between A and E. After Pye & Blott (2014a).

	HAT	MHWS	MHWN	MSL	MLWN	MLWS	LAT
A: Upper estuary above Slaughden	1.70	1.50	1.10	0.20	-0.30	-0.70	-1.60
B: Home Reach, Lantern Marshes	1.66	1.47	1.08	0.20	-0.29	-0.68	-1.55
C: Halfway Reach, Sudbourne, Kings North and upper Butley Marshes	1.62	1.43	1.05	0.20	-0.28	-0.65	-1.50
D: Orford, Kings South and Chillesford Marshes	1.57	1.40	1.03	0.20	-0.26	-0.63	-1.45
E: Lower estuary below Gedgrave Marshes	1.53	1.36	1.00	0.20	-0.25	-0.60	-1.40

Table 3. Return periods of extreme high waters near the mouth of the Alde-Ore. Taken from Environment Agency (2011) 'Coastal flood boundary conditions for UK mainland and islands' (McMillan *et al*., 2011).

	Return Period (years)							
	1	2	5	10	20	50	100	200
Chainage 4196 (Thorpe Ness)	1.99	2.13	2.31	2.45	2.60	2.79	2.95	3.12
Chainage 4202 (Slaughden)	2.08	2.22	2.40	2.54	2.70	2.90	3.07	3.25
Chainage 4208 (Orford Ness)	2.18	2.32	2.50	2.66	2.81	3.03	3.21	3.40
Chainage 4214 (Orford Beach)	2.39	2.53	2.71	2.87	3.02	3.25	3.43	3.63
Chainage 4218 (Shingle Street)	2.52	2.66	2.84	3.00	3.15	3.38	3.56	3.75
Chainage 4222 (Bawdsey)	2.62	2.76	2.94	3.09	3.25	3.47	3.65	3.84

Table 4. Mean and maximum wave parameters recorded at the Sudbourne Beach AWAC during the period October 2006 and October 2009, and for the single largest event which occurred on 01/02/2009.

Significant wave Height, Hs (m)	Maximum wave height, Hmax (m)	Zero crossing period, Tz (s)	Peak period, Tp (s)
Mean values (2006-2009) 0.52	0.82	3.45	5.44
Maximum values (2006-2009) 2.95	4.92	8.40	33.30
Maximum event (01/02/2009) 2.95	4.92	3.47	7.50

Table 5. Threshold values of the barrier inertia parameter (B_i) for which overwashing is considered possible, calculated using the formulae of Bradbury (2000) and Obhrai *et al*. (2008).

Significant wave height, H_s (m)	Wave period, T_m (s)	Wavelength, L_m (m)	H_s/L_m	Threshold value (Bradbury, 2000)	Threshold value (Obhrai *et al*., 2008)
3	3.5	19.13	0.16	na	na
3	5.5	47.23	0.06	na	1.10
3	7.5	87.82	0.03	na	5.70
3	9.5	140.91	0.02	10.85	14.12
5	3.5	19.13	0.26	na	na
5	5.5	47.23	0.11	na	na
5	7.5	87.82	0.06	na	2.17
5	9.5	140.91	0.04	na	5.54

Table 6. Comparison of maximum depth (m), width (m) and cross-sectional area (m^2) of the Alde-Ord estuary at three locations, based on EA swath bathymetry and LiDAR surveys undertaken in 2012.

	Mouth	KP15	KP11
Maximum Depth	-6.33	-7.46	-4.63
Width at HAT	215	286	345
Width at MHWS	216	278	337
Width at MHWN	209	275	332
Width at MSL	204	247	297
Width at MLWN	200	225	274
Width at MLWS	192	208	260
Width at LAT	147	184	235
Area at HAT	997	1529	1209
Area at MHWS	961	1482	1165
Area at MHWN	885	1386	1076
Area at MSL	720	1174	878
Area at MLWN	629	1063	770
Area at MLWS	560	979	685
Area at LAT	426	811	503

Table 7. Tidal volumes below defined tidal levels within the active estuary (north and south of Slaughden). Values are in millions of cubic metres (x 10^6 m^3).

	Extreme Surge	Dec 2013 Surge	HAT	MHWS	MHWN
upper level:	3.5 m OD	3.1 m OD	1.7 m OD	1.5 m OD	1.1 m OD
lower level:	-1.6 m OD	-1.6 m OD	-1.6 m OD	-0.7 m OD	-0.3 m OD
Total volume below levels					
Active Estuary (North)	25.08	22.34	13.15	11.97	9.81
Active Estuary (South)	43.07	39.20	26.96	25.56	23.14
Active Estuary (North and South)	68.15	61.53	40.11	37.53	32.95
Tidal prism below levels (excluding sub-tidal volumes)					
Active Estuary (North)	22.77	20.02	10.83	8.57	5.66
Active Estuary (South)	30.51	26.64	14.41	10.16	6.30
Active Estuary (North and South)	53.28	46.67	25.25	18.73	11.96
Sub-tidal volumes					
Active Estuary (North)	2.31	2.31	2.31	3.40	4.15
Active Estuary (South)	12.55	12.55	12.55	15.40	16.84
Active Estuary (North and South)	14.87	14.87	14.87	18.80	20.99

Figures

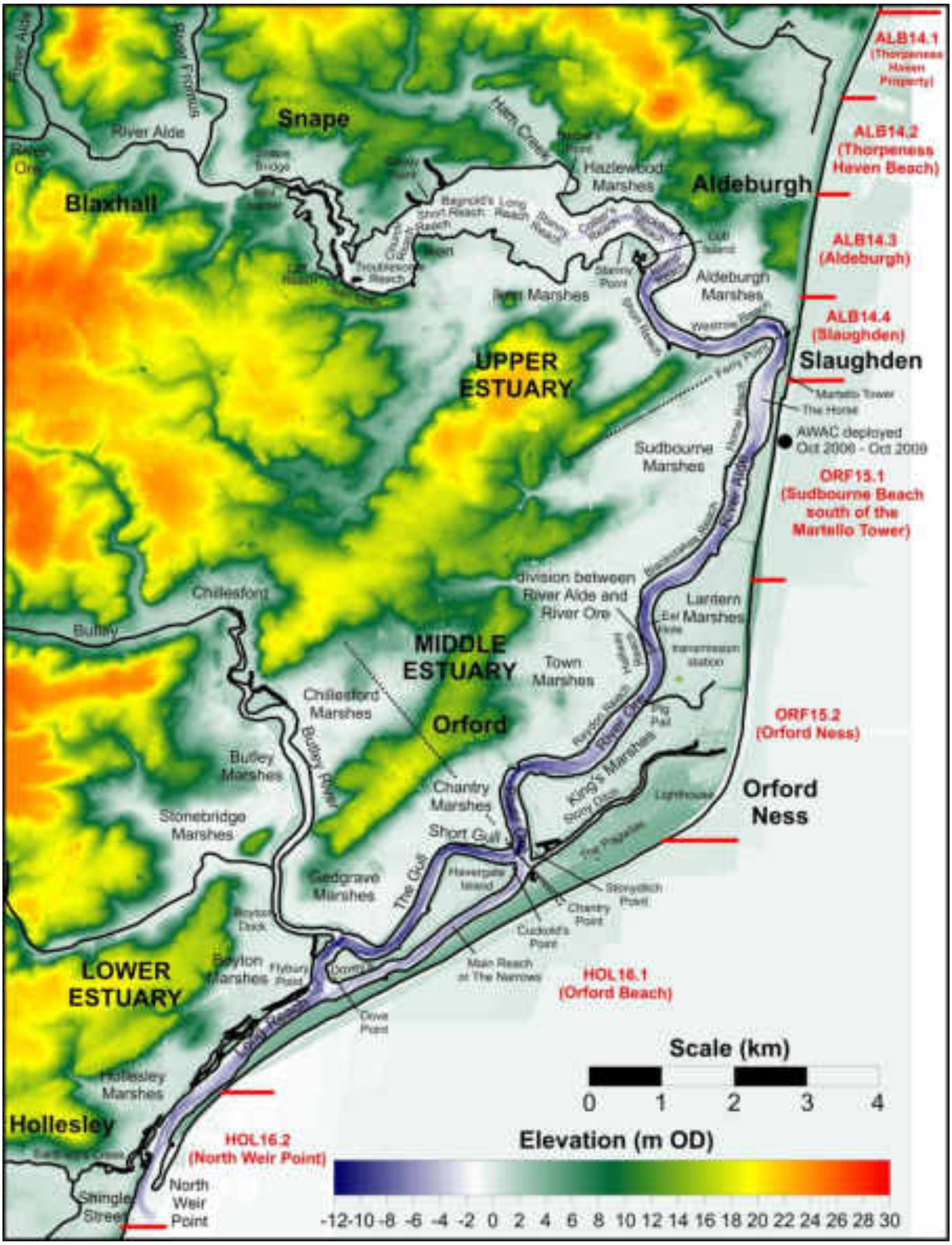

Figure 1. SMP2 policy units (red lines and lettering) overlaid on composite DEM derived from LiDAR and bathymetry data (2008-2012). Policy units are grouped into Management Area ALB 14 ('Thorpeness Haven to Aldeburgh'), ORF 15 ('Martello Tower to Orford Ness') and HOL 16 ('Orford Ness to Bawdsey Hill'). ALB 14 and ORF 15 in turn comprise Policy Development Zone PDZ 5 ('Thorpeness to Orfordness') and HOL16 comprises part of PDZ6 ('Orford Ness to Cobbold's Point'). The whole frontage is managed as part of Shoreline Management Plan 7 ('Lowestoft Ness to Felixstowe Landguard Point'). The location of rhe Sudbourne Beach AWAC deployment is also shown

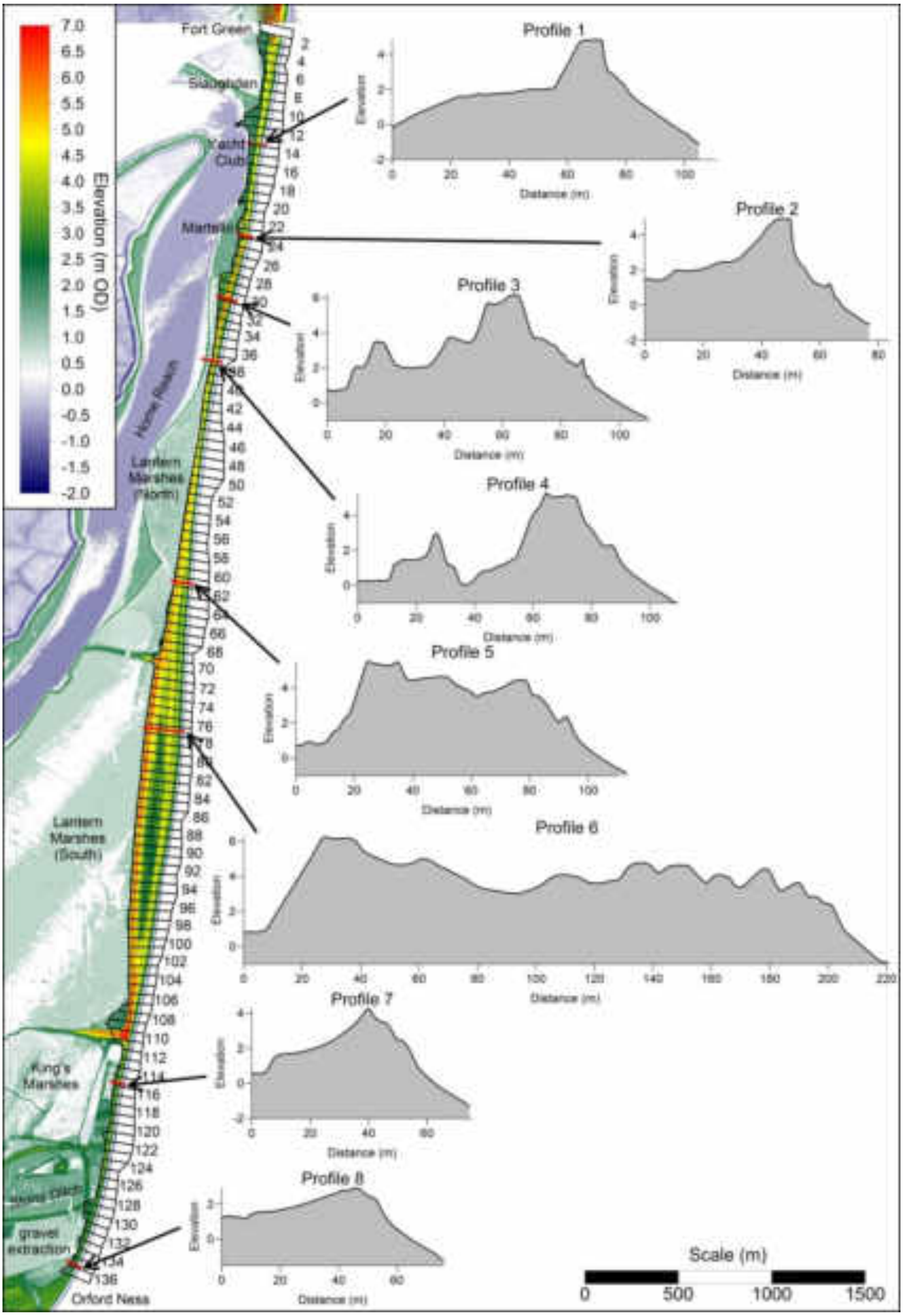

Figure 2. Beach and barrier cross-sections derived from airborne LiDAR survey on 25/01/2015. The boundaries of the defined cells used to calculate sediment volumes are also shown.

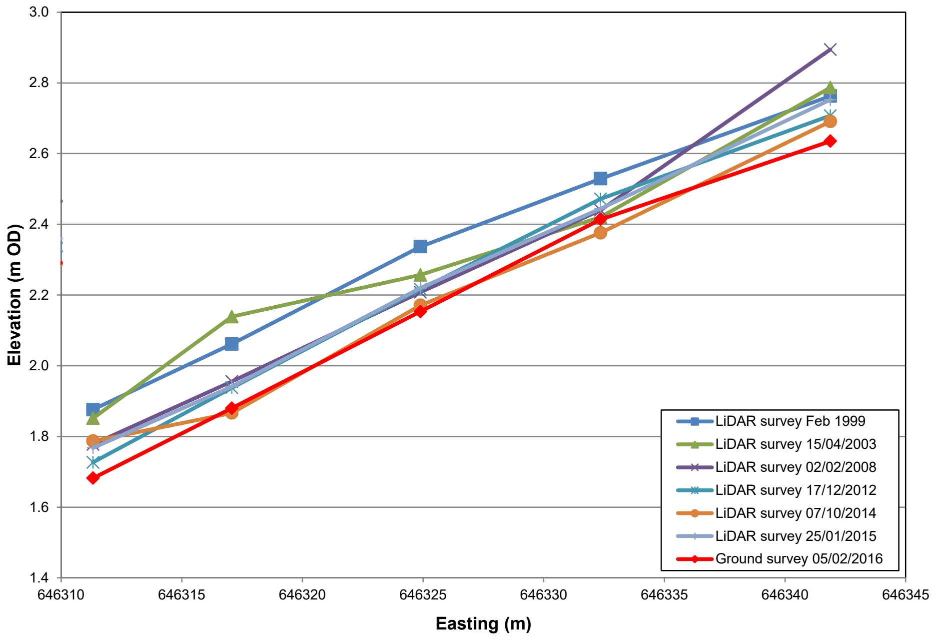

Figure 3. Comparison of surface levels along a profile across the Aldeburgh Yacht Club site determined from RTK GPS ground survey on 05/02/2016 and previous airborne LiDAR surveys. Compared to the ground survey (vertical accuracy c +/- 1 cm), the following average differences were calculated: Feb 1999 survey was 16 cm higher; 15/04/2003 survey was 11 cm higher; 02/02/2008 survey was 6 cm higher; 17/12/2012 was 6 cm higher; 07/10/2014 survey was 1 cm higher; 25/01/2015 survey was 7 cm higher. The LiDAR survey data were therefore adjusted vertically using these figures before beach and barrier sediment volumes were calculated

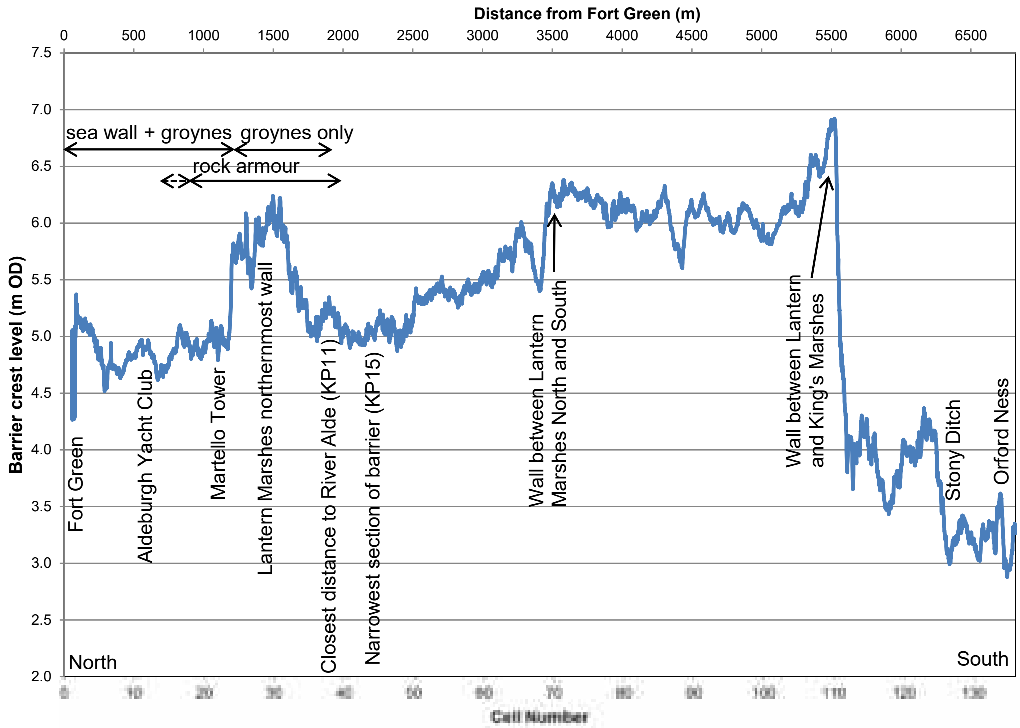

Figure 4. Variation in the crest height of the barrier, determined from the 25/01/2016 LiDAR survey

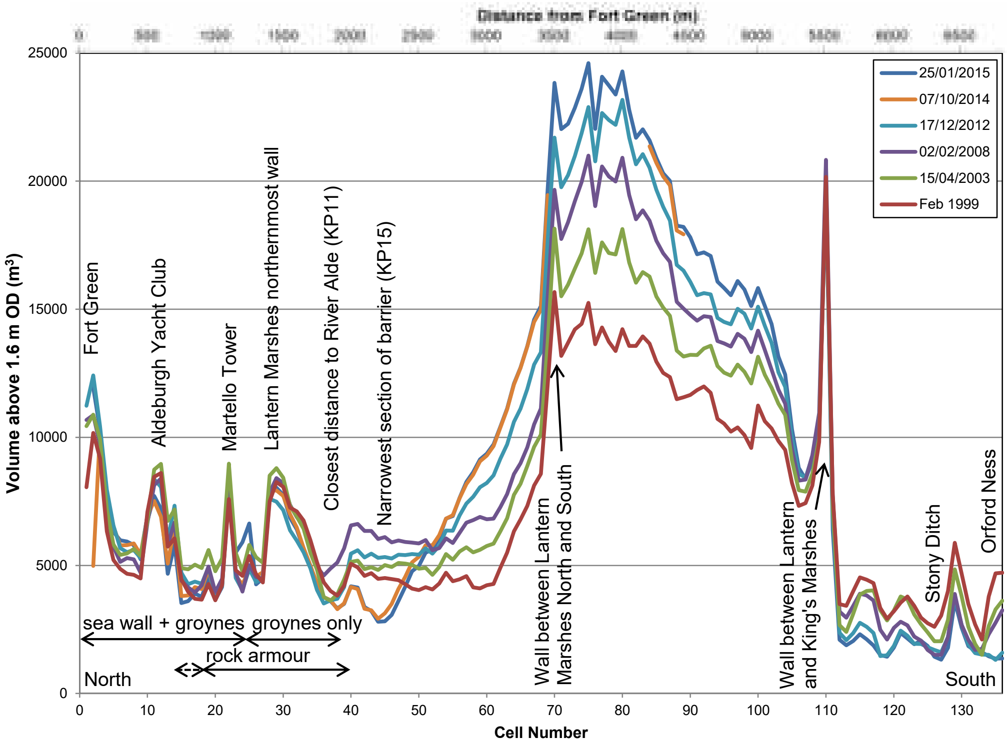

Figure 5. Alongshore variation in barrier volume above 1.6 m ODN (approximate HAT level) between Fort Green and Orford Ness, determined from six airborne LiDAR surveys between 1999 and 2015. Volumes were calculated seaward of the 1.6 m OD contour on the landward side of the barrier, in 136 cells, each 50 m wide, along the length of the barrier

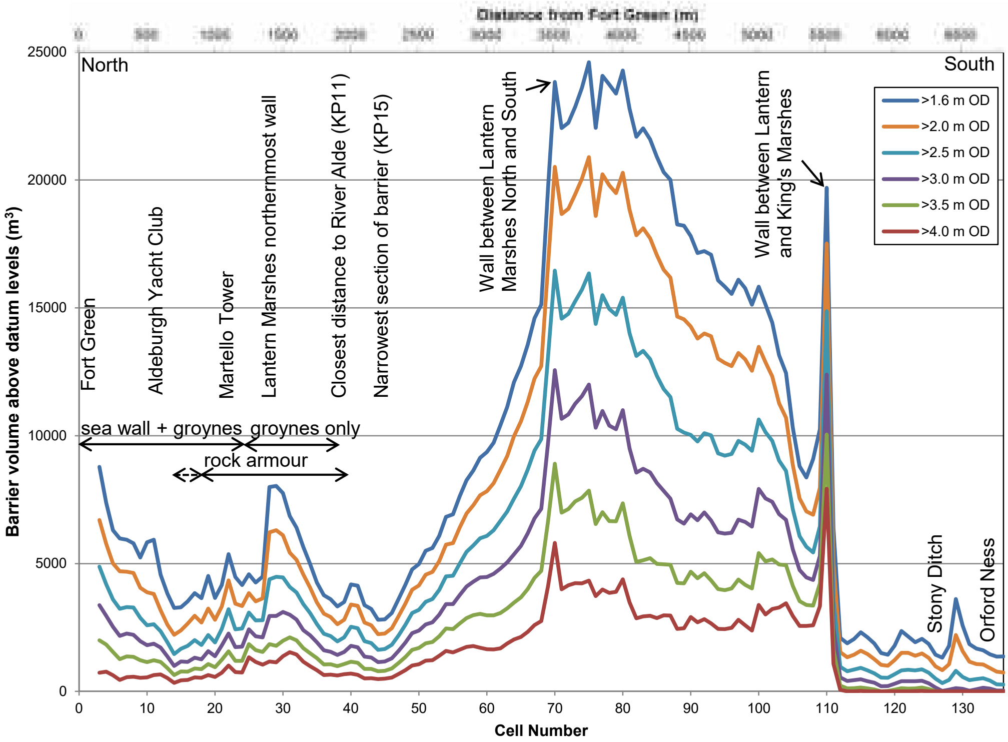

Figure 6. Alongshore variation in the barrier sediment volume above several different datum levels, and within 136 50 m wide cells, between Fort Green and Orford Ness, determined from the 25/01/2016 LiDAR survey. Volumes were calculated seaward of the 1.6 m OD contour on the landward side of the barrier

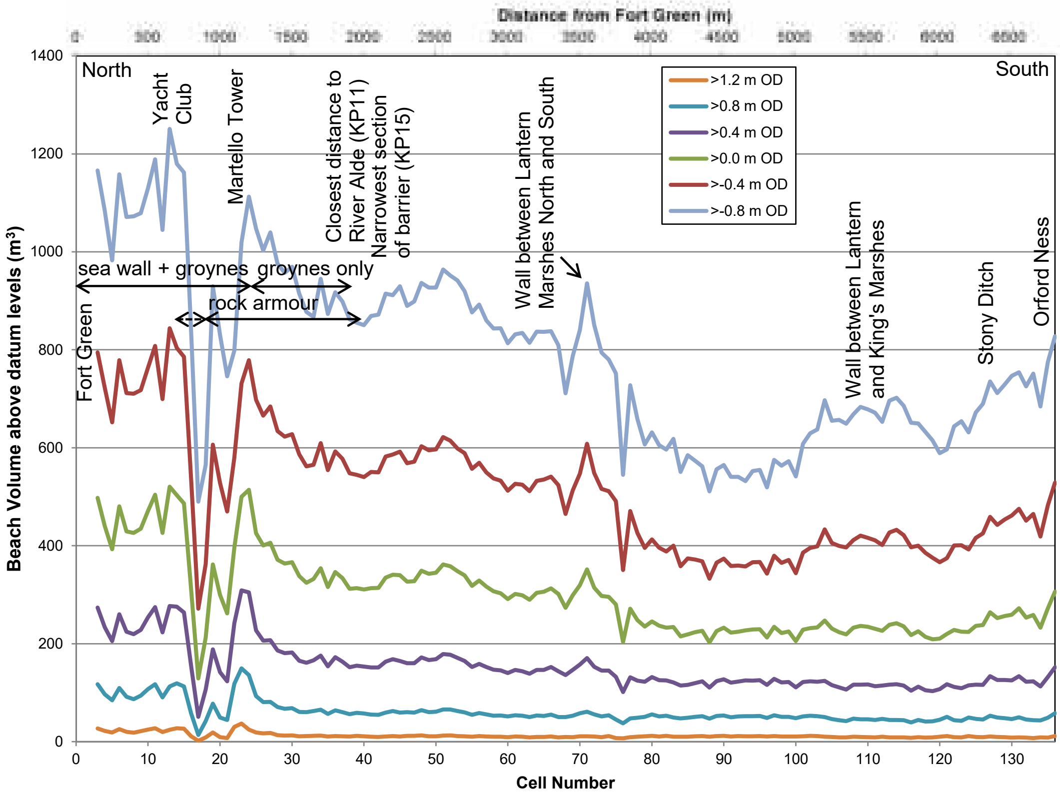

Figure 7. Alongshore variation in beach volume seaward of the 1.6 m ODN contour and above selected lower datum levels, and within 50 m wide cells, determined from the 25/01/2016 LiDAR survey

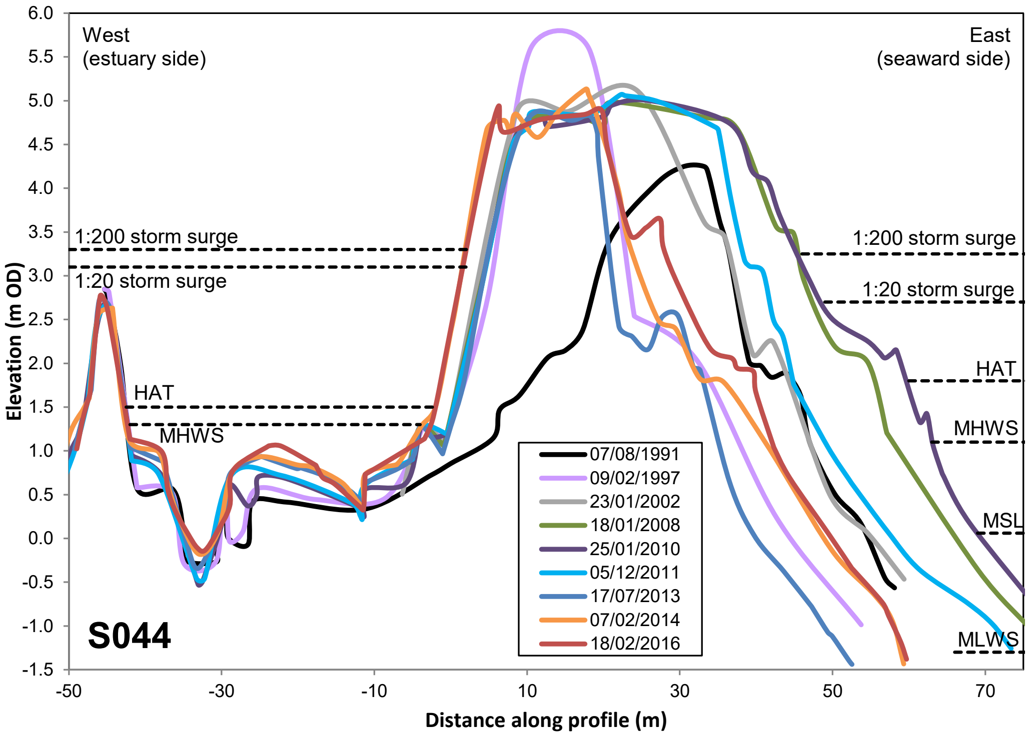

Figure 8. Changes in the cross-sectional morphology of the barrier at EA strategic profile S044 between 1991 and 2015

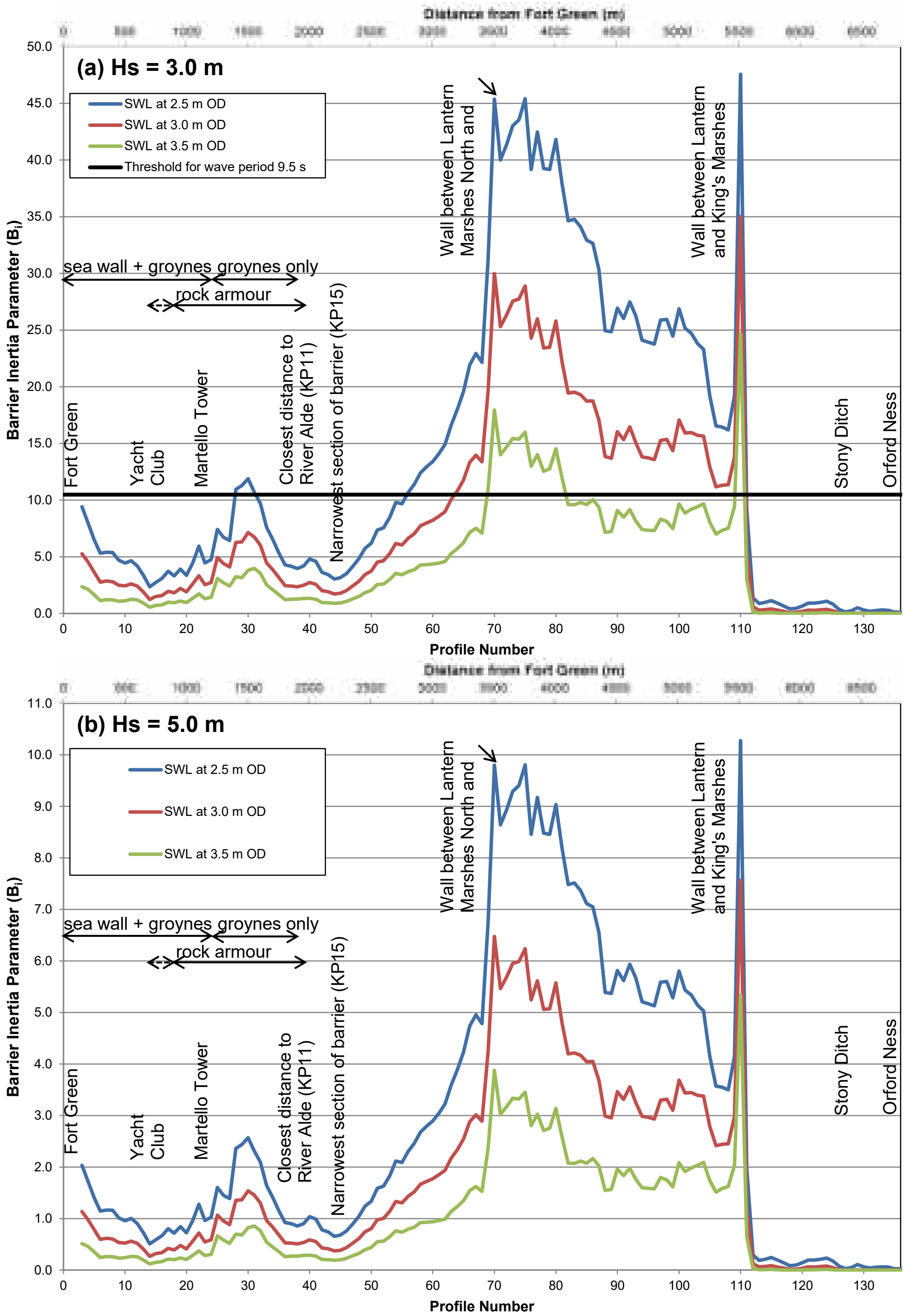

Figure 9. Alongshore variation in the Bradbury (2000) barrier inertia parameter (B_i), calculated for three different still water levels (SWL = 2.5 m, 3.0 m and 3.5 m ODN), morphological parameters extracted from the 25/01/2016 LiDAR DEM, and two significant wave heights (H_s = 3.0 and H_s = 5.0 m) selected following examination of the recorded at the Sudbourne Beach AWAC record 2006 - 2009. The critical threshold for overwashing was determined using Eqn. 4 (Section 4.2 above) for the significant wave heights and mean wave periods of 3.5 s, 5.5 s, 7.5 s and 9.5 s. Only those threshold values considered top be valid in terms of wave steepness are shown as horizontal black lines on the diagram (H_s/ L_m range of 0.015 to 0.032 quoted by Bradbury *et al*., 2005)

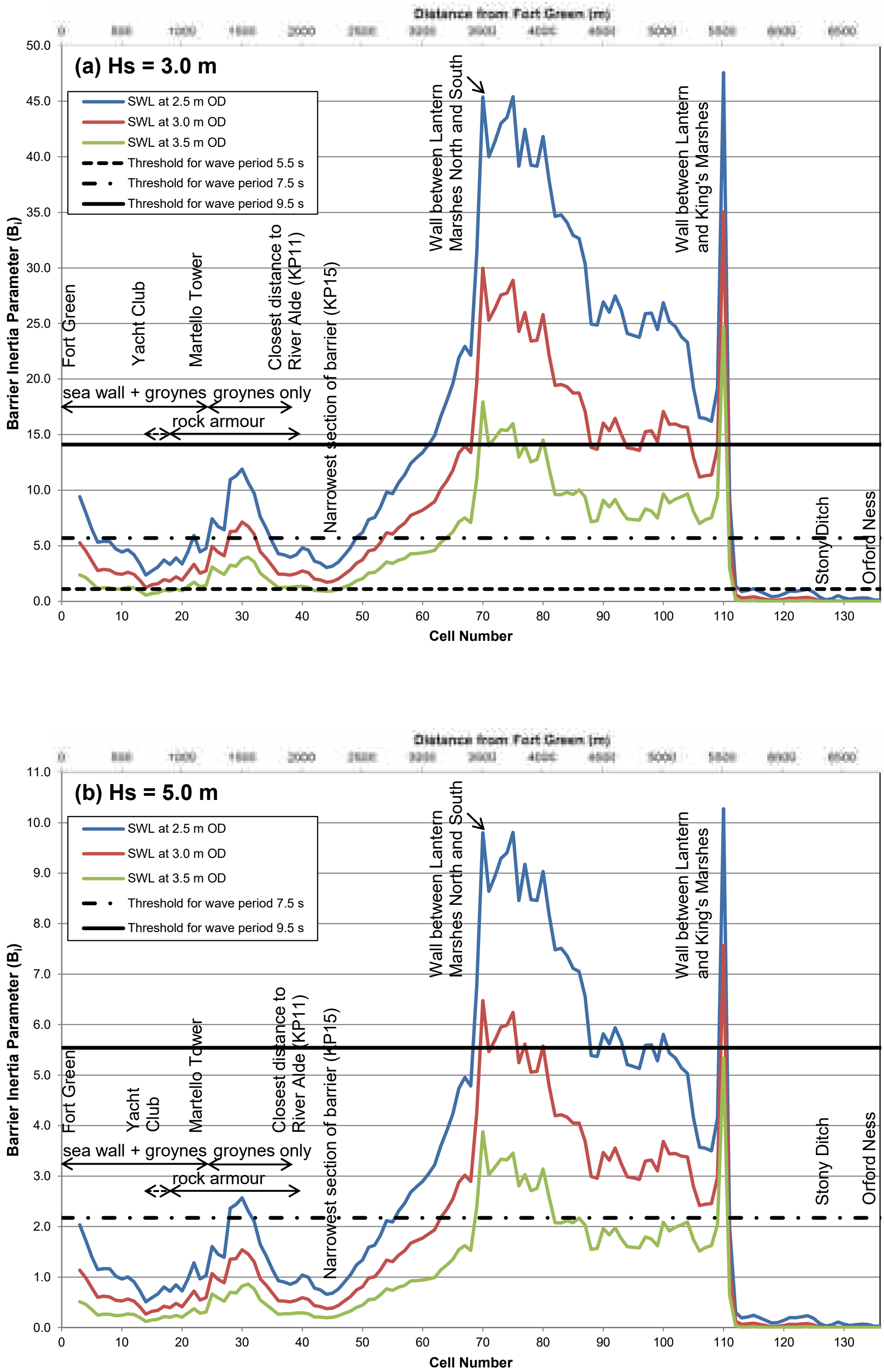

Figure 10. Alongshore variation in the barrier inertia parameter (B_i), calculated using the modified formulae proposed by Obhrai *et al.* (2008), for three different still water levels (SWL = 2.5 m, 3.0 m and 3.5 m ODN), morphological parameters extracted from the 25/01/2016 LiDAR DEM and two significant wave heights (H_s = 3.0 m and 5.0 m). The critical threshold for overwashing was determined using Eqn. 5 (Section 4.2 above) for the significant wave heights and mean wave periods of 3.5 s, 5.5 s, 7.5 s and 9.5 s. Only those threshold values considered top be valid in terms of wave steepness are shown as horizontal black lines on the diagram (H_s/ L_m range of 0.01 to 0.06 quoted by Obhrai *et al.* 2008)

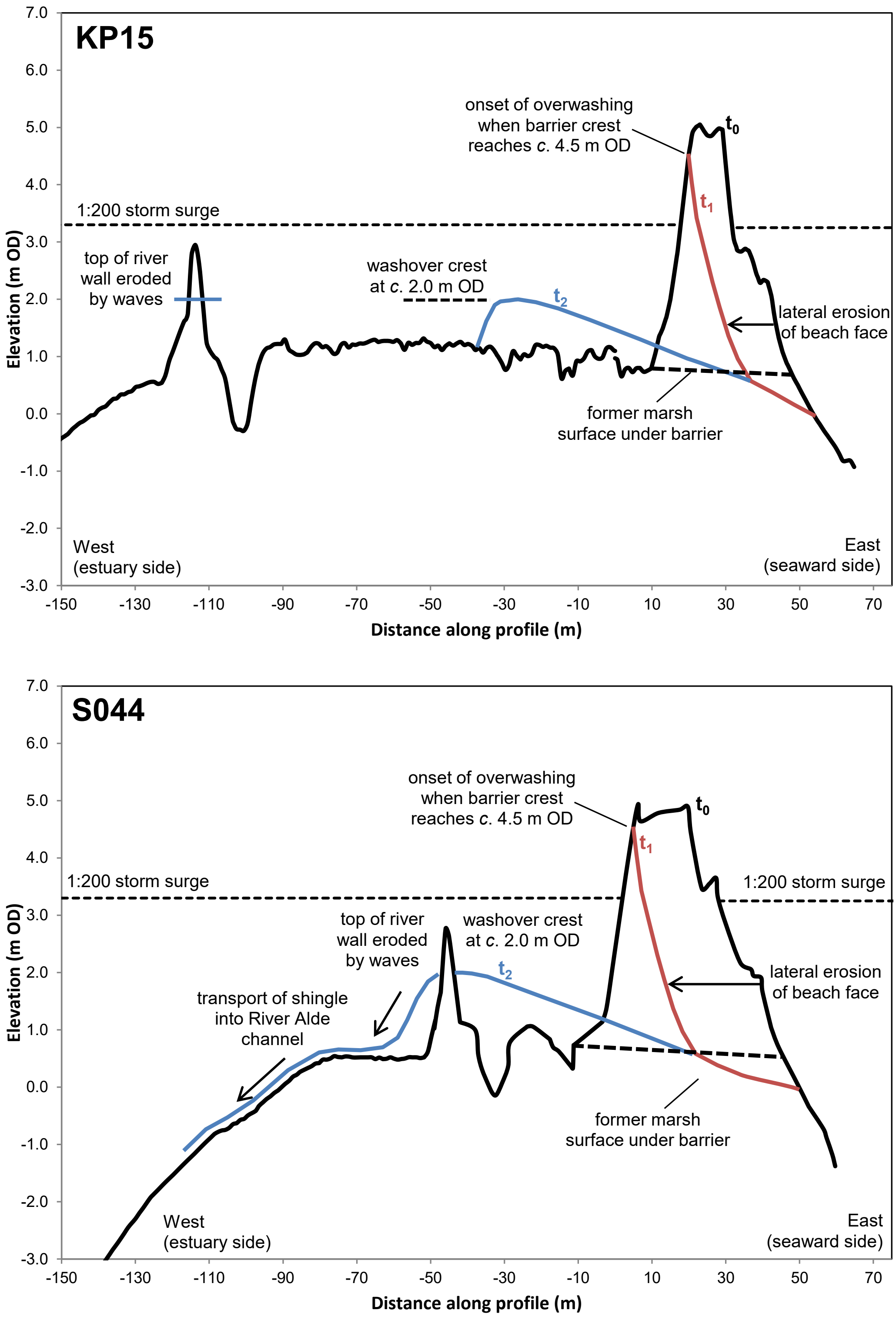

Figure 11. Schematic model showing the likely future evolution of the artificially maintained barrier at profiles KP15 and SO44, under a policy of NAI. If the beach sediment budget in this area remains negative, storms will cause recession of the seaward barrier face to a point where it removes the ridge crest, intersects the backslope, and overwashing becomes possible (change from t0 to t1). Subsequent storms will transport much of the remaining barrier sediment landwards, covering the marsh and infilling channels in the back-barrier area (change from t1 to t2). Waves passing over the flattened barrier will impact on the river wall, resulting in its progressive erosion; at that point overwashed shingle will be lost into the river

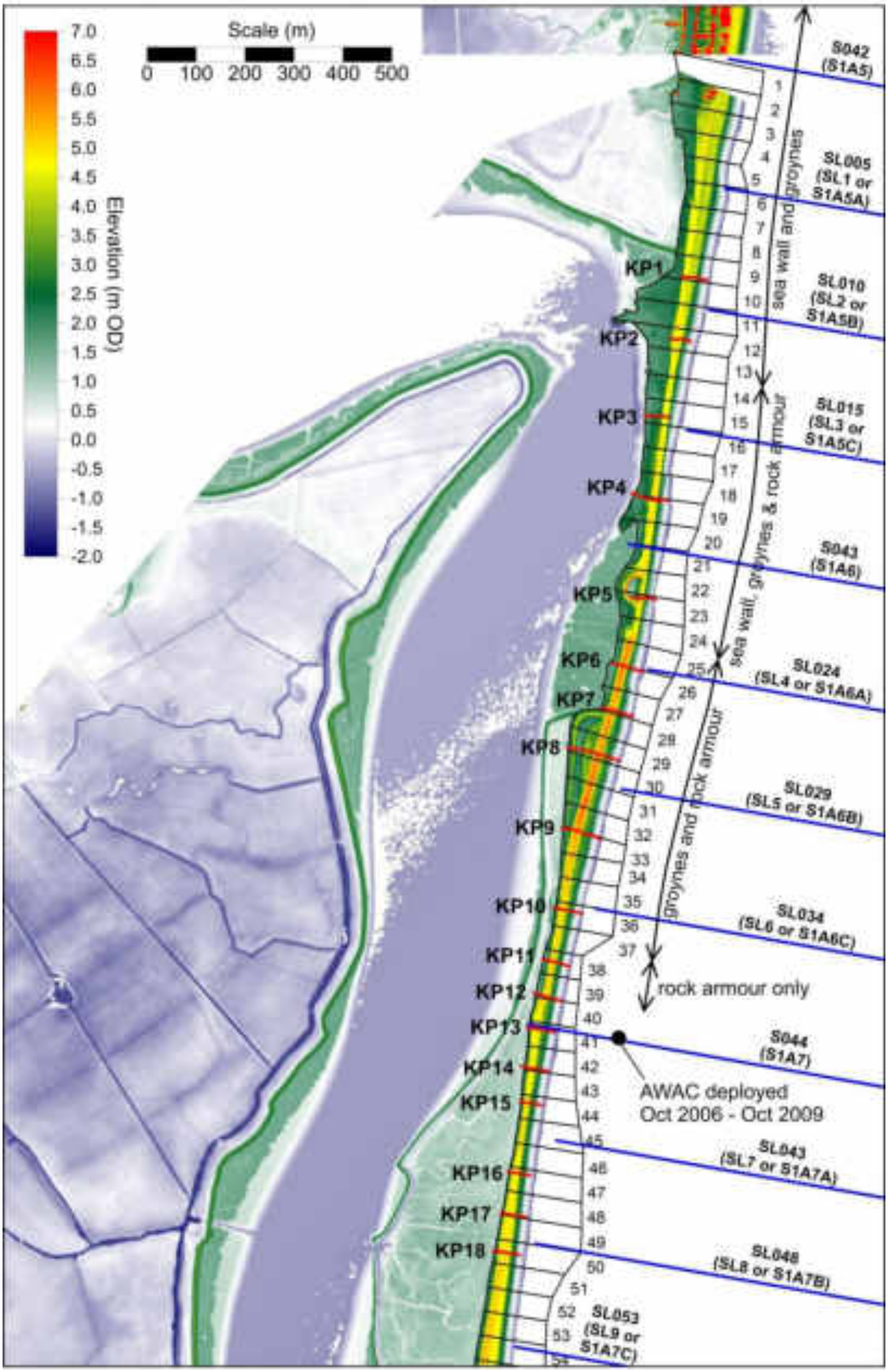

Figure 12. Locations of beach and nearshore bathymetry profiles overlain on LiDAR DEM surveyed on 25/01/2015: red lines surveyed by KPAL on 05/02/2016 using ground RTK-GPS; blue lines surveyed by single beam echo sounder on 24/08/2007 and 14/03/2008 by EA contractors; black lines indicate cells used for sediment volume calculations

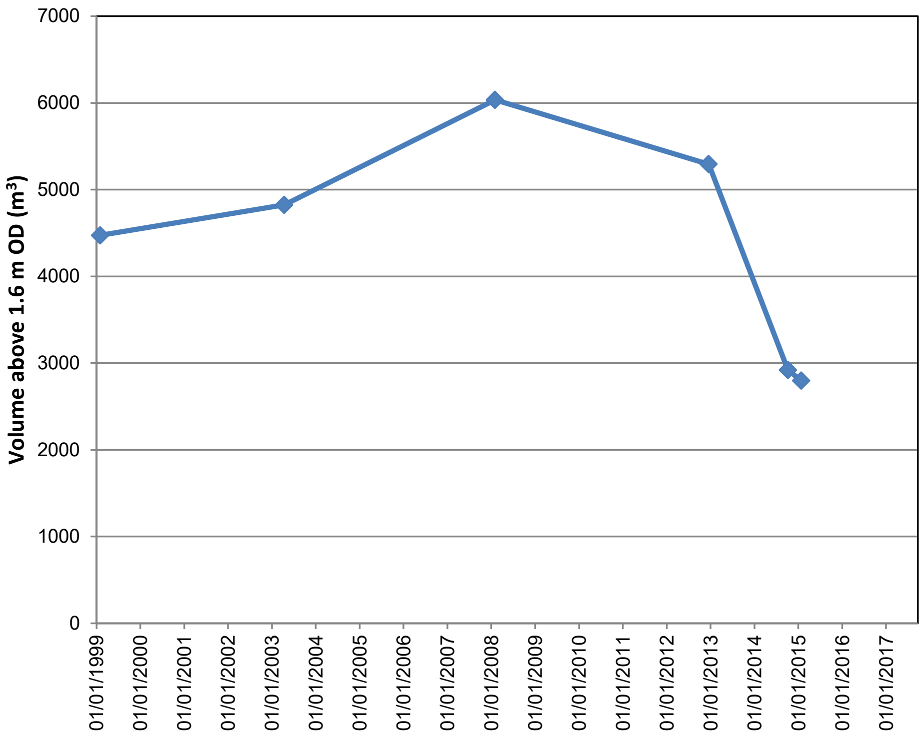

Figure 13. Changes in the sediment volume of the barrier above 1.6 m OD in Cell 44 (containing Profile KP15) over time, determined from airborne LiDAR surveys

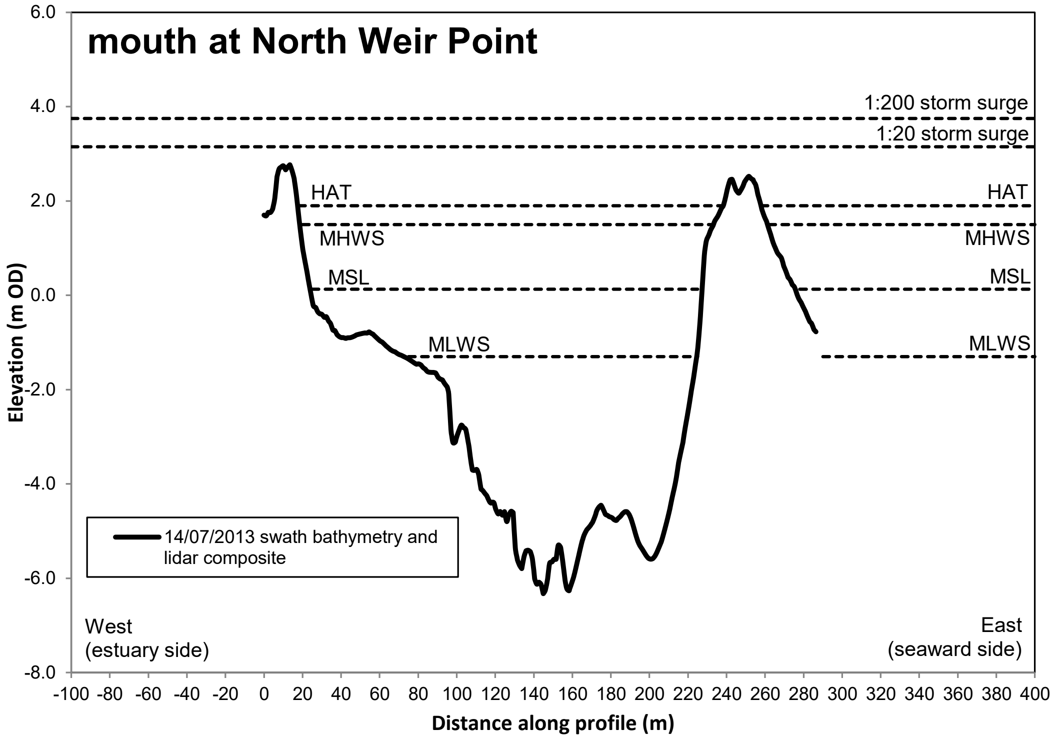

Figure 14. Cross-sectional morphology of the estuary at North Weir Point channel taken from the combined 27/12/2012 LiDAR and 14/07/ 2013 swath bathymetry DEM; the seaward end of the mid-channel shoal is just capture

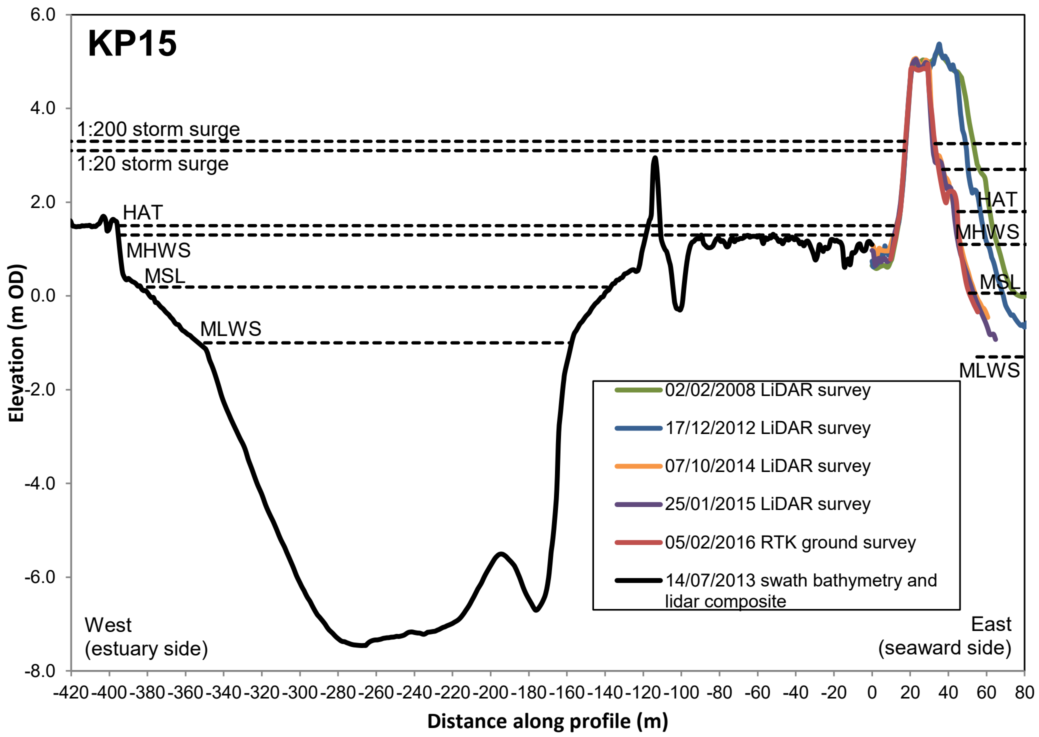

Figure 15. Cross-sectional morphology of the barrier and estuarine channel at profile KP15 (within Cell 44) based on sequential Lidar surveys, and cross-section of the Alde channel taken from the combined 27/12/2012 LiDAR and 14/07/ 2013 swath bathymetry DEM.

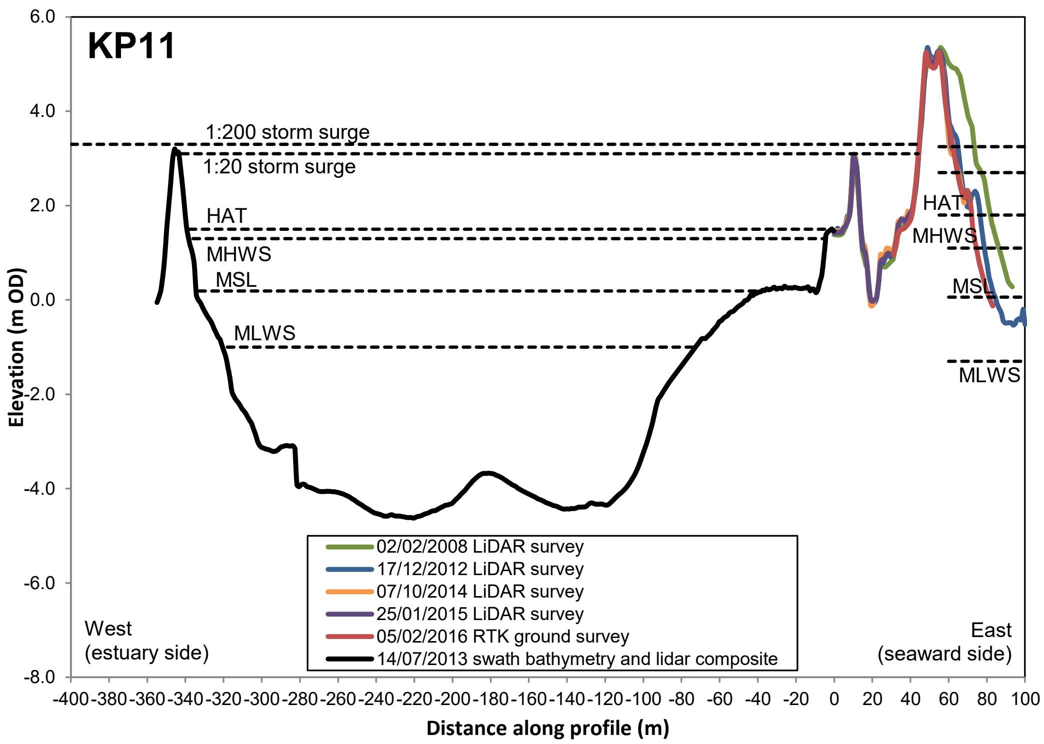

Figure 16. Cross-sectional morphology of the barrier and estuarine channel at profile KP11 (within Cell 38) based on sequential Lidar surveys, and cross-sectional morphology of the Alde channel taken from the combined 27/12/2012 LiDAR and 14/07/ 2013 swath bathymetry DEM. Note the very limited accommodation space for barrier roll-over