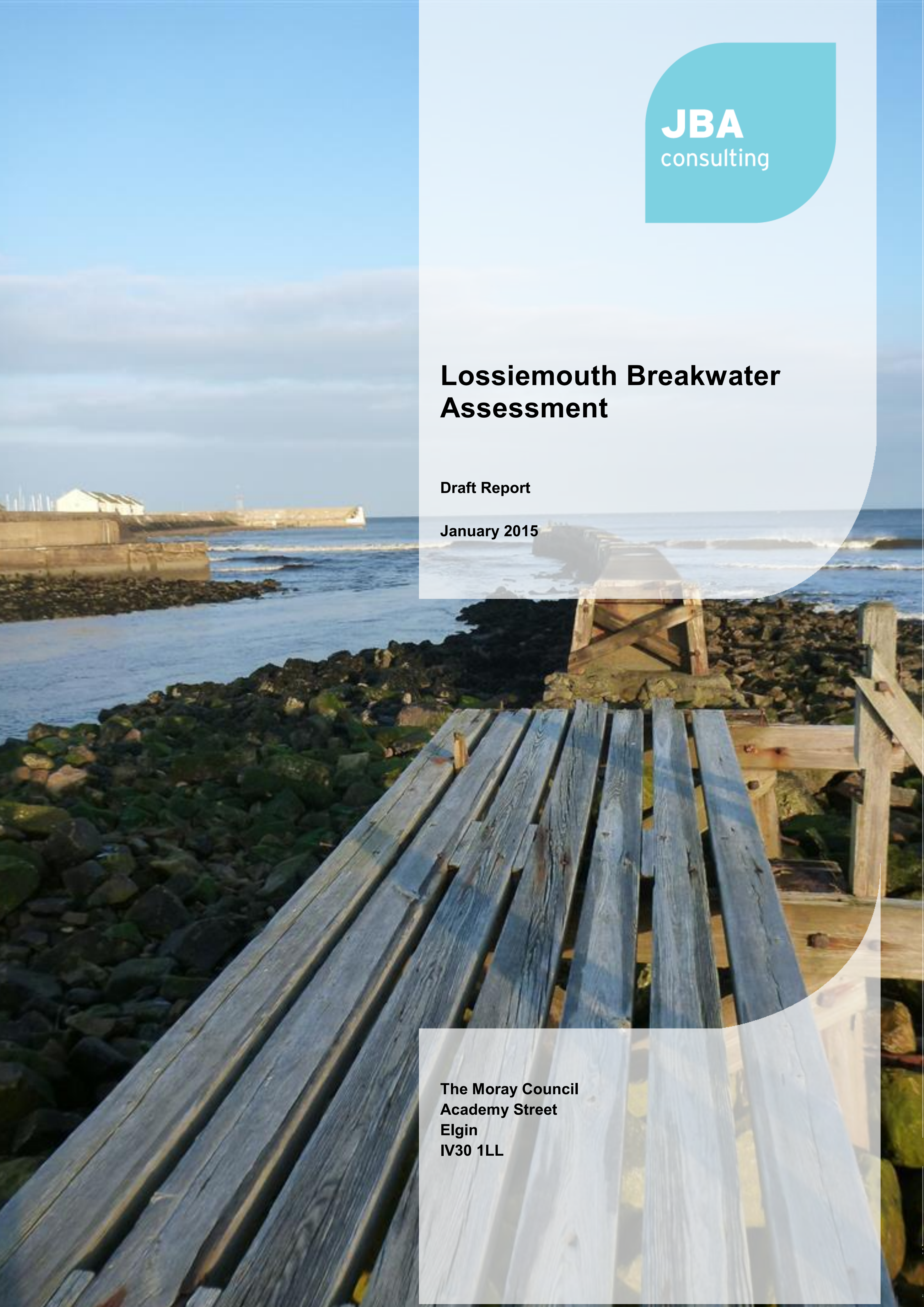

Lossiemouth Breakwater Assessment

Draft Report

January 2015

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Revision History

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Draft v1 / 23 January 2015		Debbie Halliday, Moray Council

Contract

This report describes work commissioned by Debbie Halliday, on behalf of The Moray Council, by a letter dated 3 November 2014. The Moray Council's representative for the contract was Debbie Halliday. Nicci Buckley, Dan Rodger, Johan Skanberg-Tippen and Graham Kenn of JBA Consulting carried out this work.

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Purpose

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Acknowledgements

JBA would like to thank Moray Council for the supply of data and information for use in the study.

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Abbreviations

2D	Two Dimensional (modelling)
DEFRA	Department of the Environment, Food and Rural Affairs (formerly MAFF)
DTM	Digital Terrain Model
EA	Environment Agency
FWS	Flood Warning System
ID	Identifier
LiDAR	Light Detection And Ranging
mAOD	metres Above Ordnance Datum
NERC	Natural Environment Research Council
QMED	Median Annual Flood (with return period 2 years)
SEPA	Scottish Environment Protection Agency
SSSI	Site of Special Scientific Interest

1 Introduction

JBA have been commissioned by Moray Council to undertake an assessment of the 'Old Pier' breakwater in Lossiemouth. The Old Pier is a historic river training wall constructed to the east of the River Lossie, at East Beach. Following a number of large storm events in recent years, including the damaging winter 2013-14 storms, the Old Pier has become degraded and concerns have been raised by local residents regarding the condition of the breakwater. This study has been undertaken to assess the influence of the Old Pier, with particular reference to:

- The benefits offered by the breakwater
- The consequence of partial, or complete removal of the breakwater
- Possible repair or rebuild solutions

1.1 Project background

Lossiemouth is located on the north facing coastline of the Moray Firth, Scotland, adjacent to the River Lossie, as shown in Figure 1-1. To the east the landscape is dominated by a sand-topped shingle spit and a long sandy beach that extends for approximately 12km to Kingston-on-Spey and beyond to Portgordon. The town itself sits on a rocky promontory, with the rocky shoreline designated a Site of Special Scientific Interest (SSSI) for its geological interest. West of this rocky promontory, the sand and shingle beaches continue for approximately 4km, followed by a stretch of rocky cliffs and small pocket beaches that extend to Burghead.

The Lossiemouth coastline is varied in its nature, including low cliffs and rocky platforms, intertidal flats, sand dunes, salt marsh and carse land habitats (Figure 1-2)[1]. A sizable dune complex is supported at the western end of the spit, providing natural protection to the town from waves. The River Lossie flows north from Elgin, and the spit acts to divert the Lossie estuary to the northwest, along the back of the spit until it reaches the rocky headland upon which the town is located. It is then directed north-east past the old harbour, now a promenade adjacent to Seatown Road. The river discharges into the Moray Firth through two training structures, a left bank pier and the Old Pier (refer to Figure 1-3 and Figure 1-4). The left bank pier acts to deflect the flow eastwards against the breakwater and thus away from the harbour entrance to the north where the flow jet may affect navigation. The Old Pier, located at the western end of the sand spit, then redirects flows north. This study investigates the effect of the Old Pier on wave and sediment processes.

Other features of interest to this study are the old harbour located on the opposite bank at the mouth of the River Lossie, the long footbridge which provides pedestrian access from the town to the spit and beach, and the Spynie Canal, which drains the hinterland and discharges into the River Lossie near its mouth. An overview of the history and development of these features is detailed within Section 2.1.

[1] Hansom, H.D. & Black, D.L. 1996 Coastal Processes and Management of Scottish Estuaries II: Estuaries of the Outer Moray Firth. Scottish Natural Heritage Review No. 51.

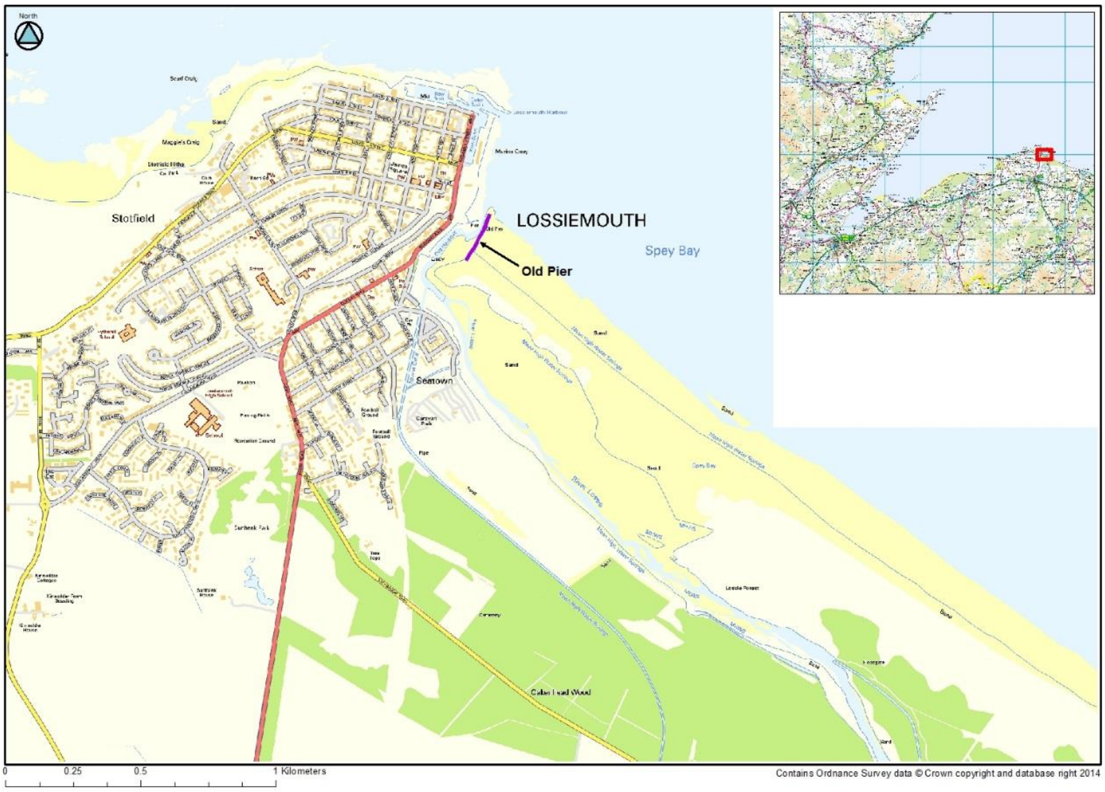

Figure 1-1: Site location plan

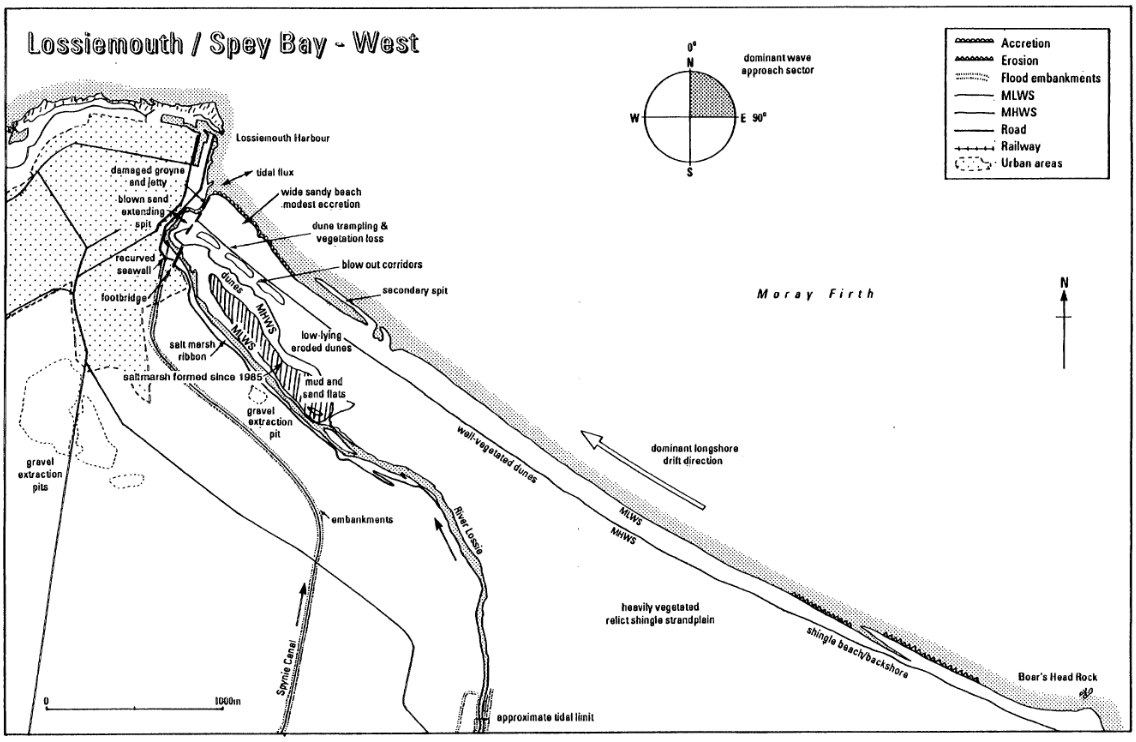

Figure 1-2: General features and layout[1]

Figure 1-3: Old Pier, Lossiemouth

Figure 1-4: Old Pier and surrounding features

1.2 Site visit and condition summary

A site walkover was undertaken by Nicci Buckley of JBA Consulting on 16 December 2014. This has been supported by previous inspections undertaken by JBA engineers on 10 March 2014, which coincided with coastal inspections around the Moray Firth.

During the site visit it was noted that the breakwater is comprised of two sections; a timber groyne at the landward extent, and a mass concrete breakwater arm at the seaward end (Figure 1-5). Light grade rock armour surrounds both the timber and concrete structures acting as toe and scour protection.

Figure 1-5: Breakwater section type and condition

The timber and concrete elements of the breakwater are damaged and in a state of disrepair. Sections of the timber breakwater are completely missing, and are visible along the right hand bank of the River Lossie upstream of the footbridge as well as in the channel beneath the bridge. The landward concrete element stretching between chainage ~100m and 120m has had repairs in recent years, and is considered to be in a fair condition. However, the seaward concrete element stretching from ~120m to 204m has extensive fractures, voids, spalls, and has been severely undermined. Examples of these defects are presented in Figure 1-6 to Figure 1-8, with additional photographs from the site visit presented within Appendix A.

The pier present on the opposite bank of the River Lossie is comprised of concrete surrounded by rock armour toe protection. Damage to this structure is limited to the northern end, consisting mainly of concrete spalls with some minor fracturing where it is most exposed to storm conditions.

Debris that had been washed up on to the promenade were observed directly behind the left bank pier. This area forms a low point in the promenade wall and is also located directly opposite the gap in the breakwater, thus meaning that it may be more exposed to wave action from certain directions.

The River Lossie has a clear channel, defined along its left bank by the promenade wall (the old harbour wall) with a small amount of rock armour and along its right bank by sand deposits that have extended past the breakwater structure and extended the spit westwards.

Figure 1-6: Repaired breakwater (left), start of dilapidated structure (right). Courtesy of Moray Council[2]

Figure 1-7: Dilapidated mass concrete breakwater. Courtesy of Moray Council[2]

[2] Moray Council, 2014. Lossiemouth Breakwater Condition Report.

Figure 1-8: Dilapidated mass concrete breakwater. Courtesy of Moray Council[2]

2 Literature review

2.1 Background

Online resources provide a good overview of the history of Lossiemouth, the development of the harbour and estuary areas and the features being discussed herein.

Loch Spynie, now a small landlocked loch drained by the Spynie Canal, was historically a large sea loch and served as a natural harbour for Elgin[3]. However, over time the area was affected by heavy siltation and became blocked, with a succession of storms depositing large amounts of sand into the river mouth that eventually closed off the loch from the sea. It is believed that in order to prevent flooding, the River Lossie was subsequently diverted from the loch to its current location at Caysbriggs[4]. As a result, a new harbour was developed at the mouth of the River Lossie, commonly shown on maps as the 'Old Harbour'[3]. Reports note that the harbour was "ruinous and choked with sand" and that in 1780 an attempt was made to "increase the scour of the current by building another pier on the opposite bank"[5]; it is believed that this is what is now referred to as the 'Old Pier' that forms the focus of this study.

By the start of the 19th century the river harbour had become exceptionally busy and the Stotfield and Lossiemouth Harbour Company was formed in 1934 to oversee the construction of a new harbour, which continues to be used to the north-east of the town.

In the early 1900's a series of disused railway carriages were placed behind the beach in order to form sand dunes and provide shelter to Seatown and the section of the River Lossie that runs behind[3].

2.2 Structural report

A structural assessment of the Old Pier was undertaken by Neil Fotheringham, Senior Engineer at The Moray Council, on 28 October 2014.

The assessment noted that the structure is undermined in a number of places and that this appears to have deteriorated since the last survey undertaken in 2010. It is understood that concrete repairs were carried out in 2012, but that these were marginal and did not include the complete refurbishment of the structure to its original condition. It was further noted that a section of the timber breakwater has been washed out, with the timber now lying upstream of the footbridge.

The photographic report prepared following the assessment is provided within Appendix B.

2.3 Geology

The geology of the wider Moray Firth consists of metamorphosed Dalradian and Moinian sedimentary rocks with significant granitic intrusions, unconformably overlain by Lower, Middle and Upper Old Red Sandstone. Local to Lossiemouth the bedrock is comprised of a range of sedimentary rocks from the Old Red Sandstone series (Figure 2-1). These are overlain by superficial deposits largely comprised of raised or current marine deposits, as well as smaller pockets of alluvium and till (Figure 2-2).

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[3] http://www.thisismoray.com/lossiemouth-in-moray-c177.html

[4] http://en.wikipedia.org/wiki/Lossiemouth

[5] http://www.scotlandsplaces.gov.uk/record/rcahms/16729/lossiemouth-lossiemouth-harbour/rcahms

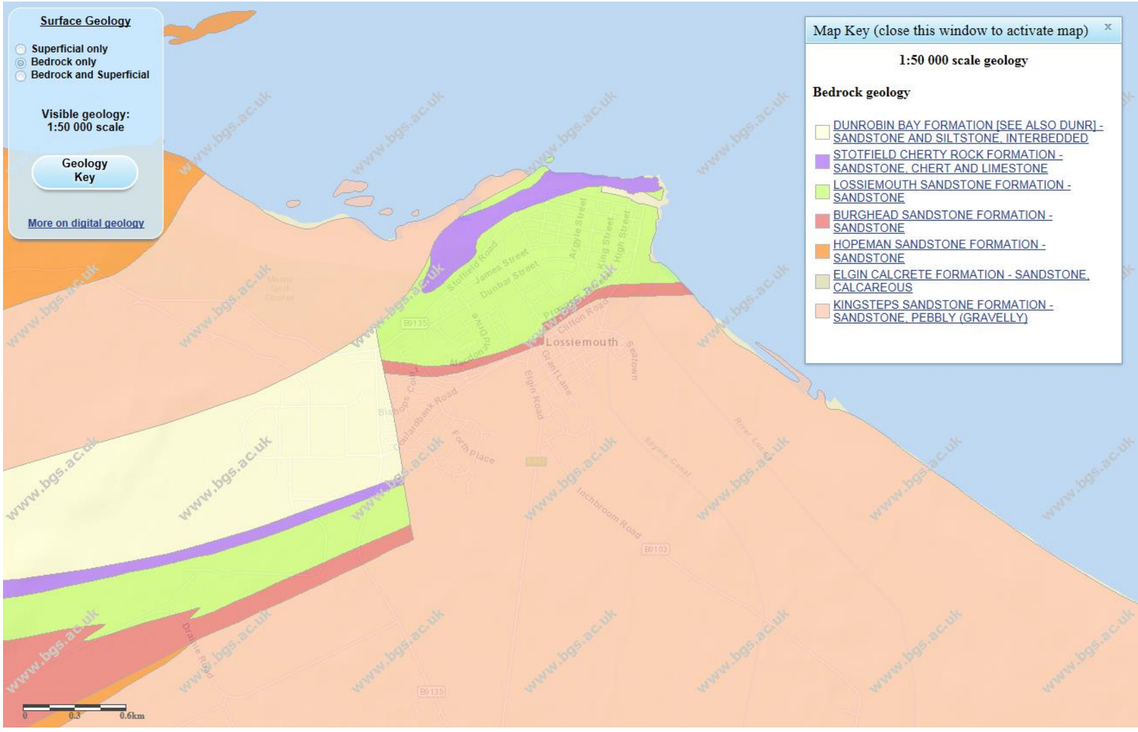

Figure 2-1: Bedrock Geology[6] (Contains British Geological Survey materials © NERC 2014)

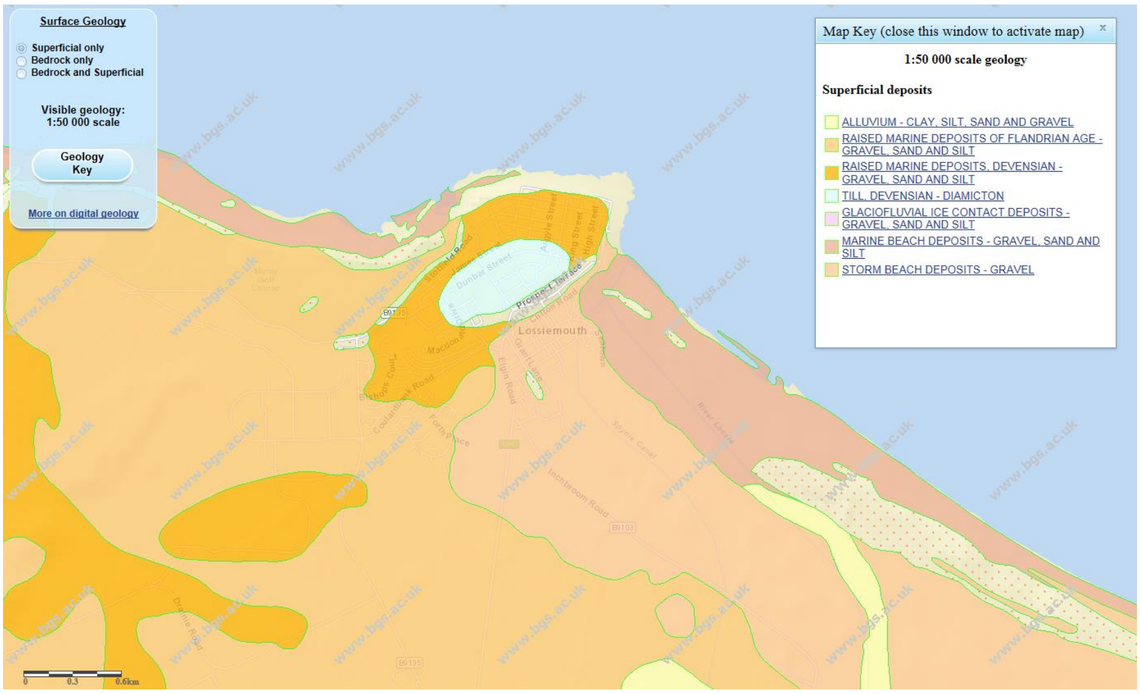

Figure 2-2: Superficial Geology[6] (Contains British Geological Survey materials © NERC 2014)

2.4 Coastal processes

An understanding of background processes is key to assessing sediment movements within the littoral zone. The rate and direction of such movements are influenced by a range of regional processes, such as waves and tidal currents, as well as local bathymetry, physical characteristics of the seabed and foreshore (e.g. sediment type and size) and local interactions. At Lossiemouth these include the river flows from the River Lossie, as well as the headland and constructed training walls which cause an interruption to the longshore sediment pathway.

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6 http://mapapps.bgs.ac.uk/geologyofbritain/home.html

2.4.1 Tidal regime

The Moray Firth is a mesotidal coastal system experiencing semi-diurnal tides. Lossiemouth has a spring tidal range of 3.5m and a neap tidal range of 1.6m, with each high tide propagating from west to east, i.e. high tide occurs earlier in Lossiemouth than in Fraserburgh[7].

Extreme sea levels can occur in the Moray Firth when adverse weather conditions, such as storm surges, coincide with high tides. The superimposition of weather conditions on the astronomical tide results in the still water level (SWL) at a given location. Key astronomical tidal levels[8], as well as extreme still water levels for a range of different return periods[9] for Lossiemouth are provided within Table 2-1.

Table 2-1: Tidal levels at Lossiemouth

Astronomical Tidal Level	Chart Datum (m)	Ordnance Datum (m)
Highest Astronomical Tide (HAT)	4.7	2.6
Mean High Water Springs (MHWS)	4.1	2.0
Mean High Water Neaps (MHWN)	3.2	1.1
Mean Low Water Neaps (MLWN)	1.6	-0.5
Mean Low Water Springs (MLWS)	0.6	-1.5
Lowest Astronomical Tide (LAT)	0.0	-2.1
SWL - Return Period (years)	**Chart Datum (m)**	**Ordnance Datum (m)**
1	4.78	2.68
2	4.85	2.75
5	4.93	2.83
10	5.00	2.90
25	5.08	2.98
50	5.14	3.04
100	5.20	3.10
200	5.26	3.16

2.4.2 Wave climate

Waves within the Moray Firth are comprised of both swell and wind waves. Swell waves are those generated beyond the North Sea and that propagate into the Firth from the 0 to 90° sector. Swell waves tend to have a more regular nature, composed of a well ordered wave train of similar characteristics (e.g. wave height, period and direction). Wind waves are those generated locally within the confines of the Firth, and can include a wider range of wave heights, periods and directions. The wave climate at a given time is a combination of these two elements, resulting in an irregular sea state.

Figure 2-3 shows wave height, period and direction data for the Moray Firth Directional Waverider buoy[10] from August 2008 to February 2014 (data recorded every 30 minutes). It can be seen that 55% of the recorded wave heights were less than 1m, with a peak recorded wave height of 7.66m occurring on 15 December 2012. The greatest frequency of waves occurred with a period between 3 and 4 seconds, with a maximum of 11 seconds occurring on 12 March 2011. The bimodal split in period indicates that the coastline is subject to wind and swell waves, introducing two different characteristic forms of wave loading. The directional data shows two clear bands, with waves originating from both the northeast and southwest, and the greatest frequency from between 50 and 60°. These two directional bands correspond to the orientation of the opening of the Firth and the prevailing south-westerly winds respectively. The analysis shows that the largest waves (those recorded over 5m wave height) originate from a distinct window between 90 and 100°. As waves approach the coastline they will be affected by processes like refraction, diffraction shoaling and breaking which will be dependent on the nearshore bathymetry and local features such as headlands and reefs.

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[7] Ramsay, D.L & Brampton, A.H. 2000. Coastal Cells in Scotland: Cell 3 – Cairnbulg Point to Duncansby Head. Scottish Natural Heritage Research, Survey and Monitoring Report No. 145.

[8] Total tide software

[9] Coastal Flood Boundary Conditions for UK Mainland and Islands, Project SC060064/TR2: Design Sea Levels

[10] http://cefasmapping.defra.gov.uk/Map

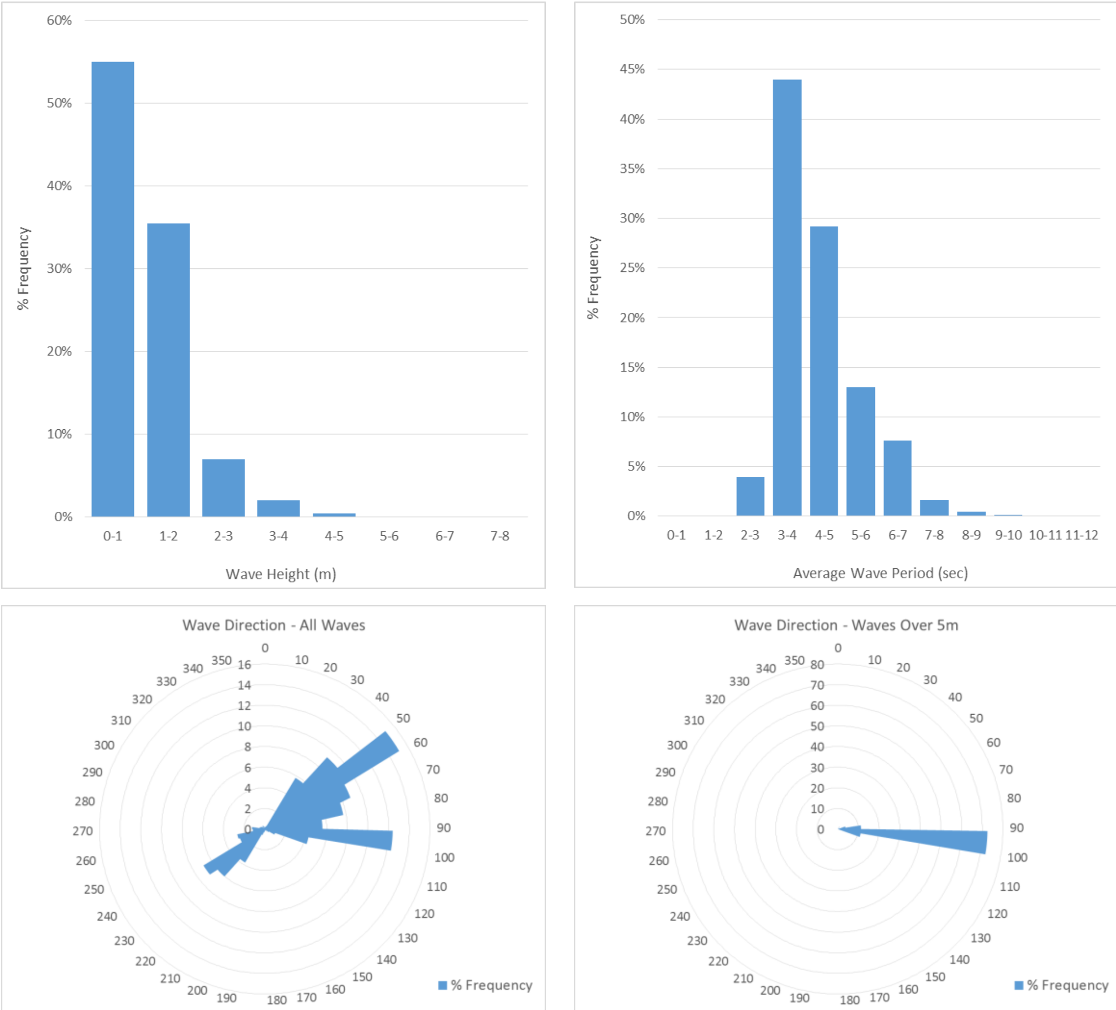


Figure 2-3: Wave Height, Average Wave Period and Wave Direction at the Moray Firth Waverider Buoy

2.4.3 Winds

Although located on a broadly north-facing coastline, the prevailing south-westerly winds from the Atlantic tend to be funnelled up through the Great Glen and Spey valleys. These south-westerly winds appear to be prevalent as far east as Lossiemouth and Spey Bay[1]. However, further east, for example at Fraserburgh, wind directions are more variable[1]. Figure 2-4 presents recorded wind data at Kinloss, approximately 20km to the east of Lossiemouth, and shows the dominance of south-westerly winds. These winds are the cause of the relatively high frequency of waves from the southwest, as shown in Figure 2-3.

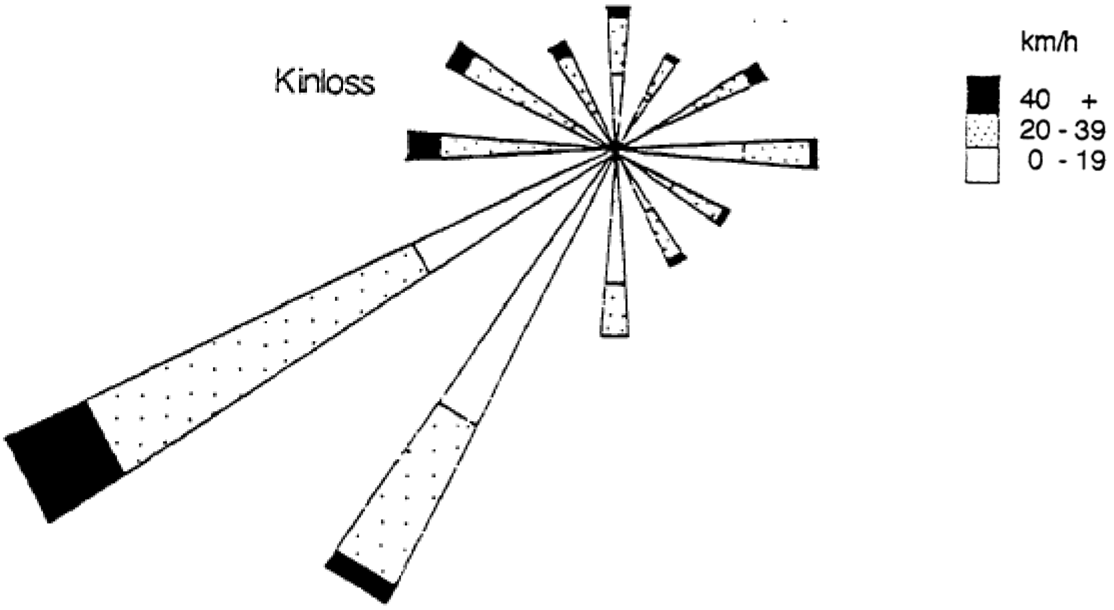


Figure 2-4: Wind Rose for Kinloss[1]

2.4.4 Currents

Current flow patterns in the centre of the Moray Firth are dominated by tidal action and are channelled by the fault-controlled nature of the sea bed[1]. Closer to shore, structural and alluvial shelves limit the tidal stream currents, with wave approach angles tending to drive currents parallel to the shoreline[1].

Tidal induced currents

Tidal currents within the Moray Firth are considered to be relatively low, with a peak spring rate typically less than 0.25m/s between Portknockie and Burghead[7]. Available reports suggest this increases slightly towards Spey Bay, where flood tide currents typically reach 0.4m/s and ebb tide currents 0.3m/s[11]. As the tide passes the Moray Firth, it floods southerly down the Caithness and Wester Ross coastline, and then east across the southern coast of the Firth, with current reversal on the ebb tide resulting in westward and then northward flow.

Due to the low tidal current speeds, wind and wave induced currents are considered to have a greater effect on nearshore current velocities.

Wave induced currents

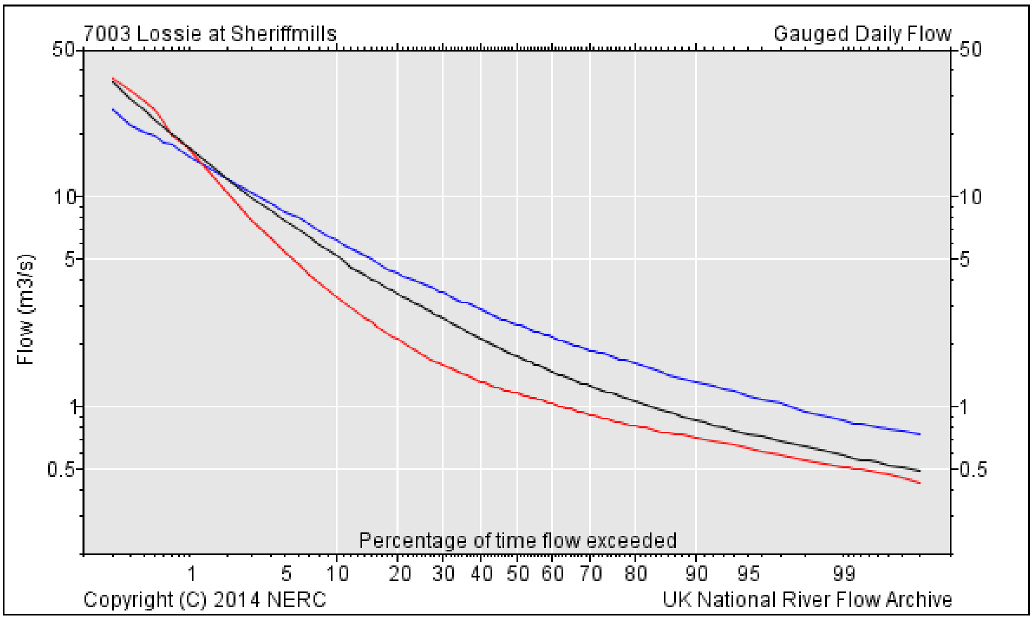
At Lossiemouth, the dominant angle of wave approach is from the north to northeast sector (Section 2.4.2). This dominance has a strong influence on the alignment of coastal features along the north facing coast of the Moray Firth. At Lossiemouth, where the coastline is aligned east-west, this incident wave angle meets the coast obliquely and results in a net westerly current.

2.4.5 Fluvial flows

Fluvial flows within the River Lossie also have an influence on local currents and sediment processes. The River Lossie is gauged at Sheriffmills (ref. 7003), approximately 20 km upstream of the river mouth. The flow duration curve at the gauge is presented within Figure 2-5, from which it can be seen that a flow of approximately 2 m^3/s is exceeded 50% of the time, and a flow of approximately 20 m^3/s is exceeded 1% of the time. QMED at the gauge is approximately 45 m^3/s[12]. The effect of these typically low flows, in combination with the flat grade of the lower estuary, result in a low energy potential at the outlet, prone to sediment accumulation.

[11] Spey Mouth – Management Review, Kingston-Upon-Spey, Royal Haskoning, Final Report, November 2012

[12] http://www.ceh.ac.uk/data/nrfa/data/station.html?7003



Key: Black line - annual; blue line - December to March; red line - June to September.

Figure 2-5: Flow Duration Curve for the River Lossie at Sheriffmills[12]

2.4.6 Sediment transport

Sediment sources

The stretch of coastline between Portknockie and Burghead is rich in beach sediment deposits, with Figure 2-6 showing the coastline in this area to be comprised of sand and shingle. These have been derived predominantly from glacial deposits, either from offshore or material reworked by fluvial processes[7].

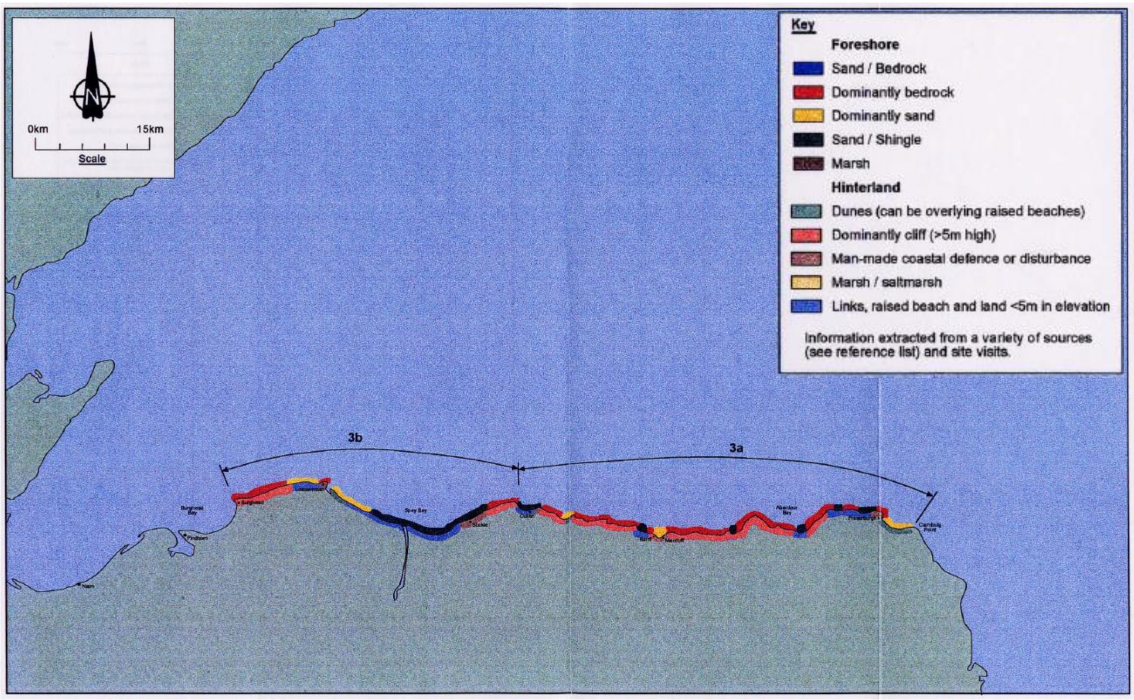


Figure 2-6: Foreshore and Hinterland Characteristics[7]

Offshore sediments within the Moray Firth are shown within Figure 2-7, from which it can be seen that the sediments offshore of Lossiemouth are predominantly sand.

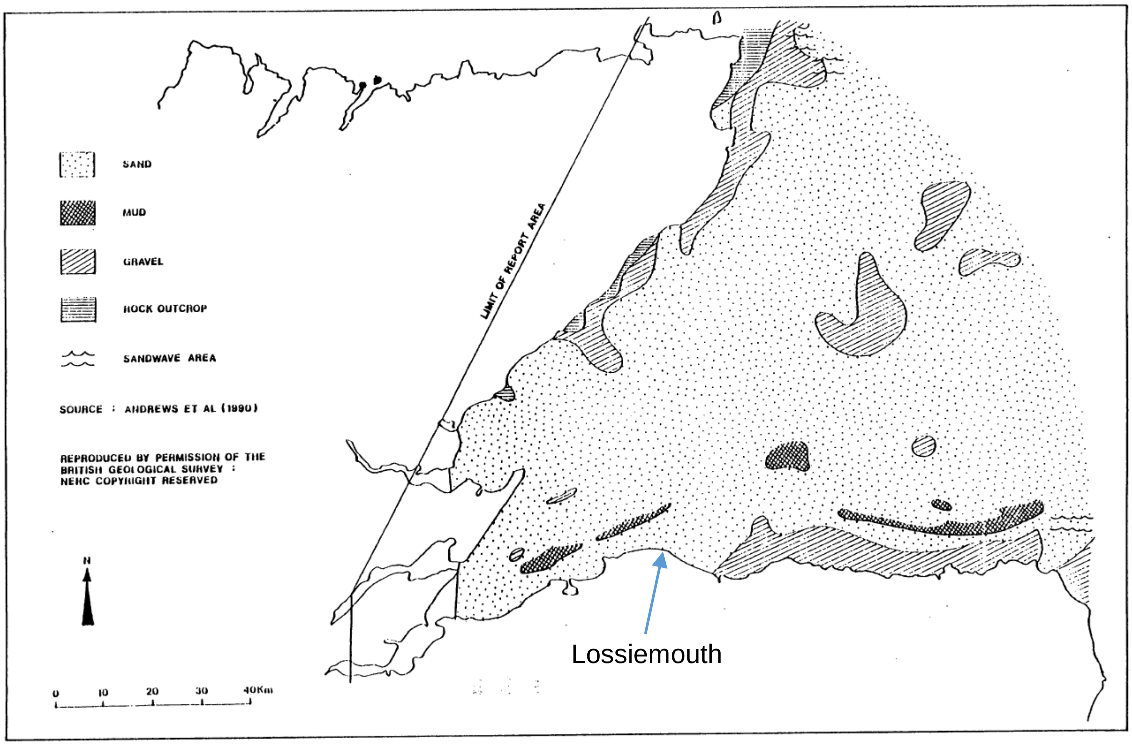


Figure 2-7: Distribution of Offshore Marine Sediments[1]

Sand is also being re-worked from the frontal dune system along the spit; despite the width of the intertidal beach, erosion of the dunes due to storm action is evident, with the over steepening of the seaward face resulting in slumping and loss of material.

With regard to the potential contribution of fluvial sediments, a study into the regional variation of fluvial sediment yield in Scotland estimates an annual sediment yield of approximately 4.44 t/km^2/yr for the River Lossie. Comparing this to the values presented for other local river catchments, e.g. the River Spey at 11.83 t/km^2/yr and the River Findhorn at 48.37 t/km^2/yr, it can be seen that estimates of sediment yield for the River Lossie are relatively low[13].

From this it can be concluded that the contribution to local sediment from the River Lossie is likely to be limited, with fluvial material dominated by that from the River Spey, which is then reworked in the coastal environment and moved westward towards Lossiemouth by the dominant sediment transport processes. The River Spey is noted to be one of the few river systems in Britain still acting as a major source of beach material[7].

Sediment transport

Based on the assessment of winds, waves and currents, the north facing Moray Firth coastline is believed to experience net westerly longshore sediment transport (LST), which has resulted in the large spit present to the east of Lossiemouth. This pathway transports sand through the littoral zone and the swash zone, pushed west by breaking waves. This pathway ends at the Old Pier at Lossiemouth, which has experienced a long-term accumulation of sediment.

Historical mapping[14] shows the spit increasing in width due to sediment accretion. In addition, sediment deposits have been identified to the west of the Old Pier within the River Lossie, narrowing the river channel[7]. This is attributed to the build-up of sand on the beach and intertidal areas, which is then blown westwards across the spit and over the Old Pier. Movement of the spit and river mouth further westward is prevented by the rocky headland, the constructed training walls and the River Lossie tidal regime. Further upstream the river channel is relatively stable, attributable to its low gradient and energy potential.

The interaction of the wave induced westerly drift and river flows from the Lossie tends to push material around the headland resulting in a feed of material to the west. As a result, there are noted to be some problems with siltation at the main harbour entrance[7].

[13] http://www.irtces.org/isi/isi_document/iahs236/iahs_236_0157.pdf

[14] http://www.oldmapsonline.org/#bbox=-3.315979,57.669933,-3.206803,57.754844&q=&datefrom=1000&dateto=2010

2.4.7 Effect of the Old Pier Breakwater

The Old Pier performs several functions, which may be effected by the future deterioration of the structure.

- It acts as an important terminal groyne, reducing the westward drift of sediment across the river mouth. The deterioration of the Old Pier currently allows coastal sediment to bypass the structure through aeolian and wave run-up processes. This redistribution of sediment has resulted in the River Lossie channel moving westward so that it now flows against the Old Harbour wall.
- It acts as a river training wall to prevent the river meandering or silting and allows the outflow of the river to remain in a concentrated northward jet. Although not specifically included, the current arrangement of the river mouth is intrinsically included in the flood modelling assessments undertaken by Royal Haskoning for the River Lossie, with the tidal section of modelling extending up as far as Elgin. If these downstream river conditions were to change, either due to the loss of hydraulic capacity due to siltation or through a new river breakout, the consequences may include changes to upstream water levels.
- It acts as a breakwater to reduce wave attack on the Seatown area of Lossiemouth. If the Old Pier was to deteriorate further there would be potential for larger waves to propagate into the river mouth, thus increasing risk from wave overtopping.

Consequently, either the continued deterioration, or the partial or complete removal of the breakwater would likely alter the transportation of sediment, encouraging additional deposition at the river mouth. Over time, increased sediment deposition would gradually reduce the mouth opening, decreasing the water depth at the mouth, and potentially also at the main harbour entrance. Coupled with this, the flood dominance would lead to an increased net sediment transport rate into the Old Harbour, reducing the storage capacity within the river mouth. These factors could result in increased fluvial flood risk upstream, and could also result in the River Lossie creating a new breakthrough location further east.

The partial or complete removal of the breakwater would also reduce protection against wave attack. Seatown and the Old Harbour promenade benefit the most from the protection offered by the breakwater, and its removal would heighten the risk of wave attack, and wave induced flooding.

2.5 Flood risk

As well as the potential effects on sediment transport, implications on flood risk arising from changes to the Old Pier are also an important consideration, and are indeed interlinked.

2.5.1 Coastal flood risk

Coastal flood risk at Lossiemouth is present from both still water levels and wave overtopping. Photographs of historical coastal flood events in Lossiemouth are provided within Figure 2-8, with the locations of the photographs shown within Figure 2-9.

The effect of the Old Pier on coastal flood risk is to reduce the wave risk to the coastline behind. During the site visit, the one section of the promenade wall where debris was observed to have been washed up by recent storm events was located directly opposite the gap in the breakwater.

1		Wave overtopping to the north of the river mouth and south of current harbour, December 2012[15]
2		Water level at the footbridge across the River Lossie to the spit, December 2012[16]
3		Overtopping of Old Harbour wall, December 2013[17]
4		Old Harbour wall, top of Old Pier visible in background, December 2013[17]

[15] http://news.stv.tv/north/206171-moray-communities-left-reeling-after-freak-tides-batter-the-coastline/

5		Undermining of the harbour wall, December 2013

Figure 2-8: Photos of Historical Coastal Flooding

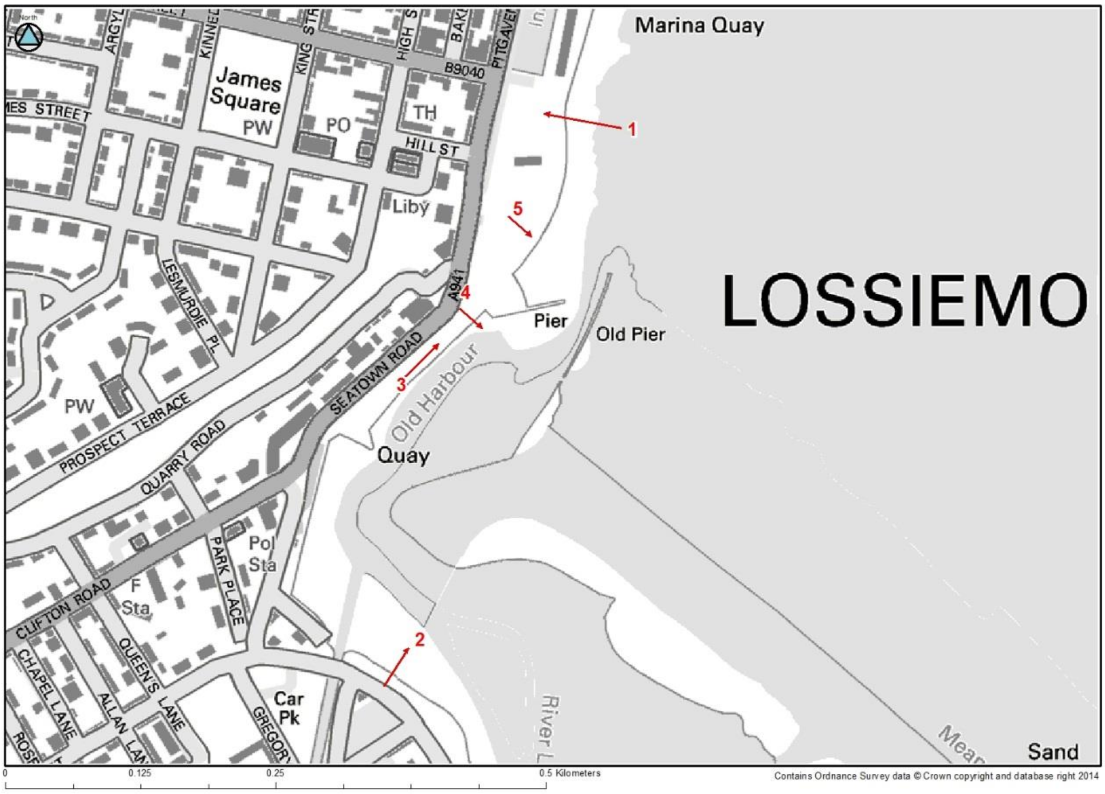


Figure 2-9: Location plan for photos of coastal flooding

The effect of the breakwater on waves, wave overtopping and the resulting flood inundation is assessed in detail within Chapter 3.

2.5.2 Fluvial flood risk

Unfortunately the existing hydraulic model of the River Lossie constructed by Royal Haskoning for Moray Council doesn't extend downstream as far as Lossiemouth, but finishes some 5km upstream near Arthur's Bridge[18]. A quantative assessment of fluvial flood risk from the River Lossie is therefore not readily available. However, Section 2.1 details how the breakwater was constructed in order to reduce sedimentation at the mouth of the Lossie, and in turn this reduction in sedimentation will aid the River Lossie in discharging freely across the beach.

[16] Image from SEPA's Flood Database

[17] Photo from Moray Council

[18] Tidal Lossie Modelling Report, Royal Haskoning, Final Report, April 2002

3 Wave assessment

A key element of considering the potential effects of changes made to the breakwater is the assessment of waves and the resulting flood risk to Lossiemouth through wave overtopping. This assessment considered four different breakwater profiles and was undertaken in three stages; wave transformation modelling, wave overtopping and flood inundation mapping.

3.1 Breakwater profiles

In order to determine the protection offered by the breakwater, four profiles of the structure have been considered. These profiles are:

1. Existing breakwater – provides a baseline of the current protection offered (Figure 3-1).
2. Breakwater repaired – determines the increase in protection offered from repair. This includes rebuilding the currently missing ~13m midsection, and the repair and reinstatement of the existing breakwater (Figure 3-1).
3. Partial removal – determines the decrease in protection due to partial loss, or removal of ~35m of the breakwater head (Figure 3-2).
4. Full removal – determines the decrease in protection due to the full removal of the structure (Figure 3-2).

It should be noted that none of the model scenarios include any alterations to the bathymetry surrounding the structure, which would likely occur due to the effect that the partial or full removal of the breakwater would have on the sediment regime.

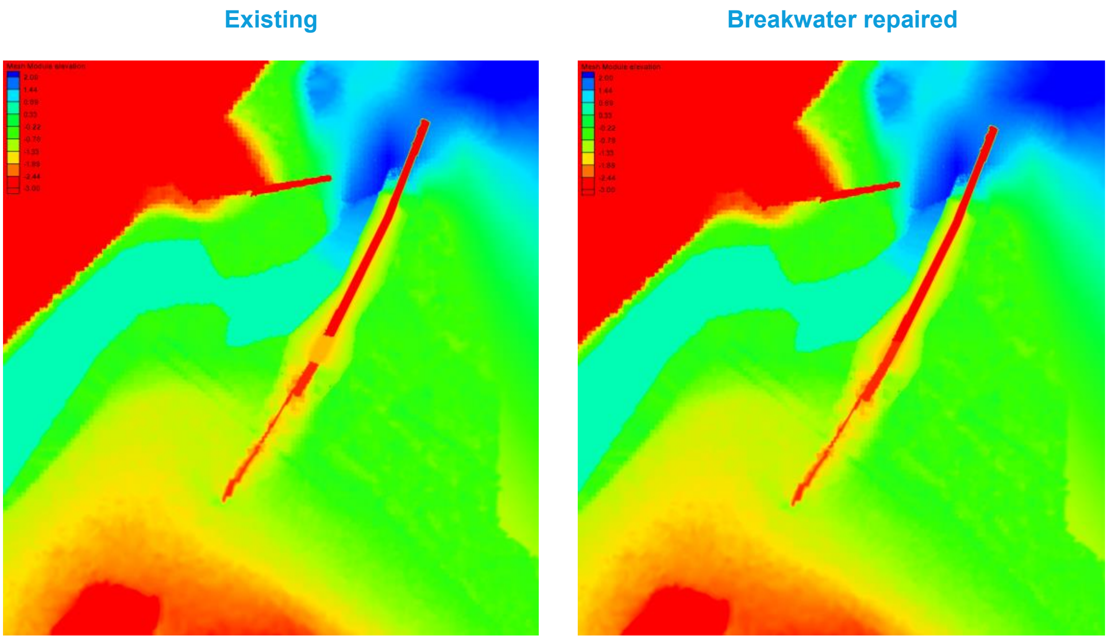


Figure 3-1: Model bathymetry for the existing and repaired breakwater profiles

Partial removal **Full removal**

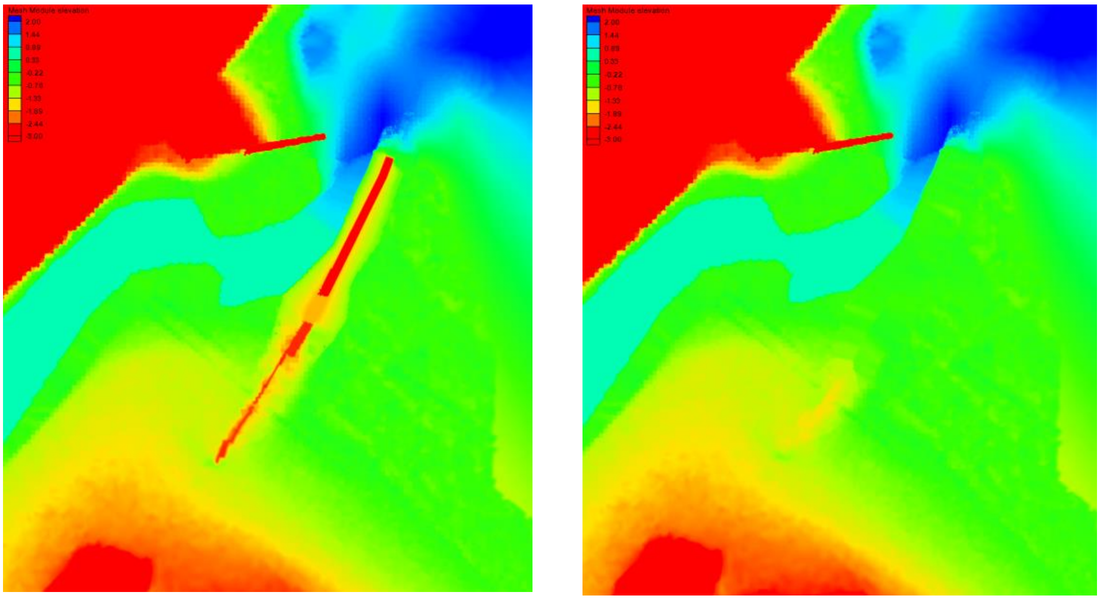


Figure 3-2: Model bathymetry for the partial and full removal breakwater profiles

3.2 Wave transformation modelling

The wave transformation model developed for the study was created using the industry-standard modelling software SWAN (Simulating WAves Nearshore). SWAN is a third generation wave model that simulates wave propagation in coastal and inland areas. SWAN can calculate steady state wave conditions for specific inputs of wave height, period and direction at an offshore boundary, and wind speed and direction applied across the model domain. Sea levels can also be set to account for tidal/surge variations.

Details of the model setup and input parameters are provided within Appendix C.

3.2.1 Wave transformation model limitations

Wave modelling has some inherent uncertainty, firstly associated with the derivation of the input parameters and secondly from the modelling process itself. Certain modelling techniques and software is not always applicable in all environments. The application of SWAN to this study is considered appropriate in order to transform deep water wave parameters from the offshore estimates into the nearshore zone. However, once in the lee of the breakwater and the confines of the river mouth, this scenario is out with the intended boundaries of the model. Despite this, the assessment undertaken herein is considered the most appropriate given the high level nature of the study. Should the project be progressed to detailed design in the future, further work to refine the modelling in this area would be required.

3.2.2 Wave transformation model results

Results from the wave transformation modelling show that the Old Pier acts so as to reduce wave heights in the mouth of the River Lossie and the Old Harbour area. Figure 3-3 shows the model results for a 1 in 200-year run with wind and wave conditions coming from the NE, with the difference in wave heights either side of the structure clearly visible.

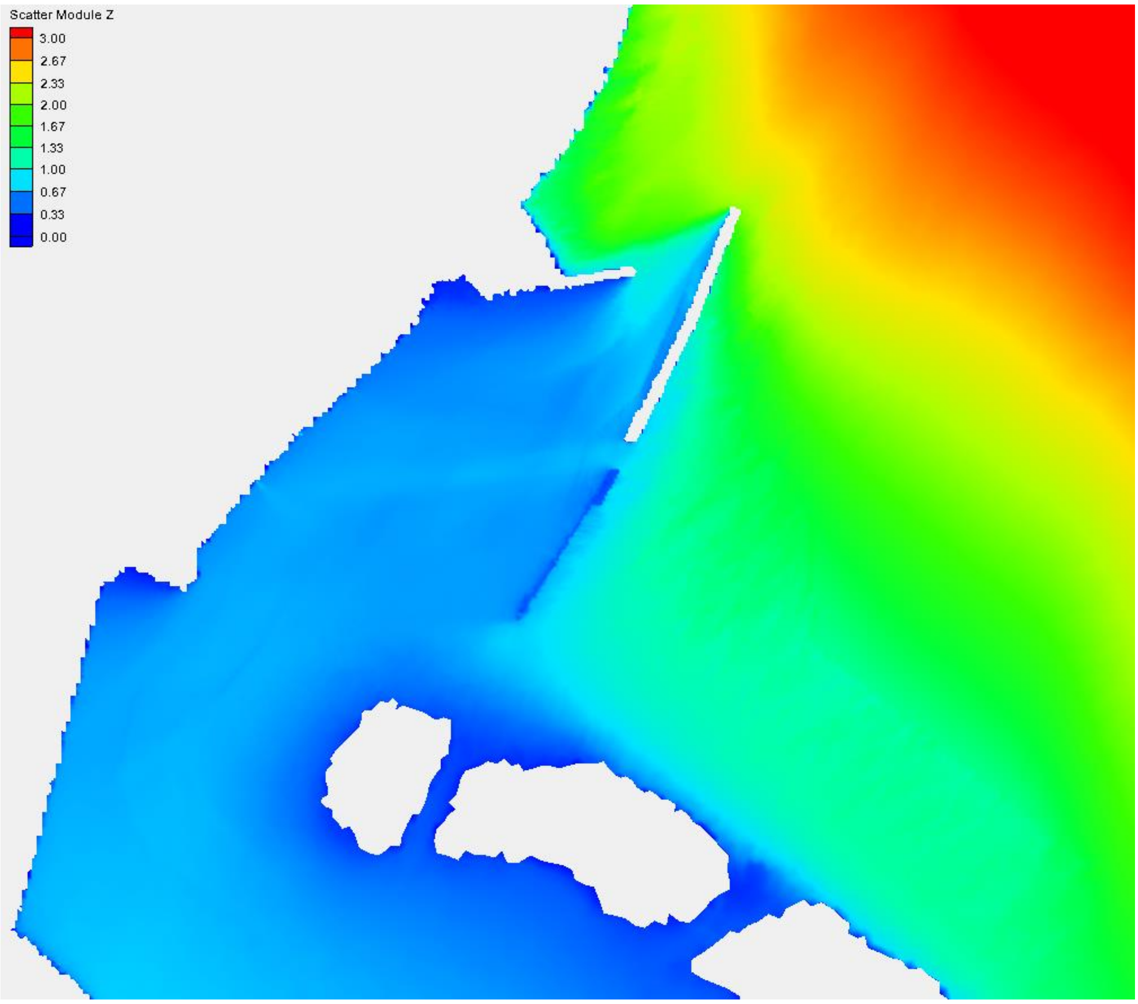


Figure 3-3: Modelled wave heights, model run 118 (200-year joint probability with wind/waves from the NE)

The modelled wave heights at specific output locations for the existing breakwater profile are presented within Table 3-1, with the difference in the modelled wave heights for the repaired, partially removed and fully removed scenarios shown within the subsequent columns; the toe locations referred to are presented graphically within Figure 3-4.

Table 3-1: Wave transformation model results for a 200-year (joint probability) event

Toe ID	Existing – Wave Height (m)	Repaired – Wave Height (m)	Partial Removal – Wave Height (m)	Full Removal – Wave Height (m)
1	0.96	-0.03	+0.01	+0.18
2	0.76	-0.01	+0.02	+0.34
3	0.94	0.00	+0.04	+0.34
4	1.41	0.00	+0.14	+0.14
5	2.07	-0.01	+0.01	+0.01
6	1.08	0.00	+0.56	+0.56
7	0.66	0.00	+0.02	+0.58
8	0.72	0.00	+0.01	+0.45
9	3.49	-0.01	+0.01	+0.01
10	0.67	0.00	0.00	0.00
11	1.75	0.00	0.00	0.00
12	2.05	-0.02	+0.01	+0.02
13	2.01	0.00	0.00	0.00
14	1.49	0.00	+0.01	+0.01
15	0.47	0.00	+0.39	+0.41
16	0.51	-0.01	+0.04	+0.35
17	0.38	-0.01	+0.12	+0.29
18	0.56	-0.02	+0.01	+0.12

19	0.54	0.00	0.00	+0.03
20	2.62	0.00	0.00	0.00
21	1.11	0.00	0.00	+0.01

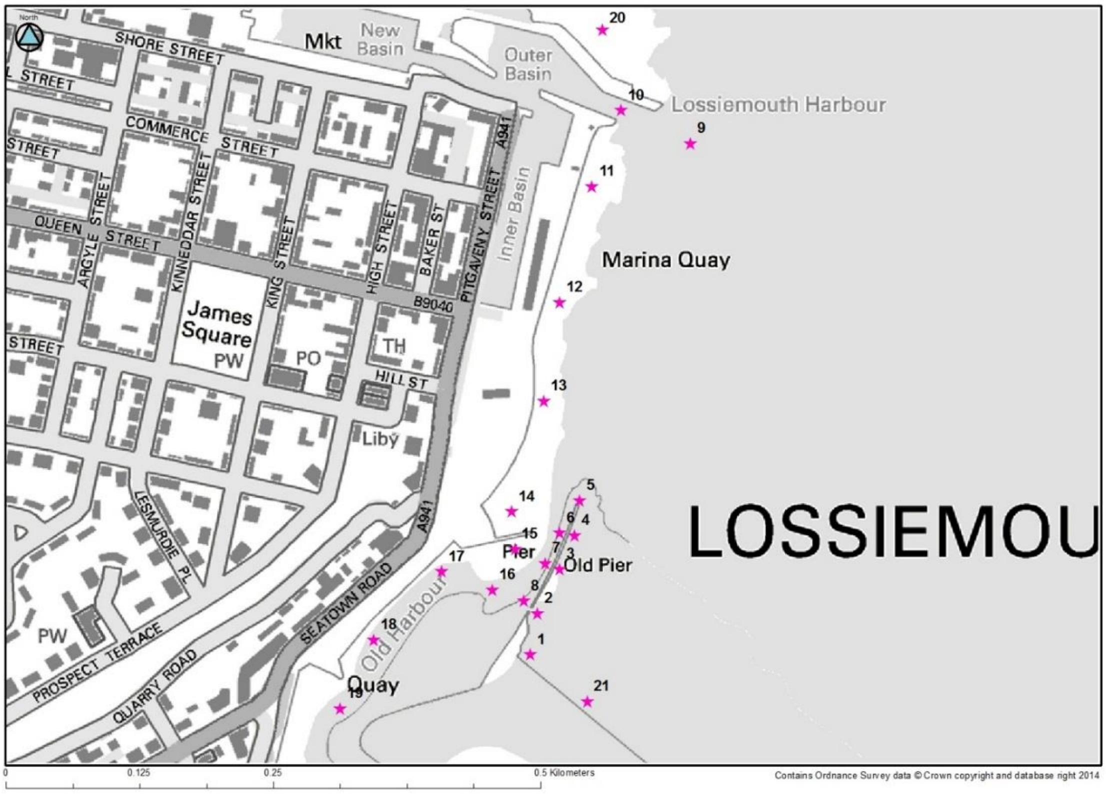


Figure 3-4: Model output locations

It can be seen from Table 3-1 that for the 200-year event, repairing the gap in the breakwater has little effect at any of the specified output locations. This effect can be seen graphically within Figure 3-5, which shows that any significant reductions in wave heights are limited to the actual location of the gap in the breakwater, with reductions in the lee of the structure reaching a maximum of value of approximately 0.10m. Plotting a range of lower return period events in this same way shows that the repair of the gap does not have an increased effect for lower return period events.

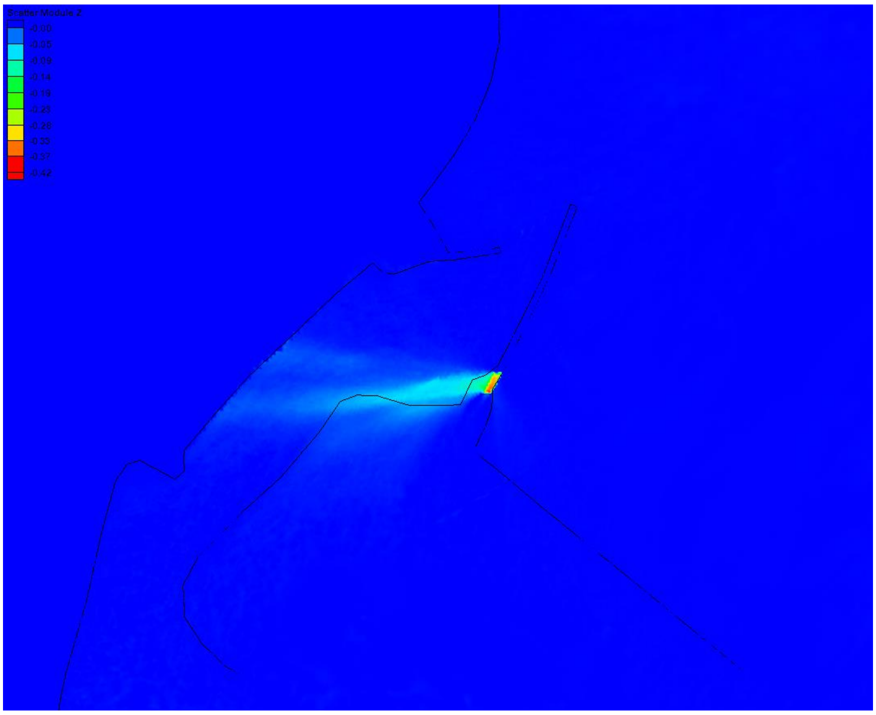


Figure 3-5: Difference in modelled wave heights, breakwater repaired vs existing (model run 118 – 200-year joint probability with wind/waves from the NE)

The options of partial or full removal are shown to have a much more significant impact on wave heights in the vicinity of the Old Pier, for example increasing the wave heights at output location 17 by 0.12 and 0.29m respectively. These results presented graphically within Figures 3-6 and 3-7. Whilst both the partial and full removal of the structure does significantly increase wave heights in the lee of the structure and the mouth of the river, it can be seen that these effects are relatively localised, and do not, for example, effect wave heights at the mouth of the harbour to the north.

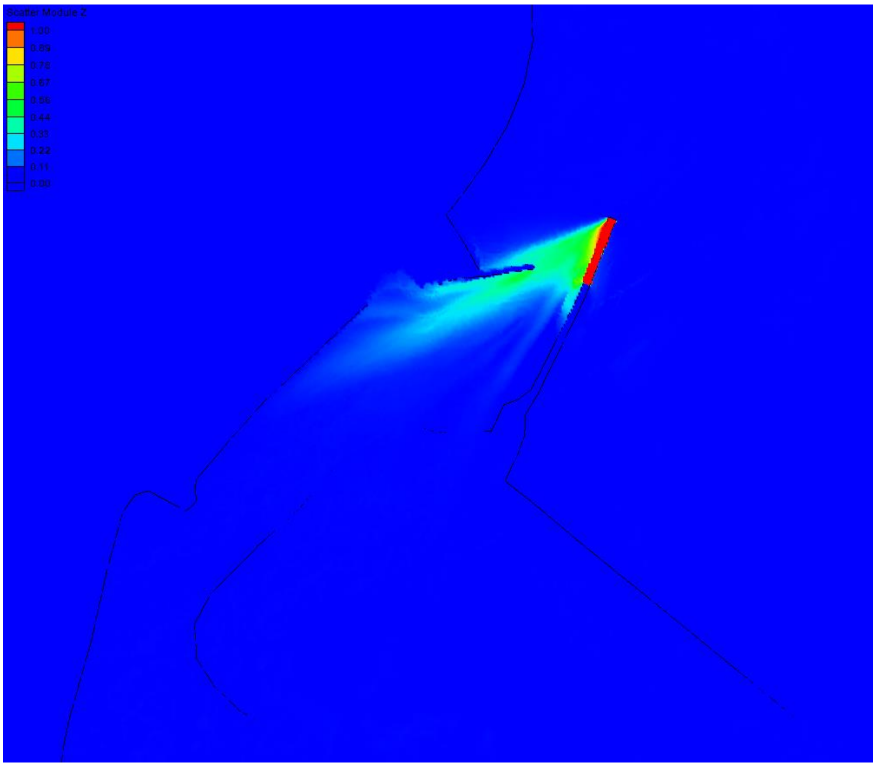


Figure 3-6: Difference in modelled wave heights, partial removal of breakwater vs existing (model run 118 – 200-year joint probability with wind/waves from the NE)

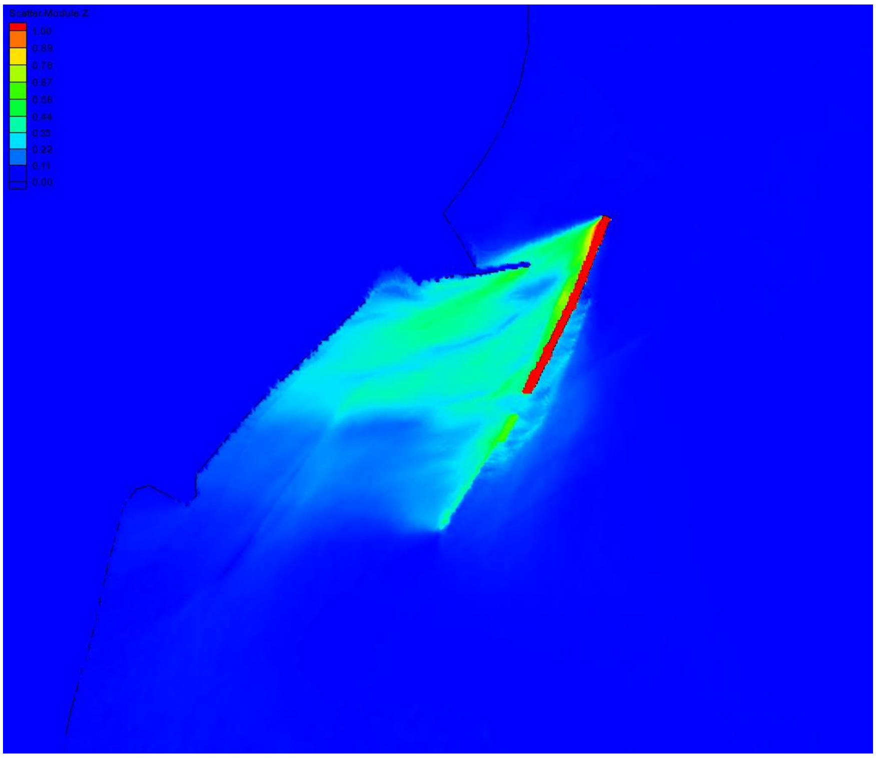


Figure 3-7: Difference in modelled wave heights, full removal of breakwater vs existing (model run 118 – 200-year joint probability with wind/waves from the NE)

Due to the potential limitations of the model within the river mouth and Old Harbour area (see Section 3.2.1), the actual values presented herein should be treated with caution. The results do however provide a good indication of the likely impacts that any changes to the breakwater would have. It should also be noted that none of the model scenarios include any alterations to the bathymetry surrounding the structure, which would be likely occur due to the effect that the partial or full removal of the breakwater would have on the sediment regime.

3.3 Wave overtopping

The results from the wave transformation modelling were fed into wave overtopping calculations. These were undertaken using the Neural Network calculation tool contained in the industry standard EurOtop. This manual is considered best practice within the UK for wave overtopping calculations. The Neural Network tool was selected as it provides the most suitable methodology to assess composite defences and allows for detailed schematisation defence profiles.

The Neural Network tool was used to calculate the rate of overtopping corresponding to the nearshore wave conditions calculated through the wave transformation model for the return periods of 1, 2, 5, 10, 25, 50, 100 and 200 years.

3.3.1 Wave overtopping model limitations

The EurOtop manual, and its associated tools such as the Neural Network, is considered to be the best practice guidance in the UK for overtopping calculations. Even so, as with all calculation approaches, the Neural Network tool has limitations. Estimates are given based on a dataset of small-scale physical model tests which are affected by model and scale effects, the accuracy of measurement equipment and wave generation techniques.

While overtopping results are displayed with a high level of precision, it is important to note that the results of the Neural Network need to be used with a degree of engineering judgement and caution.

3.3.2 Wave overtopping model results

The protective benefit of the breakwater can be observed through considering the change to wave activity within its vicinity. As identified within the wave transformation modelling, changes to the breakwater structure have a limited effect on wave heights at toe locations 11, 12, 13 and 14, located north of the breakwater. This suggests that changes to the breakwater are unlikely to affect overtopping at these locations.

Along the promenade wall, at toe location 17, both the partial removal and the full removal significantly increase wave heights by 32% and 76% respectively, and will therefore result in increased overtopping of the promenade.

Modelling of the promenade was undertaken at this location, with the results for the existing, partial removal and full removal scenarios for a range of different return period events presented within Table 3-2. These results show that for the 200-year event the modelled overtopping increases from just under 1 l/s/m with the breakwater in its current state, to approximately 3 l/m/s for the partial removal of the structure, and nearly 14 l/s/m for the full removal of the structure.

Table 3-2: Wave overtopping results for the promenade wall

Return Period (years)	Overtopping – Existing Breakwater (l/s/m)	Overtopping – Partial Removal (l/s/m)	Overtopping – Full Removal (l/s/m)
1	0.00	0.01	0.49
2	0.00	0.02	0.29
5	0.02	0.07	0.73
10	0.04	0.13	1.44
25	0.11	0.28	2.92
50	0.24	0.58	5.39
100	0.41	1.82	9.00
200	0.94	3.15	13.84

3.4 Flood inundation

The results from the wave overtopping modelling were used as inputs into a flood inundation model. This modelling was undertaken in JFlow+. JFlow+ is a 2D flood model developed by JBA which solves depth-averaged fluid flow equations to model the movement of water over a surface. The model has been developed specifically for inundation modelling, allowing it to be optimised for purposes such as coastal overtopping modelling. The model will simulate the extent of inundation based on the overtopping volume, a roughness parameter and a DTM. JFlow+ has been successfully benchmarked against other 2D model codes and has been used effectively in many previous projects.

Figure 3-8 shows the results for the 200-year scenario with the existing breakwater, the breakwater partially removed and the breakwater fully removed. It can be seen that the modelled overtopping values do not result in extensive inundation, with no properties effected.

This agrees with observations made during the site visit which suggested that, given the local topography behind the promenade wall, values overtopping the wall would have to be significant in order to reach Seatown Road the properties behind (see Figure 3-9).

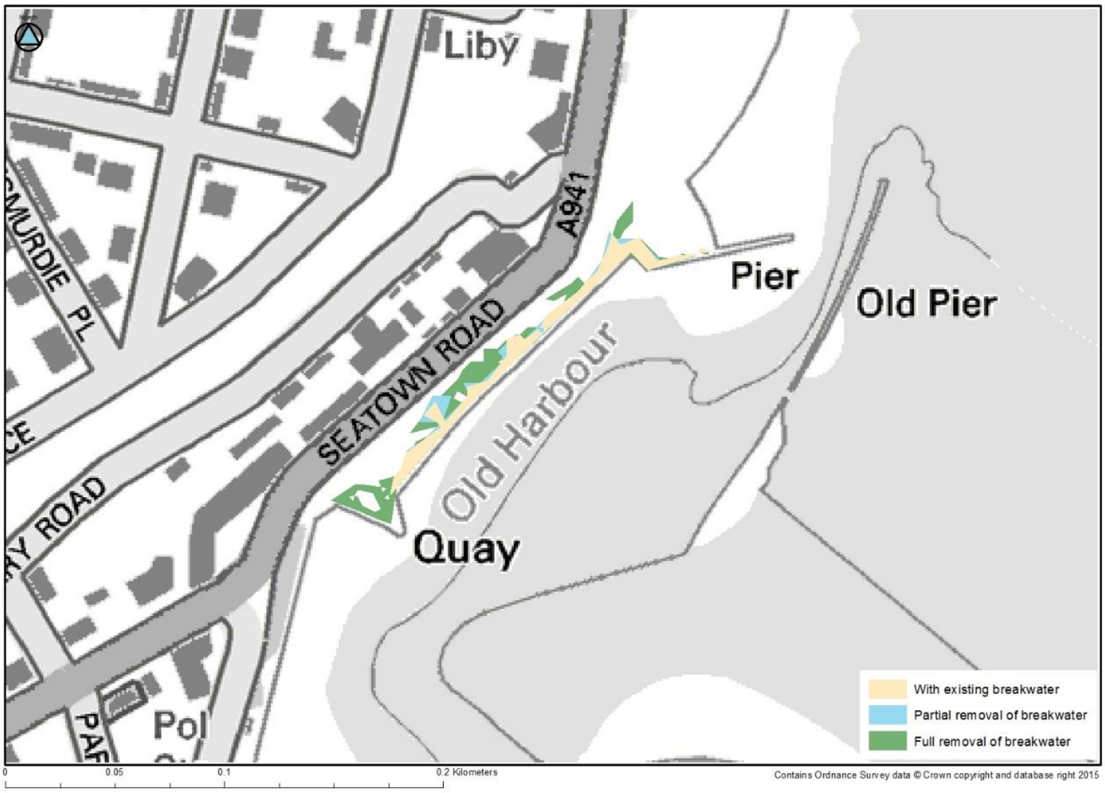


Figure 3-8: Flood inundation outlines for the 200-year event

Figure 3-9: Looking south along Seatown Road, with the promenade wall and river Lossie on the left

4 Future options

The engineering solutions available to manage the Old Pier can be categorised into three potential measures:

- Do nothing, running the risk of the partial or complete collapse of the structure
- Repair the existing structure
- Remove and replace the structure

In order to identify the best solution, it is first important to understand the control the existing structure is having on the local sediment transport regime, spit/dune preservation, and the level of defence it provides against wave attack and overtopping along the Old Harbour promenade.

4.1 Interpretation of the model results

4.1.1 Wave energy

The current breakwater, despite its condition, provides notable levels of protection against wave attack from a 1 year up to and including 200 year events. When comparing the wave activity in the river mouth, with and without the breakwater, the protection it provides is clearly evident.

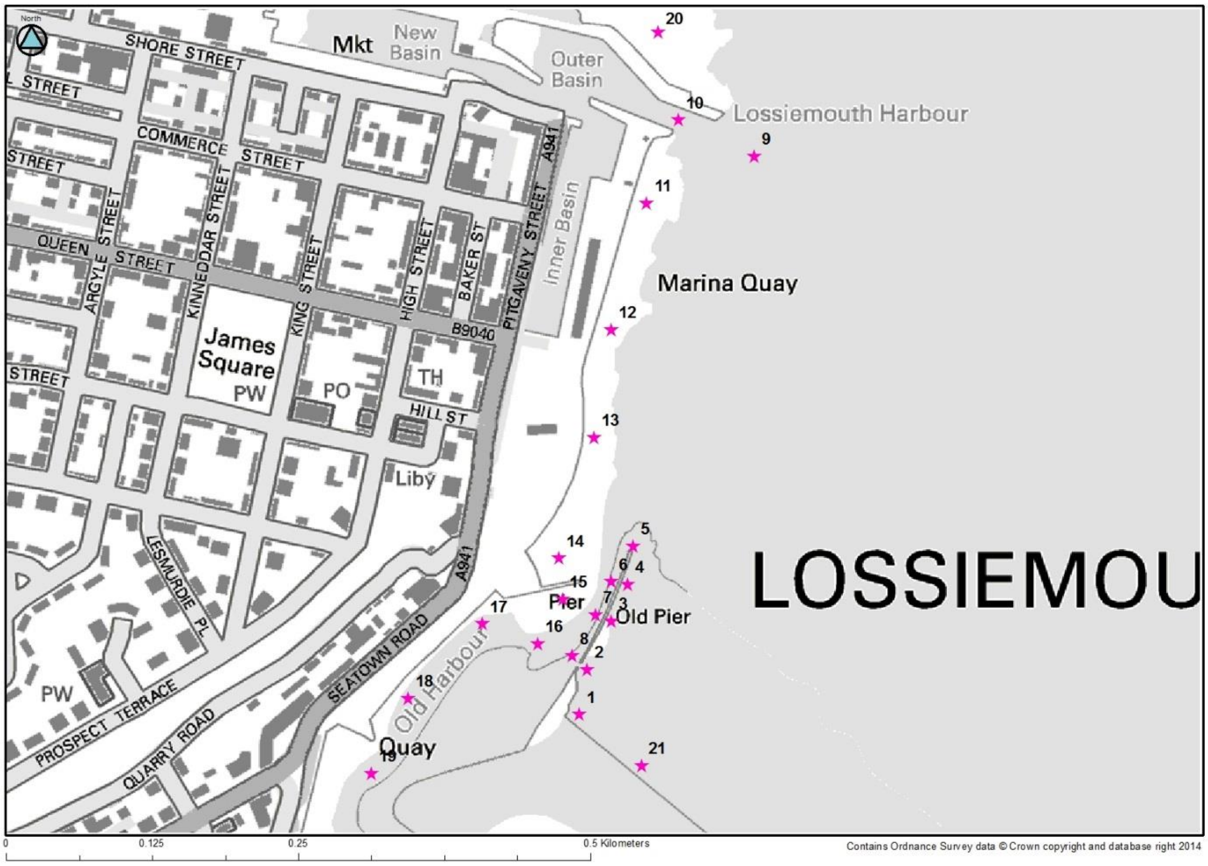


Figure 4-1: Toe locations surrounding the breakwater

The area benefiting most from the breakwaters protection stretches from the southern point of the Old Harbour (toe 19) to the Old Pier (toe 15). Given that the wave transformation model is predicting wave heights in the confines of the river mouth – which is outside its intended boundaries – the results do show wave heights in each scenario within expected orders of magnitude. However, the following presented values are only used to provide an indication of the change in wave heights, and their accuracy is entirely subject to the limitations of the model.

Modelling results suggest maximum wave heights to reach between 0.38m and 0.56m with the breakwater in place, with these increasing to 0.57m and 0.88m in its absence, an increase of 6% to 87% across toes 15 to 19. Partial removal of breakwater only marginally increases wave activity at the southern stretch of the Old Harbour wall. However, at the Old Pier (toe 15) and at the northern stretch of the Old Harbour wall (toe 17) there is a greater increase in wave height, and thus an increased risk of overtopping to the promenade.

With respect to wave energy, maintaining or replacing the breakwater in order to reduce the wave heights in the river mouth has been shown to be of most benefit in the vicinity of the Old Harbour promenade wall.

4.1.2 Overtopping

In order to identify which option is the most suitable solution with respect to wave overtopping, it is first important to understand whether the current breakwater arrangement is providing wave overtopping benefits to the inner harbour environment. The analysis presented in Section 3 shows that there is a significant increase in wave overtopping experienced at the Old Harbour promenade under the scenarios with the breakwater either partially or fully removed. However, it is important to place these overtopping values within the context of recommended guidance for safe levels of wave overtopping for pedestrians and vehicles. Table 4-1: below summarises the guidance for vehicles and pedestrians provided within the European Wave Overtopping Manual (EurOtop[19]).

Table 4-1: Limits for overtopping for vehicles (source: EurOtop[19])

Hazard type and reason	Mean Discharge q (l/s/m)	Max volume V_{max} (l/m)
Driving at low speed, overtopping by pulsating flows at low flow depths, no falling jets, vehicle not immersed.	10 - 50[20]	100 – 1,000
Driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets.	0.01 – 0.05	5 – 50 at high level or velocity
Trained staff, well shod and protected, expecting to get wet, overtopping flows at lower levels only, no falling jet, low danger of fall from walkway	1-10[21]	500 at low level
Aware pedestrian, clear view of the sea, not easily upset of frightened, able to tolerate getting wet, wider walkway	0.1	20-50 at high level or velocity

When the overtopping rates exceed these values, it can be said that it would be unsafe to engage in these activities during a storm event. The following tolerable discharge thresholds have therefore been used to assess the overtopping under the three breakwater scenarios:

- Safe: <0.1l/s/m
- Safe for aware pedestrians: 0.1 to 1l/s/m
- Safe for trained staff: 1 to 10l/s/m
- Unsafe for public and trained staff: >10l/s/m

The breakwater in its current state reduces wave energy in the mouth of the River Lossie, and provides safe access for public and staff to the promenade wall under extreme events, as presented in Table 4-2.

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[19] Pullen, T., Allsop, W., Bruce, T., Kortenhaus, A., Schuttrumpf, H & van der Meer, J (2007) 'Wave overtopping of sea defences and related structure: Assessment manual'. Accessed from www.overtopping-manual.com

[20] Note: These limits relate to overtopping defined at highways.

[21] Note: These limits relate to overtopping defined at the defence, assumes the highway is immediately behind.

Table 4-2 Overtopping volumes at the Old Harbour wall with different breakwater profiles

Return Period (years)	Overtopping – Existing Breakwater (l/s/m)	Overtopping – Partial Removal (l/s/m)	Overtopping – Full Removal (l/s/m)
1	0.003	0.011	0.490
2	0.004	0.015	0.294
5	0.020	0.066	0.730
10	0.042	0.128	1.436
25	0.108	0.284	2.920
50	0.244	0.584	5.393
100	0.408	1.817	9.001
200	0.938	3.151	13.84

Safe	Safe for aware pedestrians	Safe for trained staff	Unsafe for public and staff

The extent of protection offered by the current breakwater is expected to provide tolerable levels of overtopping for pedestrianised areas up to 200-year events, as seen in Table 4-2. Partial removal of the breakwater however, increases the predicted overtopping, and complete removal results in a further increase. The partial removal of the breakwater reduces the return period at which a 'safe' level of protection is afforded to a 10-year event, with the complete removal of the breakwater resulting in 'safe' levels of overtopping being exceeded on a yearly basis.

In this regard, while the loss of the breakwater will increase overtopping rates, the rates do still fall within the orders of magnitude of the recommended guidance for safe levels of overtopping. The structural failure, or removal, of the breakwater would unlikely have a significant impact on the risk of wave overtopping, providing the Council can accept the higher rates of wave overtopping expected at the Old Harbour Wall. The stipulation here being that the bathymetry and landform in the old harbour do not differ from those defined within the modelling process due to partial or complete loss of the breakwater – a common limitation of numerical modelling.

Alternative engineering solutions can be put in place to reduce the overtopping risk in place of the breakwater – for example raising the promenade wall – however these measures will not be explored in this report as they extend beyond the scope of this study.

4.1.3 Sediment transport

The breakwater currently acts as an important terminal groyne, reducing the westward drift of sediment. As such, partial removal of the breakwater could result in an increase of sediment transport into the river mouth, and without the breakwater this would certainly be the case.

An increase in sediment transport and deposition to the mouth of the River Lossie would over time gradually reduce the mouth opening, decreasing the water depth at the mouth, and potentially also at the main harbour entrance. Coupled with this, the flood dominant nature of the tides in the area would lead to increased net sediment transport rate into the Old Harbour, thus reducing the storage capacity within the river mouth.

Disrupting or even blocking the river flow could over time force the river to alter its course. In such a scenario it is feasible that the river will further encroach upon the spit, eroding and potentially destabilising the dune system. Moreover, if the river mouth becomes blocked and impedes flow downstream, the resulting backwater effects could result an increase of fluvial flooding, risking both properties and potentially the stability of the spit and the protection it currently offers.

The current breakwater alignment clearly has a significant control on the sediment transport regime in the area. The removal of this breakwater would increase sedimentation of the Old Harbour, leading to narrowing of the River Lossie channel, increasing fluvial flood risk and potentially affecting navigation to the harbour to the north. It is therefore suggested that allowing the structure to fail, or be completely removed will be an unviable option with respect to sediment transport.

4.2 Engineering solutions

4.2.1 Do nothing

By doing nothing, the risk of structural collapse of the breakwater arm will increase with time. The structure has been classified as poor with an estimated residual life in the order of 10-15 years in accordance with EA and DEFRA guidance on asset deterioration in the coastal environment[22].

Allowing the structure to partially deteriorate and lose the protection offered by the breakwater head, would amount to increased, yet tolerable, levels of exposure to wave overtopping on the northern stretch of the Old Harbour sea wall.

Partial deterioration could potentially incur increased sediment build-up at the river mouth and harbour entrance. Complete deterioration however is not recommended due to the adverse effects of sediment build-up. For these reasons, it is suggested that the structure should be repaired or replaced.

4.2.2 Repair

Due to the condition of the current breakwater, repairs to the concrete section is not a viable solution by itself. Instead, repair works would likely take the form of a rock armour revetment enveloping the current structure, acting as a core. The height of the revetment would be at a similar level to the current structure, with the addition of rock armour units placed on top of the current crest. The feasibility of such repair works is however subject to the condition and stability of the current breakwater.

Repair work would be beneficial for extending the defence life expectancy, and increasing the protection which it currently offers. However, due to the current location of the breakwater arm, revetment repairs would encroach on the already narrow river mouth opening.

The cost of a rock revetment repair will cost in the order of £750,000 to £1,000,000. However, these costs are dependent on a number of factors which have not been considered at this stage, and can only be thought of as a ballpark order of magnitude estimate. It is recommended that the repair options are taken to outline design stage for a better indication of the likely costs.

4.2.3 Replace

Long-term protection of the coastal environment is best served by rebuilding the breakwater. A re-design of the breakwater profile would provide:

- increased protection to Lossiemouth's sea walls
- safer navigation for vessels using the harbour to the north
- it would prevent adverse effects of sediment transport.

The most suitable breakwater designs for a rebuild would be either constructed from concrete caissons, or rock armour.

With the additional cost associated with a complete rebuild, a rock armour breakwater of similar dimensions to the revetment repair option would cost in the order of £1,500,000 to £2,000,000. However, these costs are dependent on a number of factors which have not been considered at this stage, and can only be thought of as a ballpark order of magnitude estimate. It is recommended that the repair options are taken to outline design stage for a better indication of the likely costs.

[22] Environment Agency (2013) Practical guidance on determining asset deterioration and the use of condition grade deterioration curves: Revision 1, Report - SC060078/R1, Bristol.

5 Conclusions and recommendations

JBA were commissioned by Moray Council to undertake an assessment of the 'Old Pier' breakwater in Lossiemouth. The Old Pier is a historic river training wall constructed on the right bank at the mouth of the River Lossie and also forms a terminal groyne at the tip of a large sand-topped shingle spit and beach that extends for approximately 12km to Kingston-on-Spey and beyond to Portgordon. Following a number of large storm events in recent years, including the damaging winter 2013-14 storms, the Old Pier has become degraded and concerns have been raised by local residents regarding the condition of the breakwater. This study has been undertaken to assess the influence of the Old Pier, including the benefits it offers, the consequences of either complete or partial removal of the structure, and possible repair or rebuild solutions.

From the study it can be concluded that the Old Pier performs several functions:

- It acts as an important terminal groyne, reducing the westward drift of sediment across the river mouth
- It acts as a river training wall to prevent the river meandering or silting, and allows the outflow of the river to remain in a concentrated northward jet
- It acts as a breakwater to reduce wave attack on the Seatown area of Lossiemouth

The wave modelling undertaken shows that the current breakwater, despite its condition, provides notable levels of protection against wave attack from a 1-year up to and including 200-year events. When comparing the wave activity in the river mouth, with and without the breakwater, the protection it provides is clearly evident. However, it should be noted that due to the limitations of the model within the river mouth and Old Harbour area (see Section 3.2.1), the actual wave height values presented herein should be treated with caution. The results do however provide a good indication of the likely impacts that any changes to the breakwater would have. It should be further noted that none of the model scenarios include any alterations to the bathymetry surrounding the structure, which would likely occur due to the effect that the partial or full removal of the breakwater would have on the sediment regime. The increase in wave heights at the river mouth and Old Harbour area would likely result in increased rates of wave overtopping at the Old Harbour promenade. However, whilst the modelling shows that these rates would increase, the rates would still fall within orders of magnitude of the recommended guidance for safe levels of overtopping. This is further reinforced by the flood inundation modelling which shows that the modelled overtopping rates, even with the full removal of the breakwater would not result in flooding to any properties. However, further more detailed modelling would be necessary at the concept design stage to confirm this.

The Old Pier is deemed to play a more important role with regard to sediment transport. The current deterioration of the structure is allowing coastal sediment to bypass the breakwater through aeolian and wave runup processes. This redistribution of sediment has resulted in the River Lossie channel moving westward so that it now flows against the Old Harbour wall. The further deterioration or removal of the structure would allow increased sedimentation at the river mouth and potentially also at the main harbour entrance to the north. Coupled with this, the flood dominant nature of the tides would lead to an increase in net sediment transport rate into the Old Harbour area. These factors could result in an increase in fluvial flood risk upstream or may force the River Lossie to breakout through the spit elsewhere.

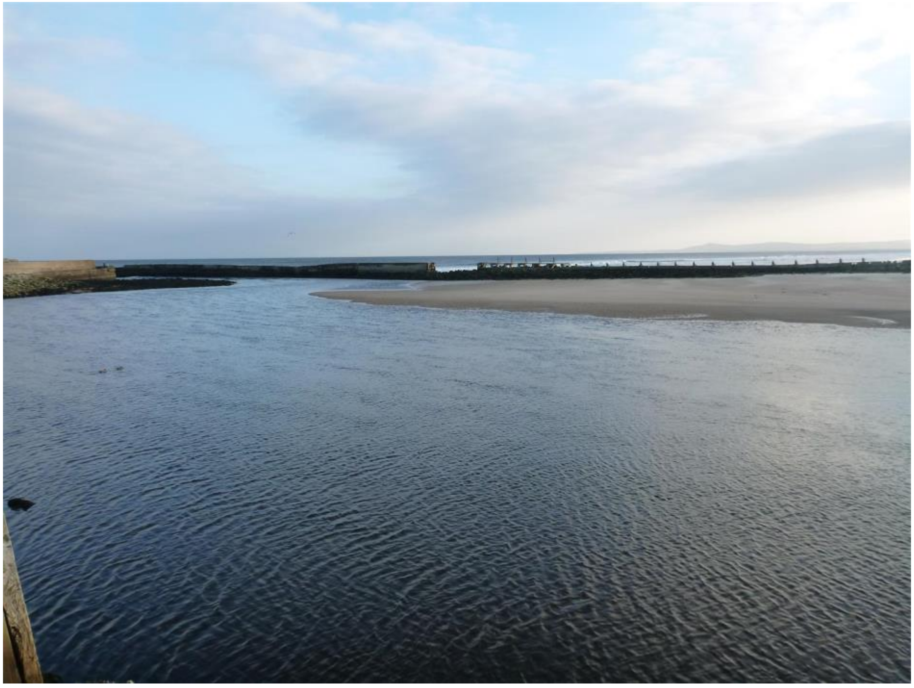
The engineering solutions available to manage the Old Pier can be categorised into three potential measures; (1) do nothing, running the risk of partial or complete collapse of the structure, (2) repair the existing structure, (3) remove and replace the structure.

By doing nothing, the risk of structural collapse of the breakwater arm will increase with time. The structure has been deemed to be in a poor condition, and has been classified to have an estimated residual life in the order of 10-15 years. Allowing the structure to deteriorate, either partially or fully, would amount to increased, yet tolerable, levels of exposure to wave overtopping on the northern stretch of the Old Harbour sea wall, as well as increased sediment build up at the river mouth and potentially the harbour entrance to the north. Due to the potential changes to the sediment regime and the implications of this, it is suggested that the structure should not be left to deteriorate but should be either repaired or replaced, with both of these options taken forward to concept design.

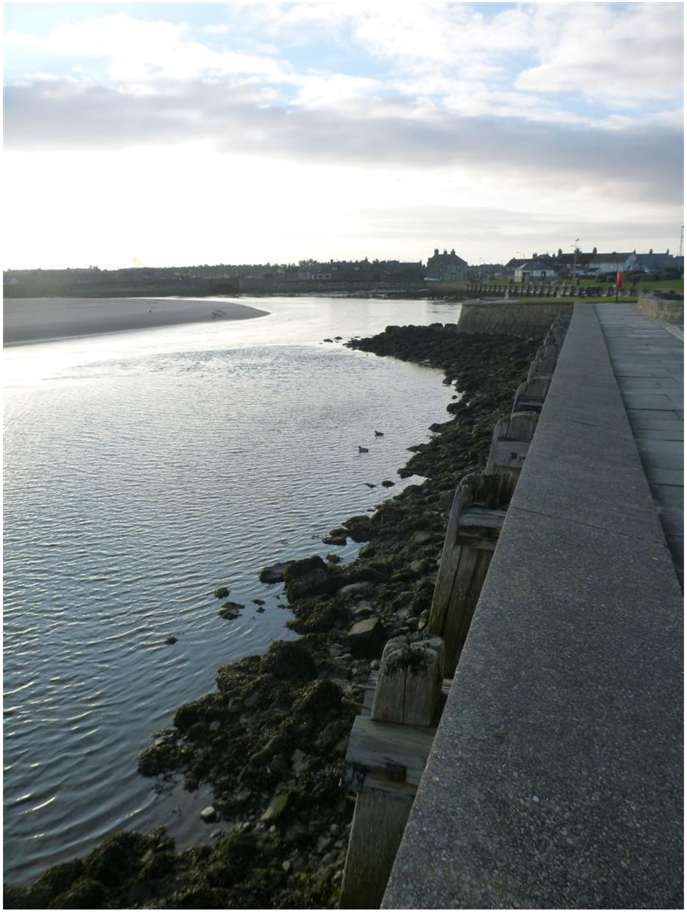
Appendices

A Photographs

Photograph 1: Overview of the mouth of the River Lossie, Old Pier breakwater, left bank pier and Old Harbour (Promenade) wall.



Photograph 2: Old Pier Breakwater from the Old Harbour (Promenade) Wall – Wooden section to right, gap in centre, concrete section to left.



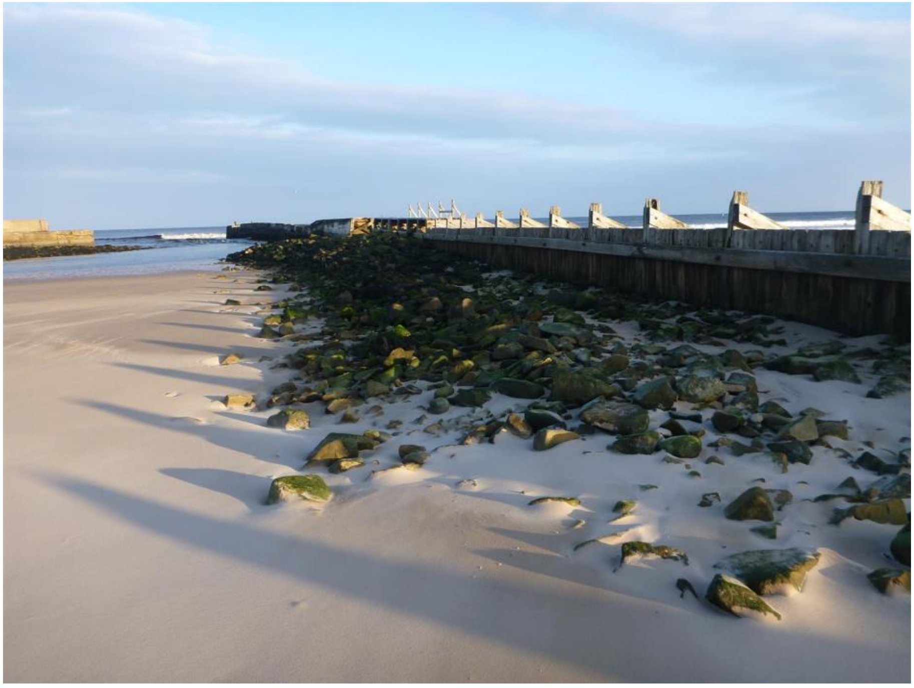
Photograph 3: Looking south along the Old Harbour wall

Photograph 4: Looking north along the Old Harbour wall

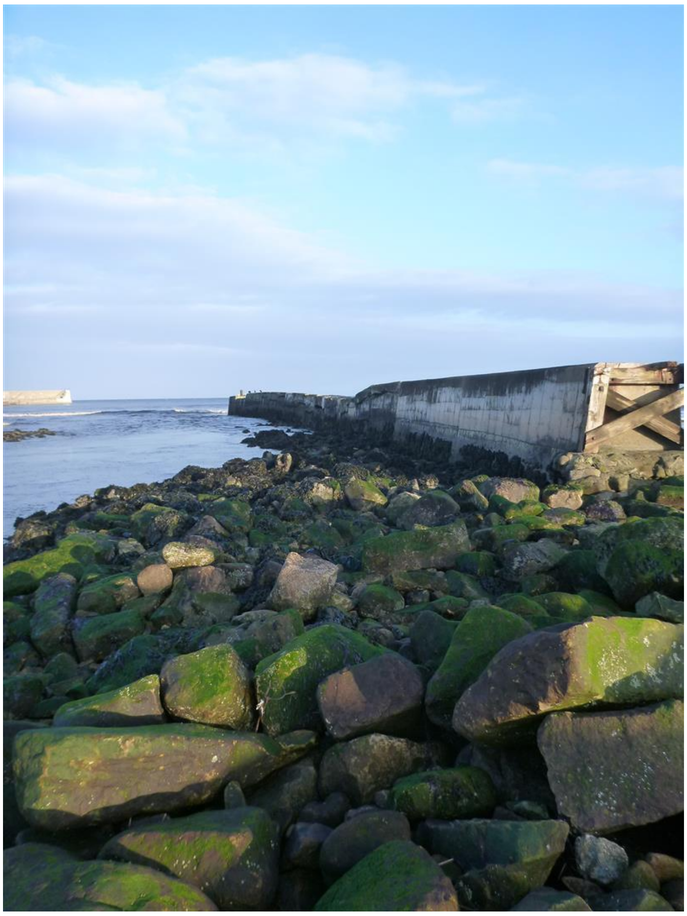
Photograph 5: Footbridge over the River Lossie connecting the spit to the town

Photograph 6: Landward extent of the breakwater from the Old Harbour

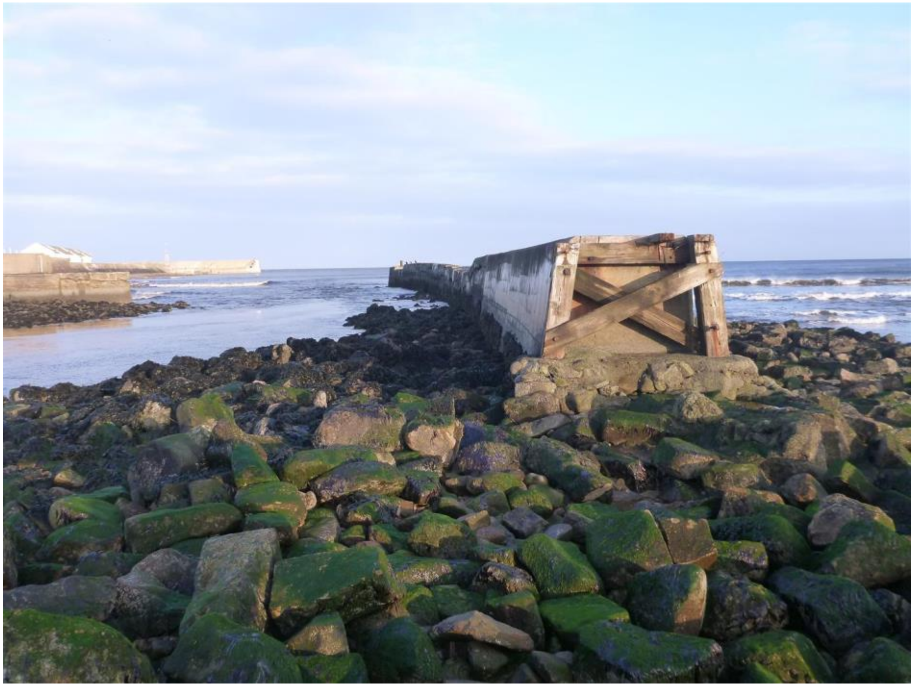
Photograph 7: Landward extent of the breakwater from the spit



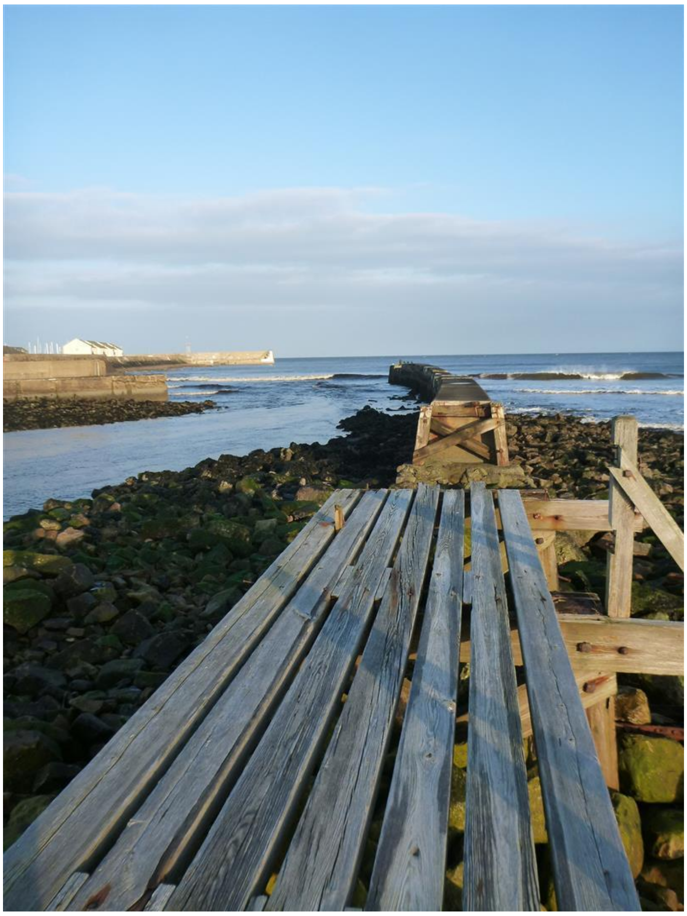
Photograph 8: Lee-ward side of the wooden section of the breakwater



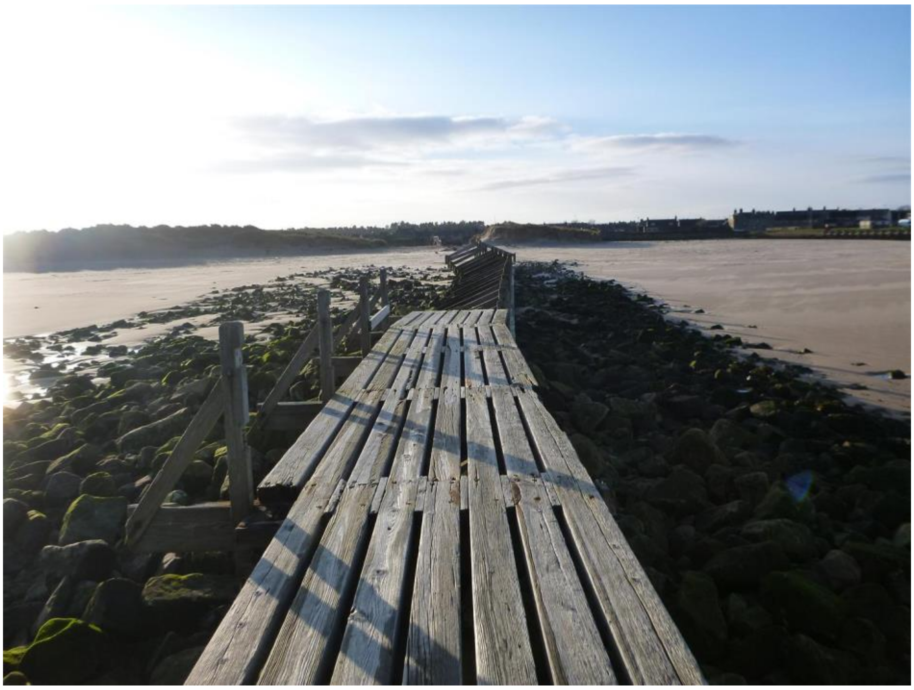
Photograph 9: Leeward side of the concrete section of the breakwater



Photograph 10: End of the concrete section of the breakwater



Photograph 11: Looking down the length of the concrete section of the breakwater



Photograph 12: Looking landward along the length of the wooden section of the breakwater

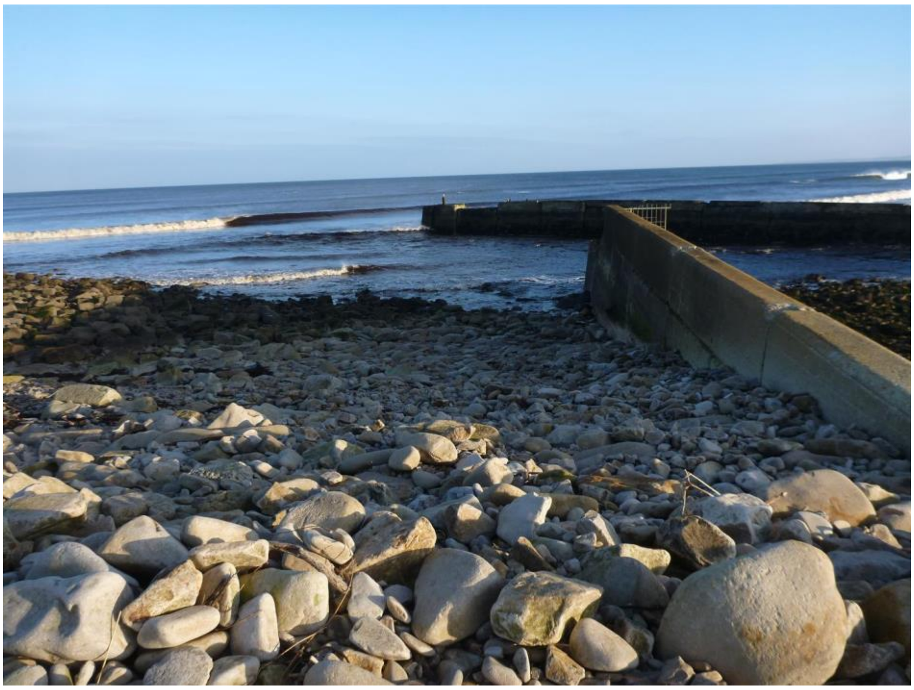
Photograph 13: Looking along the seaward side of the breakwater

Photograph 14: Erosion of the dune system on the spit

Photograph 15: Debris at the northern extent of the Old Harbour wall

Photograph 16: Looing across the mouth of the River Lossie to the end of the breakwater from the left bank pier

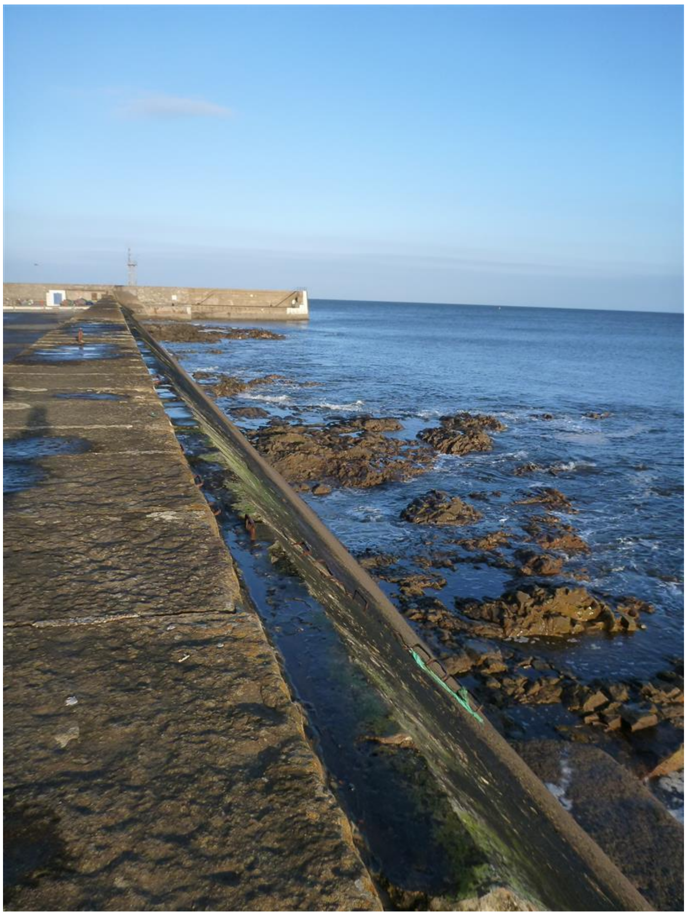
Photograph 17: Looking along the length of the left bank pier towards the Old Harbour



Photograph 18: Mouth of the River Lossie, showing the Old Pier breakwater and left bank pier in the foreground

Photograph 19: Looking north from the left bank pier

Photograph 20: Looking south back towards the river mouth and Old Pier



Photograph 21: Looking north towards the harbour entrance

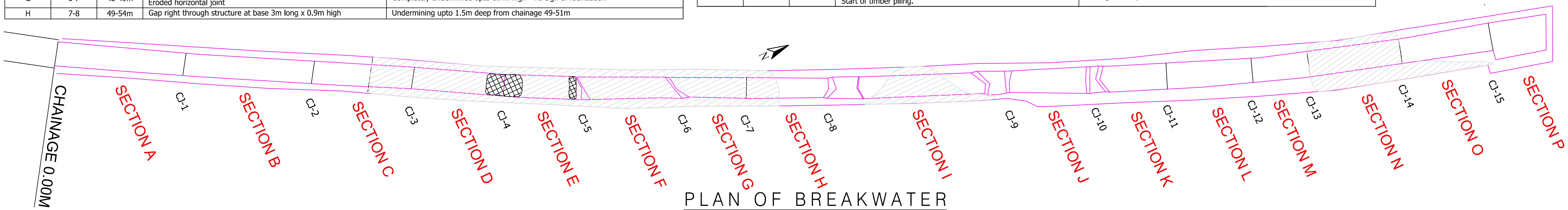
B Moray Council structural report

TABLE OF DEFECTS

(Taken from Arch Henderson report of 2010)

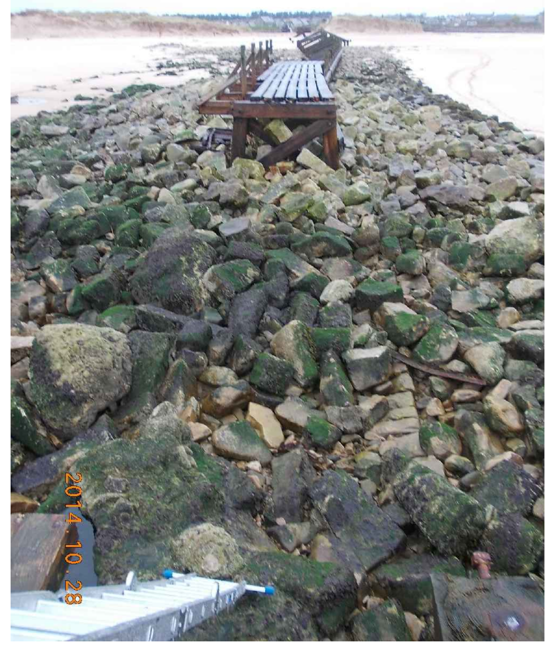
SECTION	JOINTS	CHAINAGE	OUTSIDE FACE	INSIDE FACE
A	Start-1	0-9m	Existing insitu repair	Existing insitu repair
B	1-2	9-18m	Existing insitu repair	Existing insitu repair
C	2-3	18-25m	Horizontal joint under slab open and eroded. Structure undermined right through from chainage 22	Horizontal joint under slab open and eroded.
D	3-4	25-32m	Horizontal joint under slab open and eroded. Undermined over length of section	CJ-3 open right through structure at top Horizontal joint under slab badly eroded. Undermining over 5m length - right through structure at parts
E	4-5	32-37m	2.5x0.8m hole right through structure under slab at CJ-4 Undermining continues	Horizontal joint under slab badly eroded. Undermining continues 2.5mx0.6m Hole 0.8m under deck right through wall at CJ-4
F	5-6	37-43m	0.5x0.4m hole right through structure under slab at CJ-5	0.5x0.3m hole right through structure under slab at CJ-5 Undermining continues
G	6-7	43-49m	Start of bagwork berm at chainage 43 Eroded horizontal joint	Completely undermined upto 0.7m high - no sign of foundation
H	7-8	49-54m	Gap right through structure at base 3m long x 0.9m high	Undermining upto 1.5m deep from chainage 49-51m

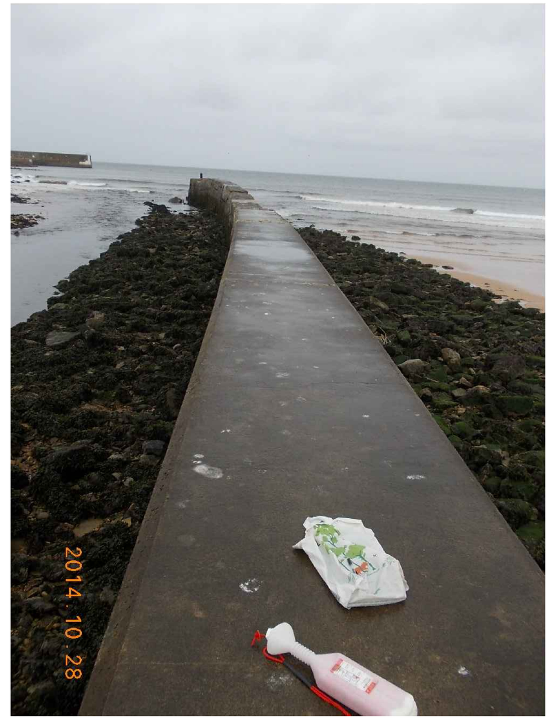
SECTION	JOINTS	CHAINAGE	OUTSIDE FACE	INSIDE FACE
I	8-9	54-67m	Bagwork berm undermined right through for 5m At chainage 58 diagonal crack full height of wall. At CJ-8 2m long x 200mm thk section of top slab missing At CJ-9 1m long x 1m deep section missing from top of wall	Top slab damage as indicated on outside face Continuous undermining - big gaps evident though outside base of wall still in place. At chainage 64 shaped stone foundation begins - some missing at CJ-9
J	9-10	67-72m	Horizontal joint eroded.	Horizontal joint eroded.
K	10-11	72-78m	Top of berm over 4m length missing - cavity behind. (The loss is recent for top 0.4m depth)	Small voids at CJ-10 Full height crack next to CJ-11
L	11-12	78-84m		
M	12-13	84-88m		
N	13-14	88-95m	End of berm. Wall undermined over full length of section	Small section of undermining 1m long x 0.4m high 0.9m deep 2.5m section shaped stones dislodged
O	14-15	95-100m	Section undermined up to 2m deep	
P	15-End	100-104m	CJ-15 open right through. Start of timber piling.	Missing timber pile at corner - undermined 0.3m.



PLAN OF BREAKWATER

(Taken from Arch Henderson report of 2010)


Start of breakwater towards land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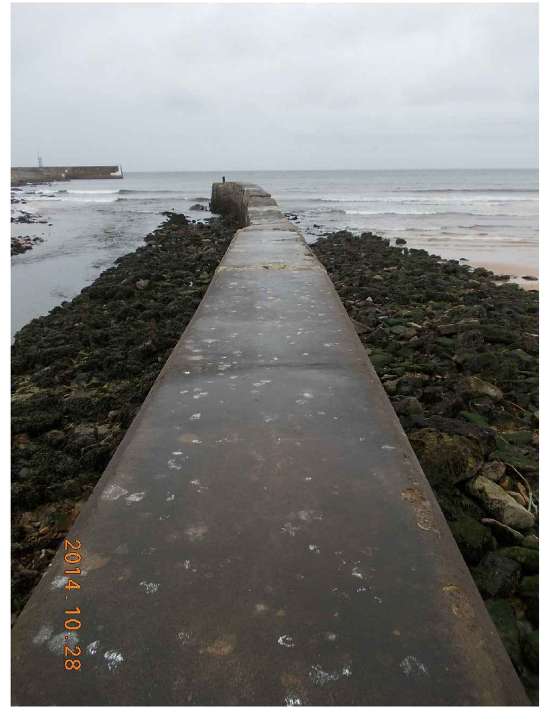
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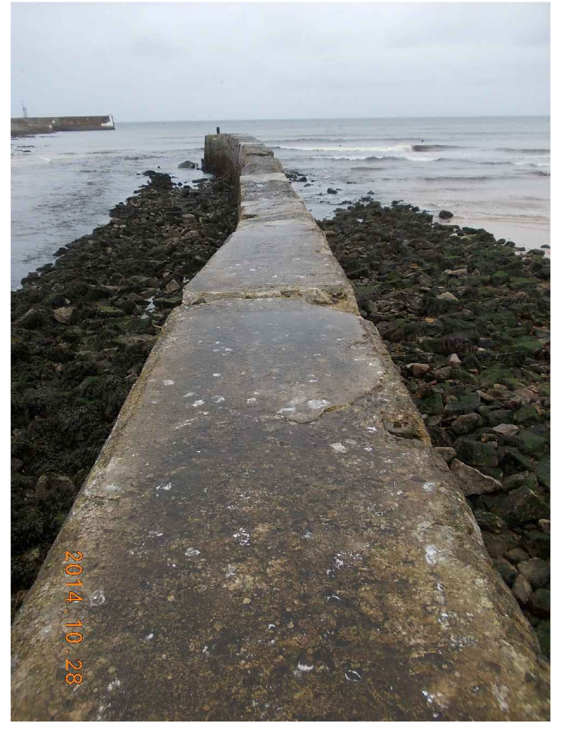
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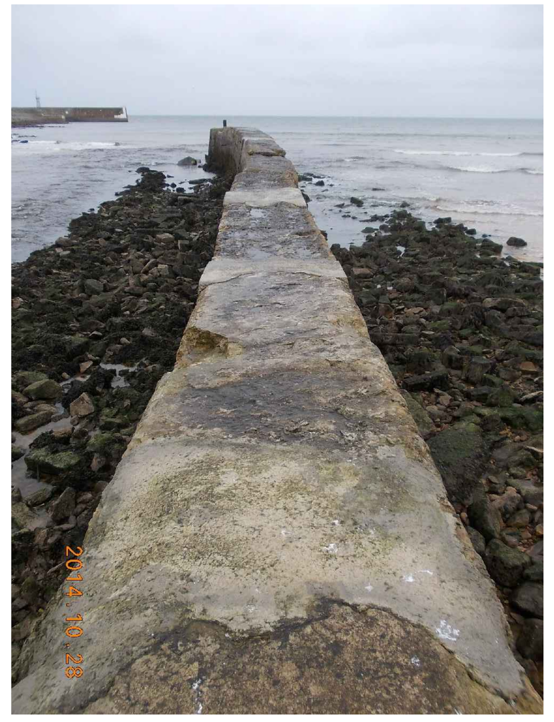
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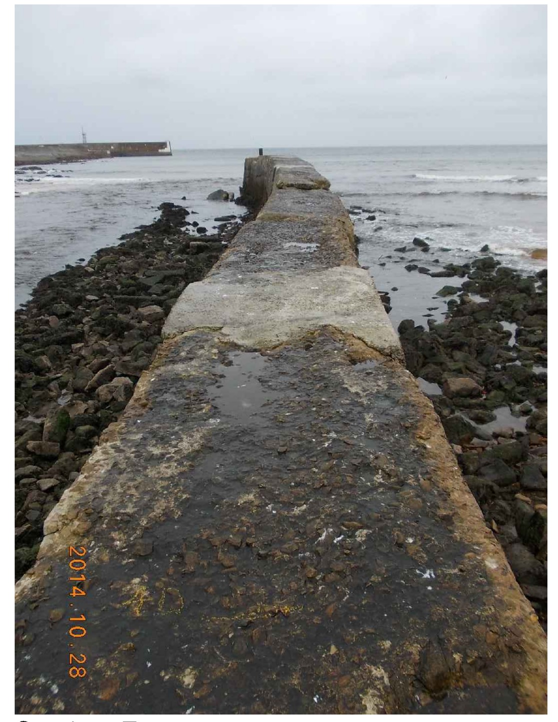
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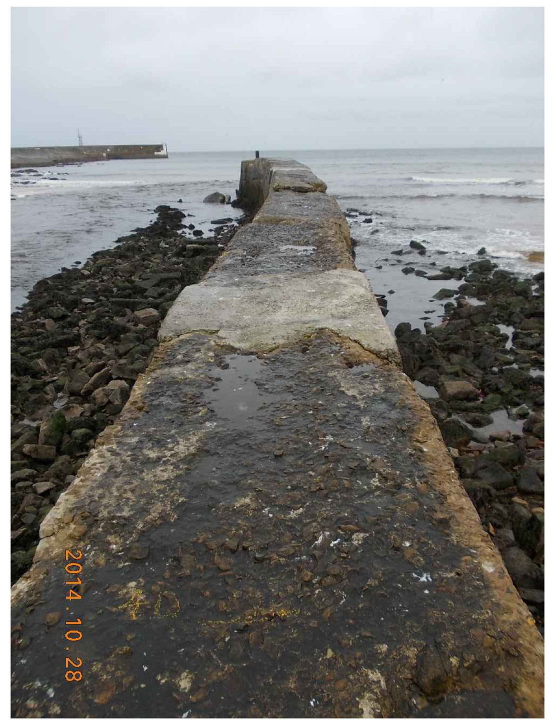
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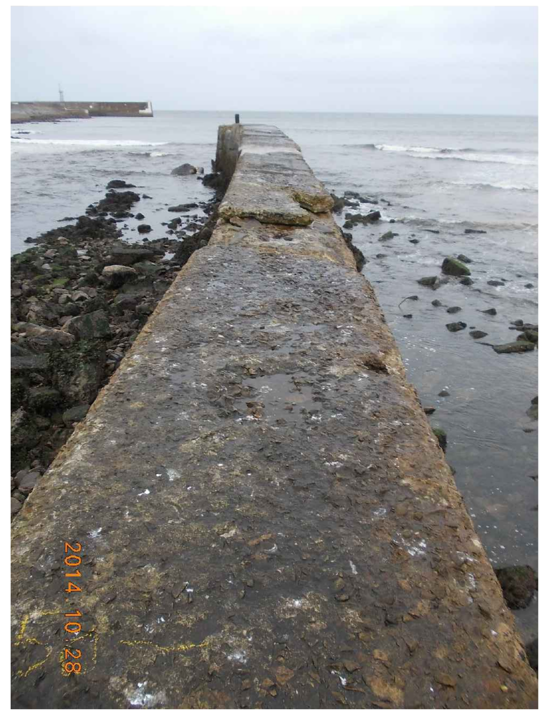
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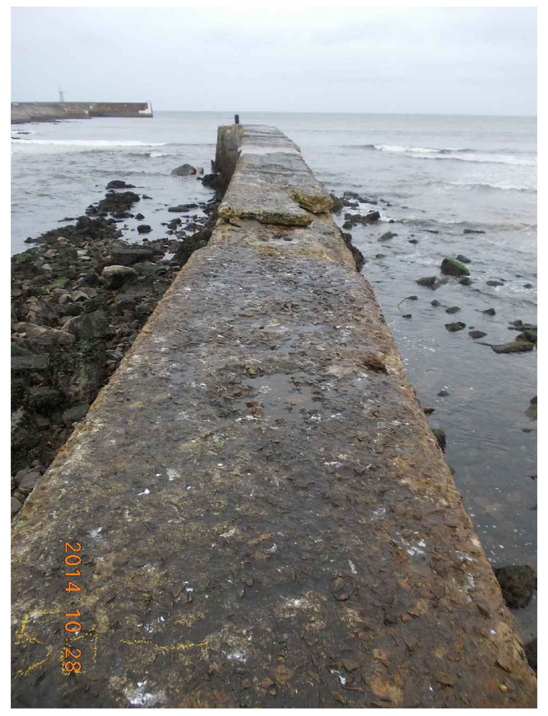
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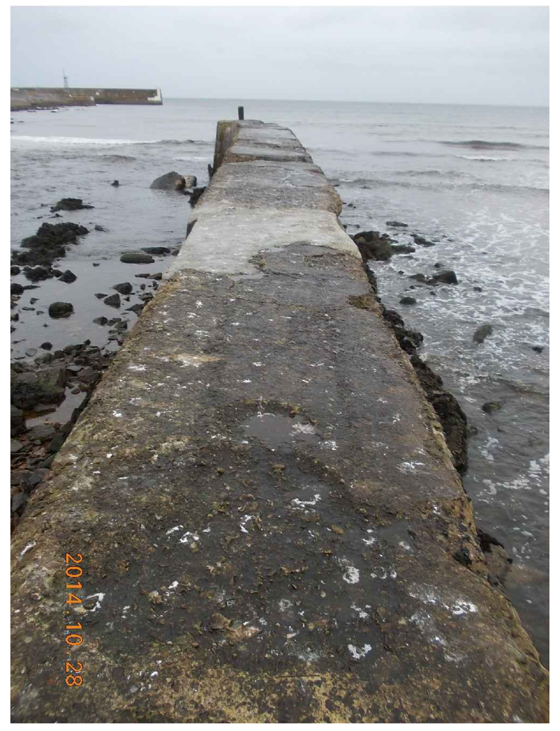
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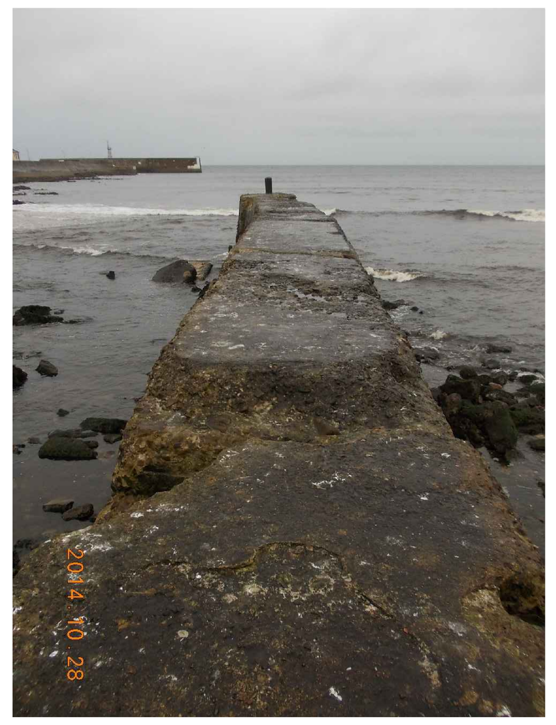
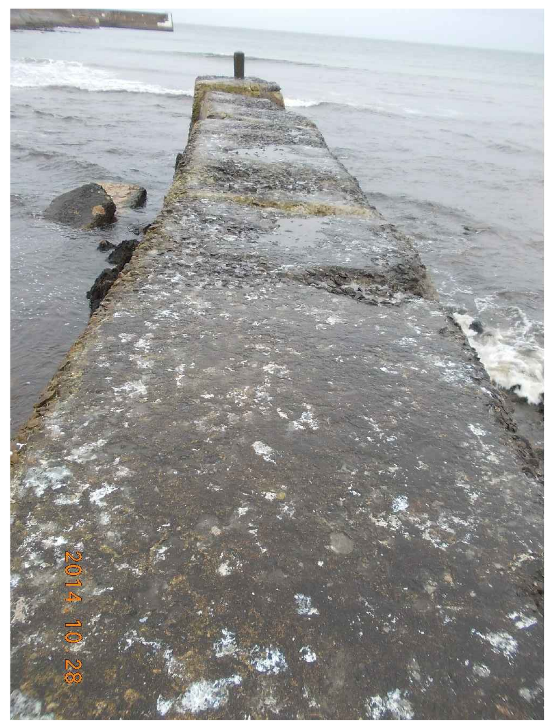

Section K


Section L


Section M


Section N


Section O


Section P


Section Q

LOOKING ALONG TOP OF BREAKWATER FROM LAND END

Certificate GB13/89299

OS GRID REF:

NOTES

GENERAL

1. All dimensions are in millimetres unless noted otherwise.
2. All levels are in metres relative to Ordnance Datum Newlyn unless noted otherwise.
3. All co-ordinates are to Ordnance Survey data unless noted otherwise.
4. All structural steelwork is to be CE marked in accordance with BS EN 1090-1: 2009.

Rev.	Amendment Details	By	Ch'd	Date
–	–	–	–	DD/MM/YYYY

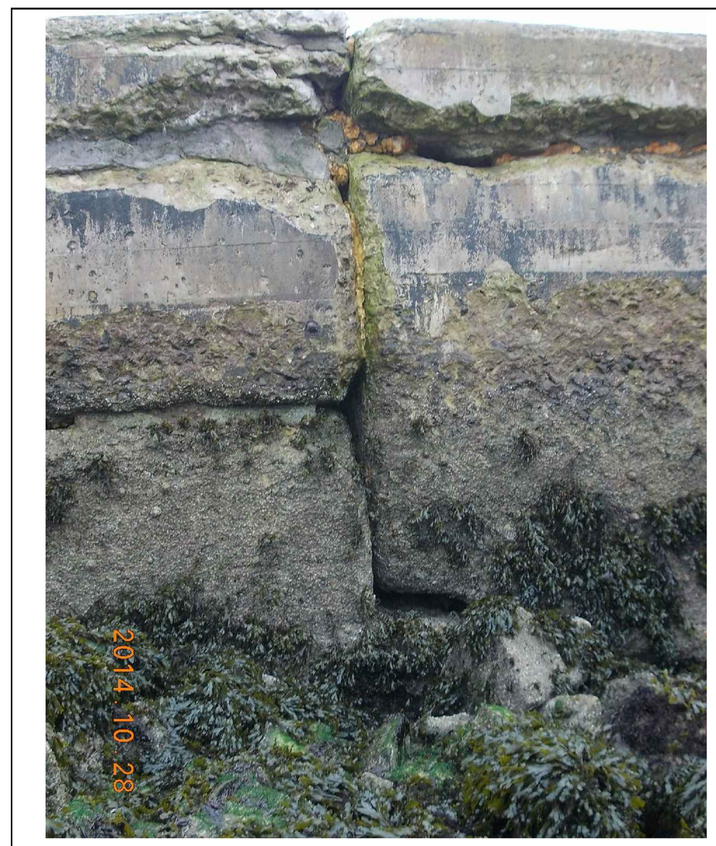
THE MORAY COUNCIL
DIRECT SERVICES - CONSULTANCY
PO BOX 6760
ELGIN IV30 9BX
TEL: 01343 543451

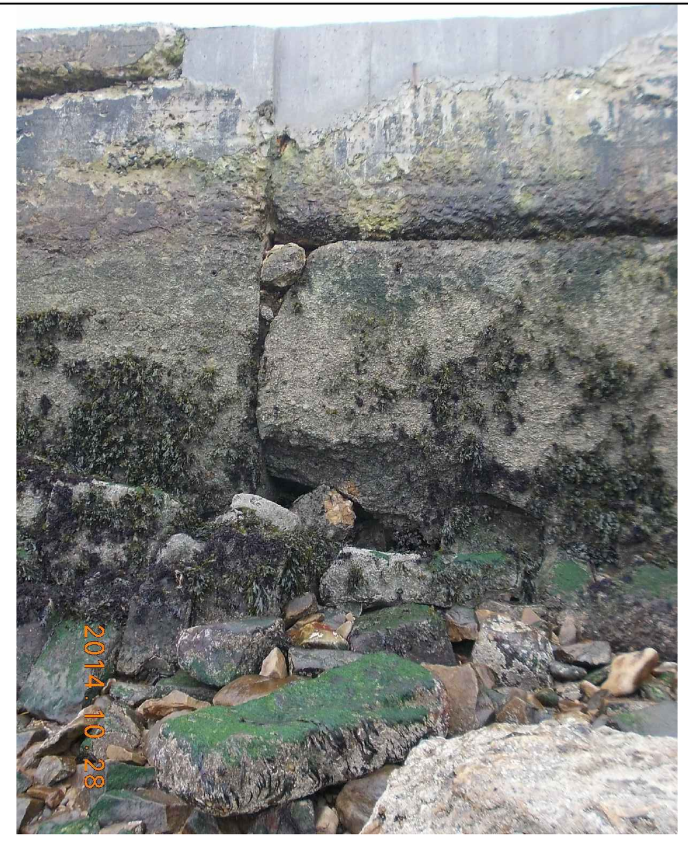
LOSSIEMOUTH BREAKWATER

CONDITION REPORT

OCTOBER 2014 PHOTOGRAPHS
SHEET 1 OF 2

Drawn: PCE	Checked: ANF	Issued for: INFORMATION	
Scales at A1: NOT TO SCALE		Date: NOVEMBER 2014	
Drawing Number: 001			Revision –



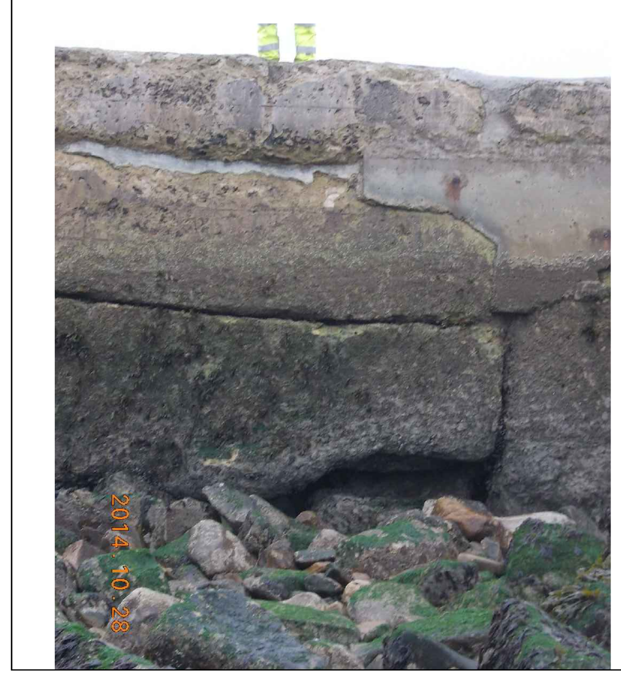


CLOSE-UPS OF SOME OF THE DAMAGED AREAS
Looking towards the east face of the breakwater from the harbour side

Seaward end

Landward end

LOOKING TOWARDS THE EAST FACE OF THE BREAKWATER FROM THE HARBOUR SIDE



Seaward end

LOOKING TOWARDS THE WEST FACE OF THE BREAKWATER FROM EAST BEACH SIDE

Certificate GB13/89299

-	-	-	DD/MM/YYYY
Amendment Details	By	Chk'd	Date

THE MORAY COUNCIL
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LOSSIEMOUTH BREAKWATER

CONDITION REPORT

OCTOBER 2014 PHOTOGRAPHS
SHEET 2 OF 2

PCE	Checked: ANF	Issued for:	INFORMATION
Scales at A1:	NOT TO SCALE	Date:	NOVEMBER 2014
Drawing Number:	002	Revision	–

C Wave assessment

C.1 Wave transformation modelling

The wave transformation model developed for the study was created using the industry-standard modelling software SWAN (Simulating WAves Nearshore). SWAN is a third generation wave model that simulates wave propagation in coastal and inland areas. SWAN can calculate steady state wave conditions for specific inputs of wave height, period and direction at an offshore boundary, and wind speed and direction applied across the model domain. Sea levels can also be set to account for tidal/surge variations.

Wave modelling has some inherent uncertainty, firstly associated with the derivation of the input parameters and secondly from the modelling process itself. Certain modelling techniques and software is not always applicable in all environments. The application of SWAN to this study is considered appropriate in order to transform deep water wave parameters from the offshore estimates into the nearshore zone. However, once in the lee of the breakwater and the confines of the river mouth, this scenario is out with the intended boundaries of the model. Despite this, the assessment undertaken herein is considered the most appropriate given the high level nature of the study. Should the project be progressed to detailed design in the future, further work to refine the modelling in this area would be required.

The spatial domain of the model is shown within Figure C-1. The SWAN model was constructed using a computational grid with a varying spatial resolution, thus allowing for a large spacing between grid points in the offshore region where a detailed representation of the deep bathymetry is unnecessary, and more detailed information in the nearshore region. The model resolution at the offshore boundary is 1km, with this decreasing at the coastline to 0.5m in the vicinity of the breakwater and the mouth of the River Lossie, and 20m elsewhere along the coastline.

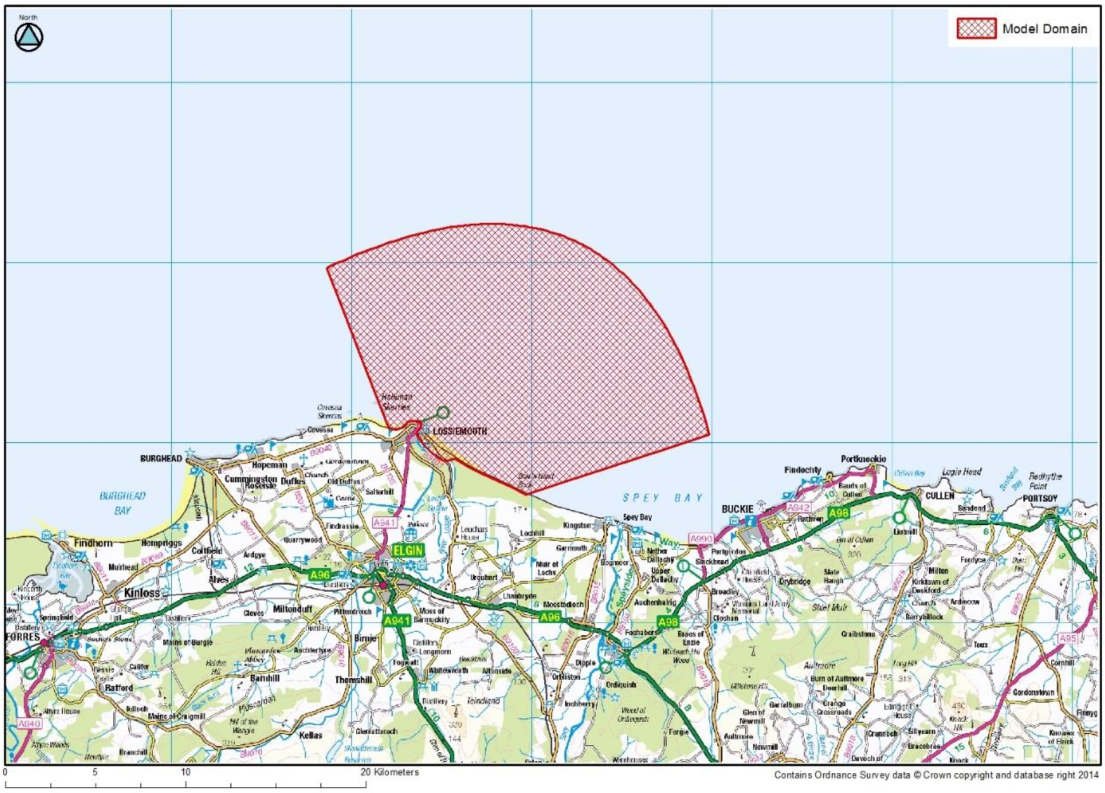

Figure C-1: Extent of Wave Transformation Model

Bathymetry data

A key element of the data required for the wave transformation model was a bathymetric dataset, which represents the elevation of the sea bed. A bathymetry grid was constructed for the model domain based on several sources of data. Bathymetry data was supplied by MarineFIND, with LiDAR provided by The Moray Council. This was augmented with measurements taken during the site visit in order to provide a good representation of the key features within the model. The bed level within the channel of the River Lossie was not well represented within either the LiDAR or bathy data, and as such was set to a level of -0.4m, which is noted to be the ‘deepest soundings in the river mouth’[1]. The resulting merged bathymetry is shown within Figure C-2.

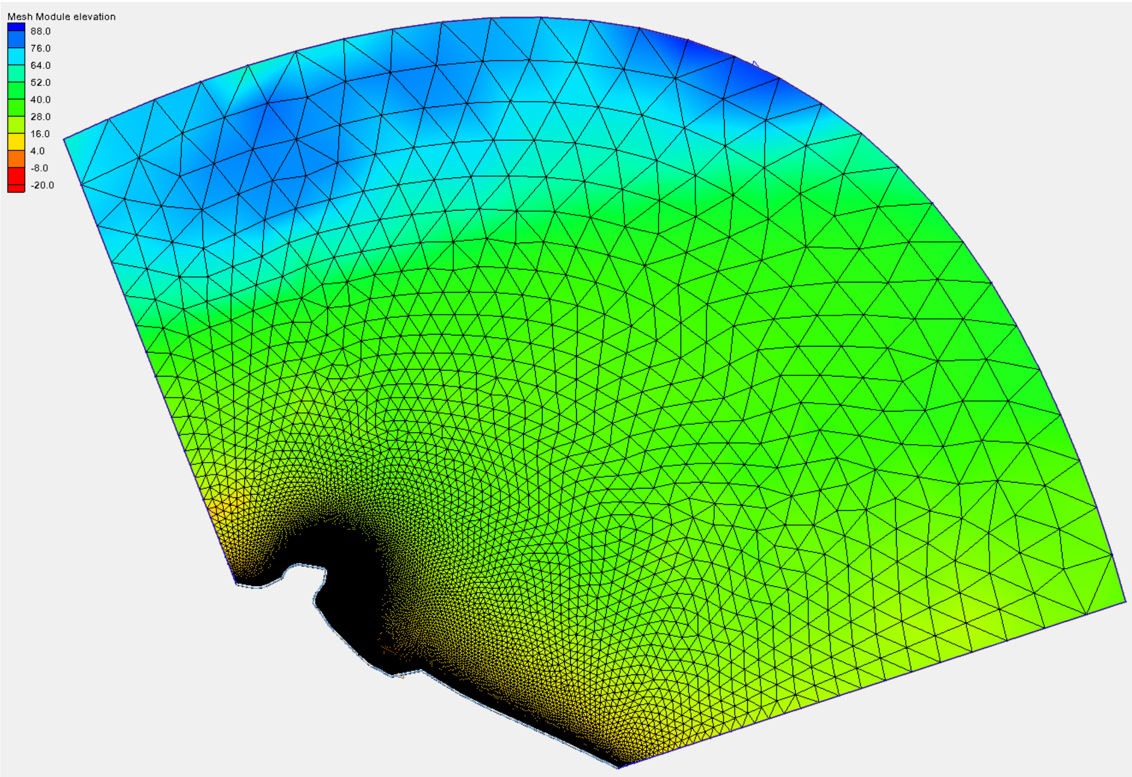

Figure C-2: Wave Transformation Model Bathymetry

Following the construction of the existing bathymetry dataset, three further grids were produced; the first with the gap in the breakwater repaired, the second with the partial removal of the breakwater and the third with the full removal of the structure. It should be noted that no other changes to the surrounding bathymetry were made, which in reality is unrealistic due to the effects that the partial or full removal of the breakwater would have on sediment transport within the area. The resulting bathymetry in the vicinity of the river mouth is presented for each model scenario within Figure C-3.

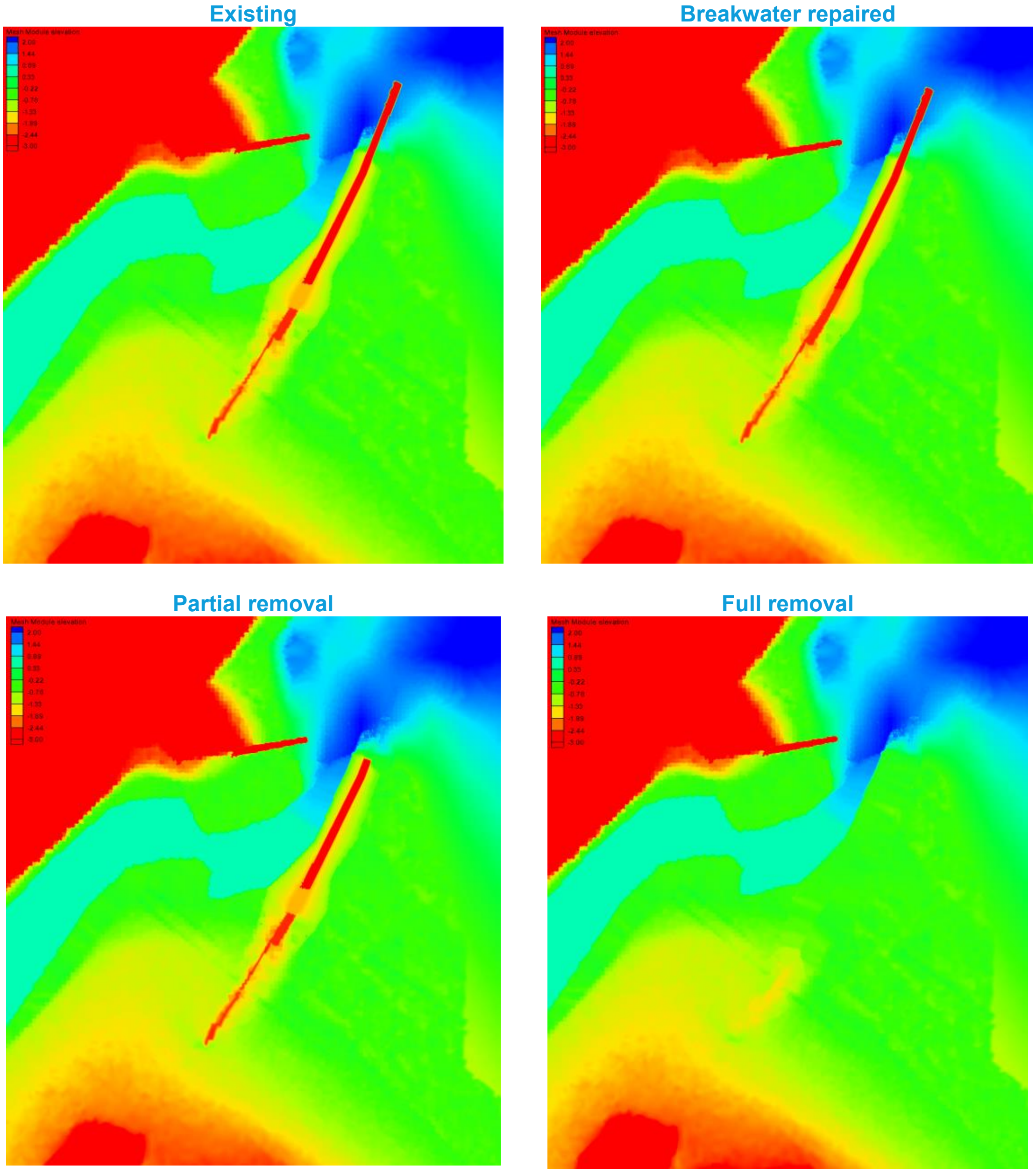

Figure C-3: Model Bathymetry at the River Mouth for the Different Model Scenarios

Calibration

No recorded wave data is available within the model domain, and calibration of the model was therefore not possible. However, JBA recently constructed a wider-scale wave transformation model for the Moray Firth during the development of the Moray Firth Flood Warning System (FWS) for SEPA[23]. Within this project, the wave transformation model was calibrated using the Moray Firth Waverider buoy. The final calibrated model had an accuracy of 8% at the buoy, with sensitivity analysis showing that model errors present within deep water, e.g. at the buoy, are greatly diminished within the nearshore due to the depth limitation of the waves. It was therefore felt appropriate to use the physics (Janssen) and roughness (Madsen) parameters from the wider scale Moray Firth model within the local model used for this study.

Input parameters

Inputs required for the model were still water level, offshore wave height, period and direction, and wind speed and direction.

[23] Moray Firth Flood Warning System, Draft Final Report, JBA Consulting, September 2014

Still water levels[9] and offshore wave heights[24] for a range of return periods are presented within Table C-1.

Table C-1: Extreme still water levels and swell wave heights

Return Period (years)	Still Water Level (mAOD)	Swell Wave Height (m)	
		North	Northeast
1	2.68	3.10	3.43
2	2.75	3.29	3.55
5	2.83	3.50	3.67
10	2.90	3.64	3.75
25	2.98	3.81	3.83
50	3.04	3.91	3.88
100	3.10	4.01	3.93
200	3.16	4.09	3.96

In order to understand the interaction of these two factors, joint probability analysis was undertaken. This followed the industry standard desk study method developed by Defra[25], with Table C-2 and Table C-3 showing the results of the joint probability analysis for waves from the north and northeast respectively.

Table C-2: Still water level and swell wave heights joint probability analysis – waves from north

Still Water Level (mAOD)	Joint Probability Return Period (years)							
	1	2	5	10	25	50	100	200
	Swell Wave Height (m)							
2.46	2.37	2.59	2.88	3.10	3.37	3.55	3.70	3.84
2.52	2.24	2.46	2.75	2.97	3.26	3.45	3.62	3.77
2.62	1.93	2.15	2.44	2.66	2.95	3.17	3.37	3.55
2.68	1.74	1.96	2.25	2.47	2.76	2.98	3.20	3.39
2.75		1.77	2.06	2.28	2.57	2.79	3.01	3.22
2.83			1.81	2.03	2.32	2.53	2.75	2.97
2.90				1.84	2.13	2.34	2.56	2.78
2.98					1.87	2.09	2.31	2.53
3.04						1.90	2.12	2.34
3.10							1.93	2.15
3.14								2.04
3.16								1.96

Table C-3: Still water level and swell wave heights joint probability analysis – waves from northeast

Still Water Level (mAOD)	Joint Probability Return Period (years)							
	1	2	5	10	25	50	100	200
	Swell Wave Height (m)							
2.46	2.97	3.11	3.29	3.43	3.60	3.70	3.78	3.85
2.52	2.89	3.03	3.21	3.35	3.53	3.64	3.74	3.81
2.62	2.69	2.83	3.01	3.15	3.33	3.47	3.60	3.70
2.68	2.57	2.71	2.89	3.03	3.21	3.35	3.49	3.61
2.75		2.59	2.77	2.91	3.09	3.23	3.37	3.51
2.83			2.61	2.75	2.93	3.07	3.21	3.35
2.90				2.63	2.81	2.95	3.09	3.23
2.98					2.66	2.79	2.93	3.07
3.04						2.67	2.81	2.95
3.10							2.69	2.83
3.14								2.76
3.16								2.71

[24] Coastal Flood Boundary Conditions for UK Mainland and Islands, Project SC060064/TR3: Design Swell Waves

[25] Defra/Environment Agency Flood and Coastal Defence R&D programme, Use of Joint Probability Methods in Flood Management: A Guide to Best Practice, R&D technical Report FD2308/TR2

The coastal flood boundary study[24] also provides details with regard to wave period. Table C-4 shows the proportion of waves of a given period for a certain wave height, along with the period used within this study in the final column.

Table C-4: Swell wave period

Wave Height (m)	Proportion of Waves of Period					Period to Use (sec)
	Tz<8s	Tz 8-10s	Tz 10-12s	Tz 12-14s	Tz 14-16s	
Hs <1	0.52	0.29	0.14	0.04	0.01	8
Hs 1-2	0.46	0.39	0.11	0.03	0.01	8
Hs 2-3	0.18	0.51	0.28	0.03	0.00	9
Hs 3-4	0.12	0.42	0.42	0.03	0.00	10

Winds for a range of return periods can be estimated using an industry standard approach outlined in the documents 'Revetment Systems against Wave Attack: A Design Manual'[26] and the 'Offshore Technology Report'[27]. The method is based on the 50 year wind speed available from contour maps and adjusting this estimate to other return periods using a number of correction factors.

The 50 year wind speed was obtained from the map published within the Floods and Reservoir Safety report[28], which at Lossiemouth was 24m/s. To convert this 50 year wind speed to other return period wind speeds, the following conversion equation outlined in the Revetment Systems design manual was used:

$$U_D = U_b S_a S_d S_p S_f S_w \qquad \text{(Eq 1)}$$

where:

U_D is design wind speed (m/s)

U_b is 50 year basic hourly wind speed (m/s)

S_a is an altitude factor

S_d is a factor to account for the wind direction (e.g. south-westerly winds tend to be stronger than north-easterlies over the UK)

S_p is a factor to adjust for different return periods

S_f is a factor to convert hourly wind speed to a more appropriate duration for the water body under study

S_w is an over-water speed-up factor to account for the effect of reduced friction as wind travels over water. This value is based on fetch.

Values for the aforementioned variables were taken from tables given in the Revetment System manual.

The values of wind speed obtained through this method for winds from both the north and northeast are detailed within Table C-5.

Table C-5: Wind speeds for range of return periods

Return Period (years)	North (m/s)	Northeast (m/s)
1	17.3	16.1
2	17.3	16.1
5	21.4	20.0
10	22.7	21.2
25	23.9	22.4
50	25.7	24.1
100	27.0	25.3
200	28.3	26.5

Model runs

Combining all these factors resulted in 122 model runs, examples of which are presented within Table C-6.

[26] McConnell, K., 1998, Revetment Systems against Wave Attack: A Design Manual. Thomas Telford, London.

[27] Bomel Ltd., 2002, Offshore Technology Report (2001/010): Environment Considerations.

[28] ICE, 1996, Floods and Reservoir Safety. 3rd Edition. Thomas Telford, London.

Table C-6: Model runs

Return Period (years)	Water Level (mAOD)	Wave Height (m)	Wave Period (s)	Wave Direction (°)	Wind Speed (m/s)	Wind Direction (°)
1	2.46	2.37	9	0	17.3	0
1	2.52	2.24	9	0	17.3	0
1	2.62	1.93	8	0	17.3	0
1	2.68	1.74	8	0	17.3	0
2	2.46	2.59	9	0	17.3	0
2	2.52	2.46	9	0	17.3	0
2	2.62	2.15	9	0	17.3	0
2	2.68	1.96	8	0	17.3	0
2	2.75	1.77	8	0	17.3	0
5	2.46	2.88	9	0	21.4	0
5	2.52	2.75	9	0	21.4	0
⋮						
100	2.98	2.93	9	45	25.3	45
100	3.04	2.81	9	45	25.3	45
100	3.10	2.69	9	45	25.3	45
200	2.46	3.85	10	45	26.5	45
200	2.52	3.81	10	45	26.5	45
200	2.62	3.70	10	45	26.5	45
200	2.68	3.61	10	45	26.5	45
200	2.75	3.51	10	45	26.5	45
200	2.83	3.35	10	45	26.5	45
200	2.90	3.23	10	45	26.5	45
200	2.98	3.07	10	45	26.5	45
200	3.04	2.95	9	45	26.5	45
200	3.10	2.83	9	45	26.5	45
200	3.14	2.76	9	45	26.5	45
200	3.16	2.71	9	45	26.5	45

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