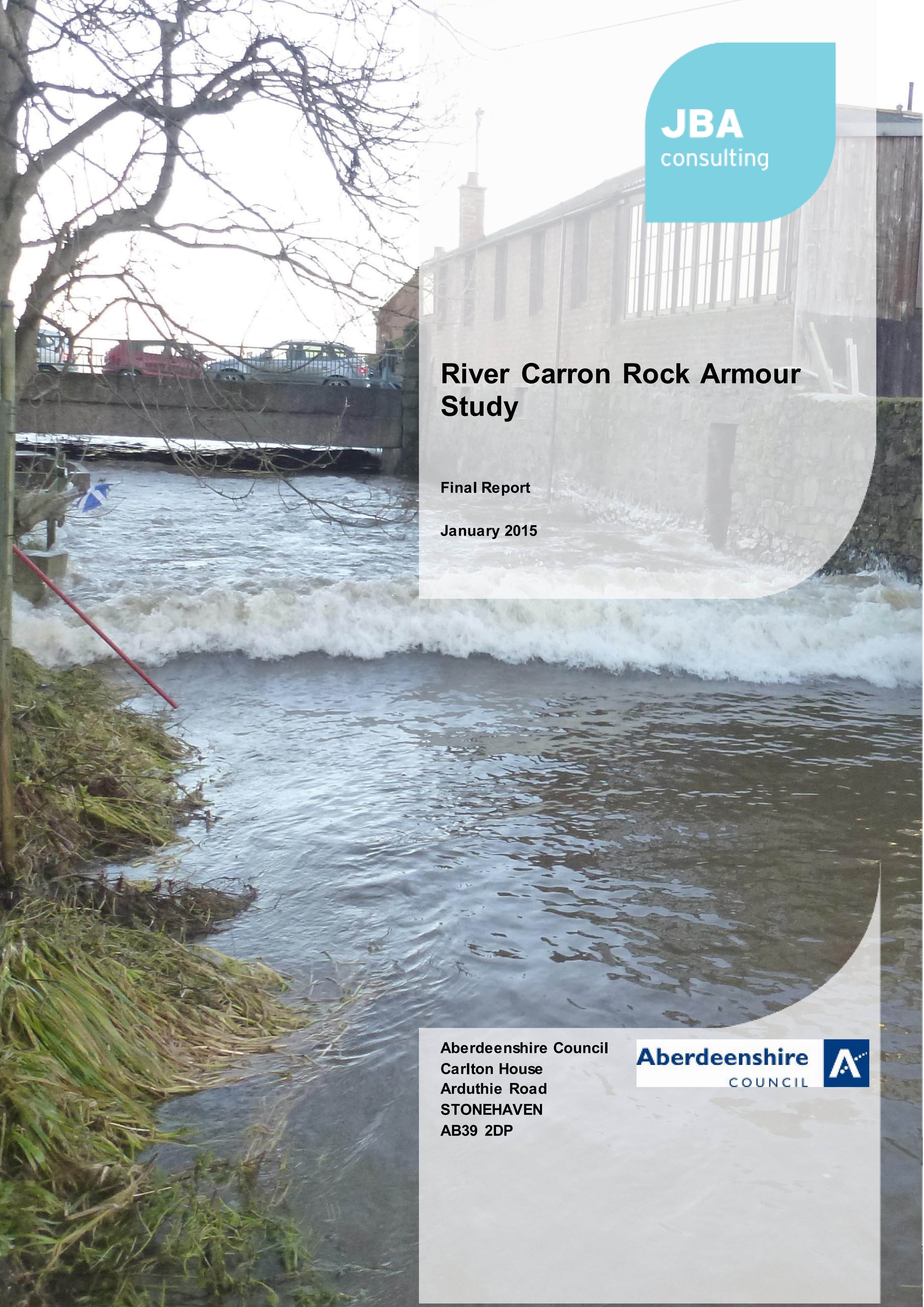

River Carron Rock Armour Study

Final Report

January 2015

Aberdeenshire Council
Carlton House
Arduthie Road
STONEHAVEN
AB39 2DP

JBA Project Manager

Daniel Rodger,
South Barn, Broughton Hall,
SKIPTON,
North Yorkshire,
BD23 3AE

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Contract

This report describes work commissioned by Aberdeenshire Council, by a letter dated 11 April 2014. Aberdeenshire Council's representatives for the contract was Rachel Kennedy. Rami Malki, Alexander Dane, Nicci Buckley and Daniel Rodger of JBA Consulting carried out this work.

Prepared by ....Daniel Rodger BEnvSci, MEng, GradDipGIS, GCert CoastalEng, CEng, MICE, CMarEng MIMarEST

Senior Coastal Engineer

Reviewed by ....Graham Kenn BSs, MSc (Eng), CEng, MICE, C.WEM, MCWEM

Technical Director

Reviewed by ....Mark Lawless BSc MSc PhD CSci CEnv MCIWEM C.WEM

Head of Coastal Risk Management

Approved by ....David Bassett BSc MSc CEnv MCIWEM C.WEM

Director

Purpose

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Acknowledgements

We would like to acknowledge Partrac Ltd for the supply of metocean data for use in this study, feedback and descriptions of wave processes from residents, and assistance from the staff at Aberdeenshire Council whilst carrying out this work.

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Executive summary

This study was undertaken by JBA Consulting, on behalf of Aberdeenshire Council, to investigate the wave propagation within the River Carron, Stonehaven.

Coastal flooding is a problem that has widespread impacts for communities, businesses and infrastructure. Near the coastline, flooding can occur from a variety of processes such as through wave overtopping, still-water flooding due to coastal surges and storms, or riverine flooding originating from rivers and estuaries. For the coastal town Stonehaven, Aberdeenshire UK, flooding occurs due to a complex mix of each of these processes, caused by extreme waves and surges from the North Sea, and river flows from the River Carron.

The interaction with the River Carron and the sea creates an area of complex coastal and fluvial processes that can act in either isolation or combination. Following rainfall the River Carron can experience high flows which can break its banks to inundate the surrounding floodplain. During significant coastal events, comprised of extreme sea levels and/or wave conditions, waves can enter the waterway channel, propagating upstream and increasing water levels. Whilst these two processes are driven by different fluvial and coastal processes, the potential for their interaction can lead to flood levels higher than either process alone.

The Aberdeenshire Council are currently working on the River Carron Flood Alleviation Scheme (FAS) to reduce fluvial flood risk. Fluvial flood assessments undertaken for the project have estimated extreme river water levels based on an upstream flow with a return period of 1 in 200-years plus climate change to 2115, and a downstream sea level of 1 in 1-year based on present day conditions. For a reference point upstream of the Bridgefield Bridge the design river water level was calculated at 5.43mAOD. When constructed, the FAS will include new river defences to minimise flood risk, constructed with a freeboard allowance of 0.45m and a defence crest at 5.88mAOD.

This study has investigated the potential impact of waves propagating within the River Carron in relation to this defence crest level.

Assessment of wave risk

The assessment to quantify the size of channel waves was based on a combination of anecdotal and historic information, photographs and numerical wave modelling. The assessment suggested the magnitude of channel waves photographed during the 15 December 2012 event had a return period of approximately 1 in 10-years, with an estimated wave height of 1.00m. Larger wave heights are expected to occur, estimated up to 1.14m upstream of the Bridgefield Bridge during a 1 in 200-year event (based on present-day conditions), which could increase to 1.36m under the influence of climate change to 2115.

The highest water levels experienced within the River Carron channel will be due to a combination of ocean waves, astronomical tides and tidal surges (combining to form an extreme still water level) and river flows, and will be affected by increasing sea levels. The assessment has calculated the peak wave-crest water level based on a 1 in 200-year joint-probability scenario, where both upstream and downstream inputs include climate change increases. The peak wave-crest water level for a 200-year climate change event was estimated to be 6.3mAOD upstream of Bridgefield Bridge.

The estimated wave-crest level is higher than the FAS river defences and suggests that if extreme waves were to coincide with an extreme fluvial event the resulting wave-crest level would overtop the river defences. However, several influencing factors have not been able to be quantified in-depth, which may reduce this wave height. The interaction of waves and flow is not able to be determined in this assessment, however a comparison of the wave speed versus flow velocity shows it is likely waves would overcome the channel velocity, which has been observed in low-flow conditions. The effect of the Bridgefield Bridge in limiting wave crests has been estimated based on analytical equations for vertical, submerged barriers, however do not take into consideration the width of the bridge. The rate of overtopping would be controlled by the frequency of wave crests, i.e. their period, which is expected to result in overtopping three to four times per minute during the high tide.

Efficiency of training wall options

The cause of the wave propagation was investigated, and is believed to be due to a number of processes occurring simultaneously. During high tides and extreme sea levels a combination of wave groups and irregular waves can lead to the combination of wave crests, which can be held stationary due to the discharging river flow. As combined waves overcome the river flow and pass into the entrance they are squeezed together due to the training wall geometry. Once a wave passes the river mouth it reaches a relatively flat channel gradient, and is able to propagate freely upstream. Narrowing sections of the channel, for example upstream of the Bridgefield Bridge, cause the waves to be squeezed together further, and results in an increase to the wave height.

The efficiency of the existing training walls was assessed, in terms of its ability to reduce incoming wave energy. The existing alignment was found to reduce incoming wave energy by approximately 30% (e.g. during extreme conditions a nearshore wave of 1m would reduce to approximately 0.7m within the channel).

Several options were considered to further decrease wave height. However, any redesign was constrained by a sewerage conduit located under the channel mouth, and a requirement to not worsen peak river discharges. The design efficiency was found to be greatly affected by the mouth geometry, with any options that accentuate the funnel shape or opened it directly to wave attack resulted in an increase to the channel wave height.

The best performing option was identified as a nearshore detached breakwater, estimated to reduce incoming wave energy by approximately 70% (e.g. during extreme conditions a nearshore wave of 1m would reduce to approximately 0.3m within the channel). Due to the low-lying nature of the river outlet, the impact of climate change and the necessity to maintain river hydraulic efficiency, it is not expected that wave propagation could be stopped completely.

Cost effectiveness analysis

A cost effectiveness analysis was undertaken to compare the costs of constructing a nearshore breakwater versus raising the River Carron FAS wall height to include an allowance for waves.

Based on a high-level assessment, the costs for constructing a breakwater would be approximately £1,800,000. However, after construction it is expected that some wave energy will remain, with 0.3-0.4m waves still propagating within the channel during an extreme event. To maintain a suitable freeboard, an increase to the River Carron FAS wall height would be required, estimated to cost approximately £1,500,000. The total cost of the breakwater plus increased freeboard option is therefore estimated to be of the order of £3,250,000.

Alternatively, waves could be mitigated by increasing the height of the FAS walls only. To maintain a suitable freeboard above the wave crests an increase of 0.9m would be required. Based on unit-rate cost estimates up to White Bridge, this would cost approximately £3,230,000.

Information relating to the likely wave impacts was provided to consider the implications of a *do nothing* approach. It is expected that any overtopping will be periodic, limited to times of high tides and extreme sea levels, and will only occur as a wave crest propagates upstream (e.g. three to four waves per minute).

Recommendations

Several key recommendations are made to address either the uncertainty in the modelling, or to assist in using the study conclusions in future planning. These are:

1. The options of a curved northern training wall, southern extension or a straightened channel should not be considered further as they were found to offer no improvement to upstream wave conditions.
2. As the construction costs for the breakwater and wall raising options are quite high, the *do nothing* option may be considered the most appropriate until a coastal protection scheme is considered to address more general wave overtopping issues.
3. The assessment shows the potential for waves to reach the soffit of Bridgefield Bridge during high water levels, therefore able to break against the bridge and parapet (which is recommended to be infilled). Any overtopped water will bypass the River Carron FAS, with the potential to flow north or south into the low-lying areas. It is recommended that the parapet upgrade incorporates a wave-return wall to direct overtopped water seaward, away from the bridge.
4. It is recommended that a revised joint probability assessment is undertaken to increase the reliability of nearshore and channel wave estimates, adopting a methodology such as

that proposed by Heffernan and Tawn[1]. This new assessment should be specifically developed to assess the joint probability between waves, coastal still water levels and river flow, and should be conducted prior to any detailed design or construction.

5. Further physical data collection is recommended from within the River Carron channel, which could be used to understand the interaction with waves and flow, and to validate future physical models. Ideally this information will capture a wave event during high flow conditions.
6. The modelling results presented in this report are considered conceptual. As with any numerical models, the results are a simplification of complex physical processes. While the modelling results serve as a useful indicator of the wave trends, it is recommended that detailed physical modelling involving waves and flow is carried out prior to any detailed design or construction.

[1] Heffernan, J.E., Tawn, J.A., 2004. A conditional approach for multivariate extreme values (with discussion). J. R. Stat. Soc. Ser. B Stat Methodol. 66 (3), 497–546.

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Abbreviations and acronyms

1D	One-dimensional
2D	Two-dimensional
3D	Three-dimensional
AWAC	Acoustic Waves and Currents
BW	Boussinesq wave (model)
CC	Climate change
CDM	Construction Design and Management Regulations (2007)
CEFAS	Centre for Environment, Fisheries and Aquaculture Science
CS3X	NOC tide/surge model
Defra	Department for Environment, Food and Rural Affairs
DEM	Digital Elevation Model
DIS	Design Input Statement
EIA	Environmental Impact Assessment
EurOtop	European Wave Overtopping Manual
FAS	Flood Alleviation Scheme
HAT	Highest Astronomical Tide
H_{m0}	Significant Wave Height (spectral domain), considered in the River Carron
$H_{m0nearshore}$	Significant Wave Height (spectral domain), considered in the nearshore
Hs	Significant Wave Height
JBA	Jeremy Benn Associates
LAT	Lowest Astronomical Tide
MHWN	Mean High Water Neaps
MHWS	Mean High Water Springs
MLWN	Mean Low Water Neaps
MLWS	Mean Low Water Springs
NOC	National Oceanography Centre
PDF	Portable Document Format
RMS	Root-Mean-Square
SWAN	Simulating WAves Nearshore (wave model)
SWL	Still Water Level
Tm	Wave Period
Tp	Peak Wave Period
UKCP09	UK Climate Projections 09
WWIII	Wave Watch 3 (a large scale wave model)

1 Introduction

1.1 Project background

This study was undertaken by Jeremy Benn Associates (JBA) Consulting, on behalf of Aberdeenshire Council, to investigate the magnitude of wave propagation within the River Carron, Stonehaven. The River Carron discharges in southern Stonehaven, and is located approximately 15 miles south of Aberdeen, Scotland (see Figure 1-1). Figure 1-2 shows the trained river mouth during a low tide.

Following flooding in November 2009, Aberdeenshire Council proposed the construction of the River Carron Flood Alleviation Scheme (FAS) to improve the standard of fluvial flood protection from the River Carron. However, under the existing channel layout waves have been observed in the river during the period of high tide, propagating into the channel from the sea. This study was undertaken to quantify the scale of these waves to allow for the robust design of the proposed defences on the lower stretches of the Carron. This project had three aims:

1. To estimate the degree to which wave propagation will increase water levels in the river based on the current armourstone alignment.
2. To assess the efficiency of the current orientation of the armourstone training structures in decreasing the ability of waves to propagate upstream and make recommendations for improvement.
3. To provide an outline design for the alignment of the rock armour to minimise the opportunity for wave propagation whilst ensuring maximum discharge from the River Carron.

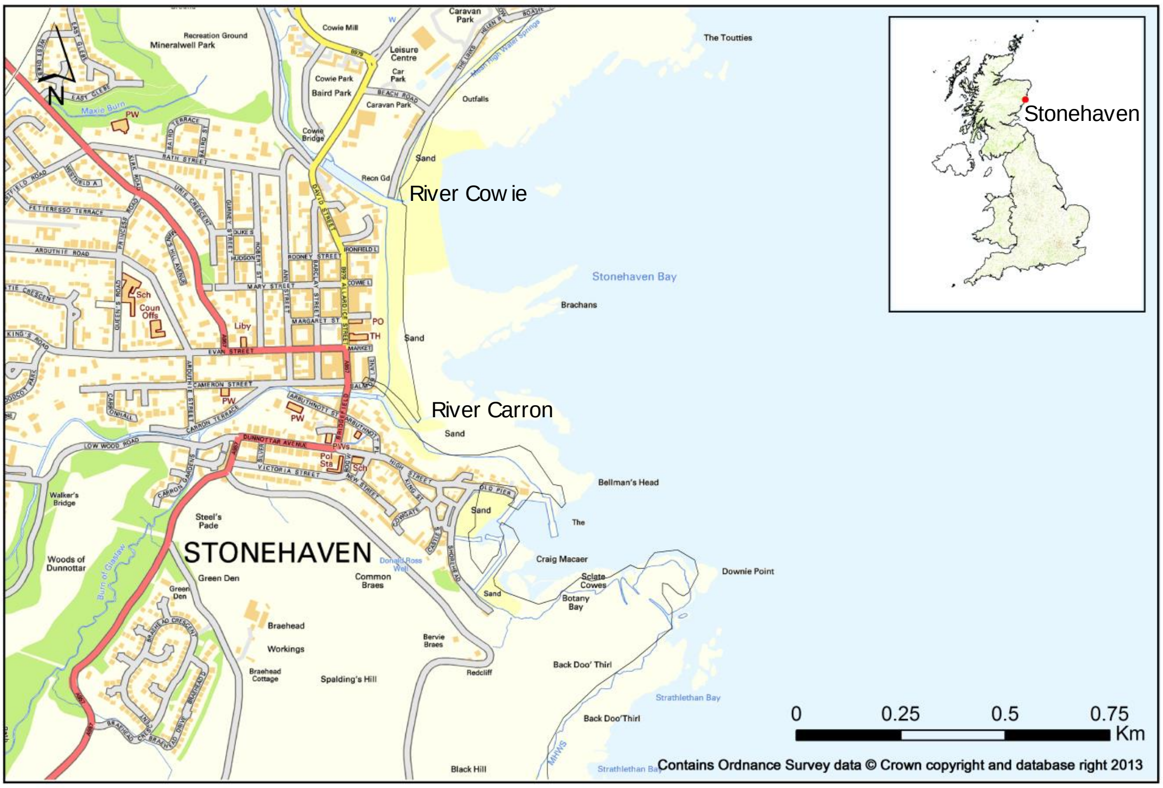

Figure 1-1: The study site at Stonehaven.

Figure 1-2: The River Carron training w alls, facing north-west from mouth (JBA Consulting 01/05/2014).

1.2 Report structure

This report consists of the following chapters:

- **Chapter 2 (Review of coastal processes)** reviews previous investigations and anecdotal information and describes the processes that lead to wave propagation within the River Carron.
- **Chapter 3 (Data collection)** describes the metocean data collected for this study, which includes tidal water levels, nearshore wave and channel water level information.
- **Chapter 4 (Quantification of existing wave propagation)** quantifies the degree to which wave propagation increases water levels within the River Carron.
- **Chapter 5 (Training wall improvement assessment)** evaluates the efficiency of the current rock armour training walls in protecting the river from wave propagation and investigates new training wall and coastal protection structures.
- **Chapter 6 (Preferred design and cost effectiveness assessment)** provides further analysis of the preferred design to minimise wave propagation and considers the cost of construction.
- **Chapter 7 (Conclusions and recommendations)** summarises the project outcomes and presents the key recommendations for the project.

2 Review of coastal processes

2.1 Introduction

Coastal flooding is a problem that has widespread impacts for communities, businesses and infrastructure. Near the coastline, flooding can occur from a variety of processes such as wave overtopping, still water flooding due to coastal surges and storms, or riverine flooding originating from rivers and estuaries. In order to reduce the impact of coastal flooding, it is first important to consider the local mechanisms of coastal risk. It is essential that any proposed solutions account for these processes in as realistic a manner as possible; otherwise the designs will be unreliable.

In this chapter the available information on coastal flood risk within the River Carron is assessed, including photographic evidence, scientific reports, anecdotal accounts and new data collected for this study. Using these data the principal mechanisms leading to wave propagation are discussed to provide a conceptual understanding for the coastal risk. This chapter is divided into the following sections:

- **Previous investigations:** reviews previous studies and reports relevant to this project.
- **Anecdotal information, available photographs and internet footage:** describes other available information and observations of the channel waves.
- **Drivers of coastal risk:** presents a conceptual model of the processes leading to wave propagation into the River Carron.

2.2 Previous investigations

While there have been a variety of coastal investigations completed at Stonehaven, not all studies have focussed on the wave propagation within the River Carron. The most relevant are considered to be the following, which were reviewed to provide background information:

- JBA (2013) Stonehaven River Carron and Glaslaw Burn Preferred Flood Protection Scheme Report (JBA Consulting, 2013).
- JBA (2014) Stonehaven Coastal Frontage Assessment.
- Canterbury City Council (2013) Topographic Baseline Survey Report 2013.
- HR Wallingford (1998) Stonehaven Seawall, Aberdeenshire, Feasibility Study of Improvements, EX 3731.
- Anecdotal information, available photographs and internet footage.

2.2.1 Stonehaven River Carron and Glaslaw Burn Preferred Flood Protection Scheme Report (JBA Consulting, 2013)

The focus of this study was on alleviation of flood risks to Stonehaven due to flow in the River Carron, however it did not specifically investigate channel waves. A number of options were considered, with the preferred design consisting of river defences on both banks of the River Carron, raising of three bridges (the Red, Green and White Bridges), localised channel modifications to increase the channel capacity, provision of pumping stations in low lying areas and infilling the parapet on Bridgefield Bridge.

2.2.2 JBA (2014) Stonehaven Coastal Frontage Assessment, and Canterbury City Council (2013) Topographic Baseline Survey Report 2013.

These investigations reviewed the beach recycling activities and sediment changes between 2008 and 2013 at Stonehaven. The assessment showed a general accumulation of sediment throughout the bay, although an ongoing loss of sediment was observed to the south of the River Carron. These trends support recommendations for the addition of short groynes to the south of the River Carron that would help stabilise the sediment deposited in this area, and may allow greater time between mechanical beach recycling works by Aberdeenshire Council.

2.2.3 Stonehaven seawall, Aberdeenshire - Feasibility study of improvements (HR Wallingford, 1998)

This study focused on wave overtopping of the seawall and flooding of the lower reaches of the River Carron, however it did not specifically investigate channel waves. The report identifies the impact of overtopping the Stonehaven coastal frontage, which can cause large quantities of shingle to be deposited on roadways resulting in the blockage of drains. Several mitigation options were investigated, such as increases to the beach level and several structural options. A number of alternative training walls were proposed and investigated to minimise the potential for long-term erosion and coastal flooding. As shown in Figure 2-1, these consist of straight and curved northern training walls, a southern training wall (unlikely to be successful as coarse material is likely to block river during periods of low flow), and a curved northern training wall with southern groynes. The reports conclude that the most cost-effective option is a curved northern training wall which must be accompanied by regular mechanical by-passing of beach material.

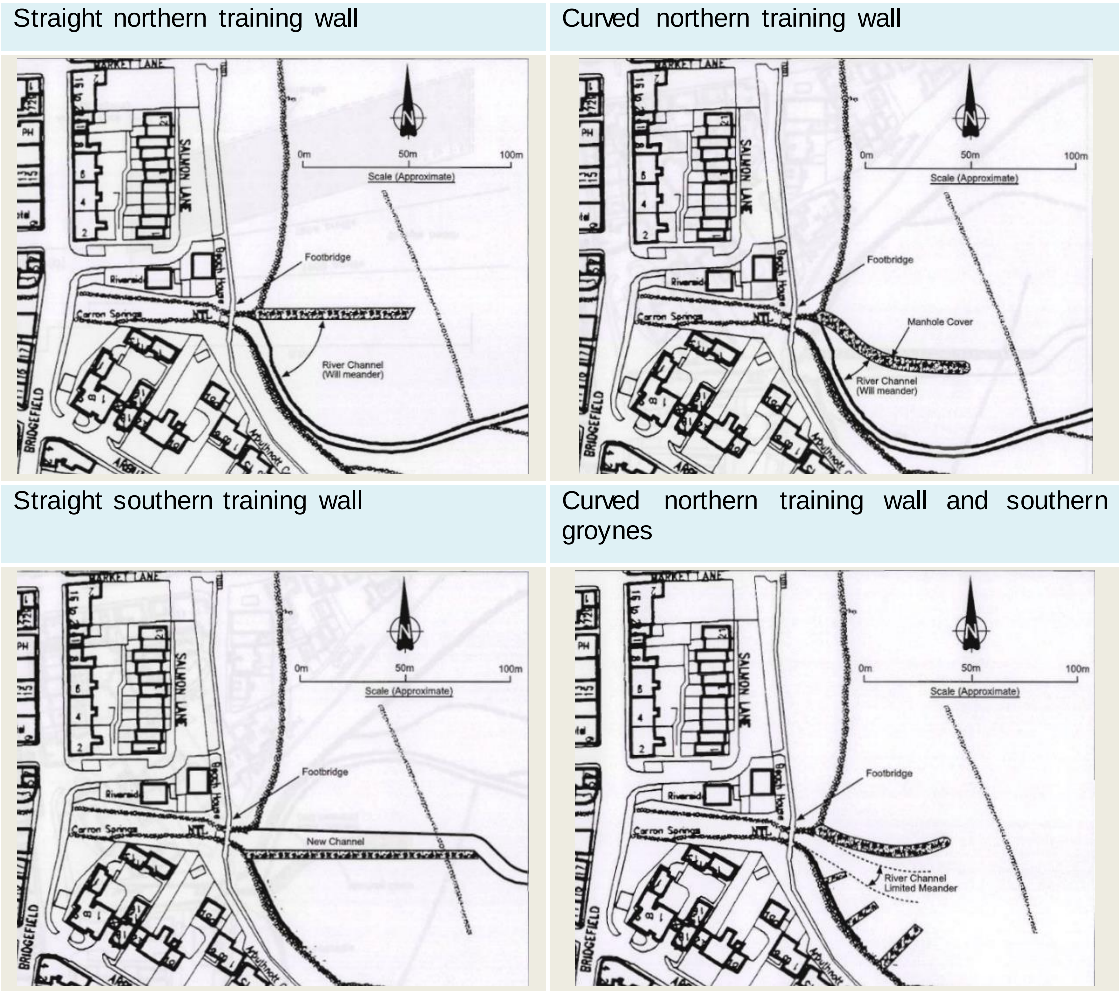

Figure 2-1: Training wall orientations proposed in the Stonehaven seawall assessment (HW Wallingford 1998)

2.3 Anecdotal information, available photographs and internet footage

The wave action within the River Carron has been recorded on numerous occasions during periods of high tide, both at the river mouth and within the channel itself. Photographs and videos show two different wave processes, with propagating waves either displaying motion characteristics of several short-crested waves in a wave train (e.g. under 12 seconds), or a long surging soliton wave. A number of observations are summarised below in order to consider these processes.

2.3.1 Waves at the channel mouth

Several videos of waves entering the River Carron channel exist. The videos show the combination of individual irregular waves and wave groups within the nearshore zone. The general sequence of events leading to a channel wave is as follows; under a constant discharge of water from the River Carron a near stationary wave can form in the channel mouth. Larger waves continue to approach the mouth and build-up behind the stationary wave. The interaction of the individual waves leads to constructive interference of the wave crests, and allow the superposition of a single large wave at the channel entrance. This large wave crashes over the river mouth and enters the channel.

Figure 2-2: Still images taken from a video of the River Carron waves. Source: McDonald (2014)[2]

2.3.2 Wave propagation within the channel

The channel gradient is controlled by a sewerage conduit which crosses under the seaward end of the training wall. The conduit maintains the channel invert level, which results in a steep beach face towards the sea and a flat gradient (and water level) extending upstream. Observations of waves entering the channel indicate the training wall geometry funnels the wave crests together, causing an increase to the wave height[3]. After crossing the sewerage conduit the heightened wave then propagates freely within the channel towards Bridgefield Bridge. Waves have been observed increasing in height upstream of the bridge, which is attributed to the narrowing channel. Through this section a range of wave forms have been observed, described as including solitary waves, wave trains, regular unbroken waves, cresting waves and broken 'white' waves.[4]

The most significant waves that have been captured photographically in the River Carron occurred on 15 December 2012. This event was characterised by relatively large sea levels (larger than a typical high water spring however lower than 1-year extreme sea level), and extremely large offshore waves with a return period over 200-years[5]. Photographed channel waves have the appearance of a steep face and a relatively long wavelength, which shows a build-up of water behind the crest (see Figure 2-3).

The height of this wave was considered to be the difference between the still water level, or average flow level, and the wave crest level. The wave during the 2012 event was estimated using two approaches. Firstly, the difference in water level was estimated against the southern channel wall where the wave passes a column of breeze-blocks. While the blocks can be observed to vary in height, assuming a standard size[6] of 225mm, the water level variation is considered to be approximately 1m. Secondly, the photographs show that the wave did not quite enter the rear of the photographer's property, although images of flattened grass indicates it came close. Using available field survey, the top of bank level is considered to be around 3.6mAOD. Subtracting the

[2] McDonald (2014) 'River Carron Soliton waves at Stonehaven Scotland. Stonehven Flood Group.', accessed from www.youtube.com on 01/08/2014: https://www.youtube.com/watch?v=Rhgtx0Wsu6c

[3] Correspondence between Aberdeenshire Council and Mr Ian G. McDonald. Report supplied by Mr McDonald: 'Wave action in Stonehaven Bay since the early nineteen fifties'.

[4] Correspondence between Aberdeenshire Council and William Munro. Email date 12 August 2014.

[5] It is expected that the occurrence of waves of this magnitude will change the statistics behind extreme wave conditions, and consequently this will be an over-estimate of the return period. For more information see JBA (2014) Stonehaven Coastal Frontage Assessment.

[6] Based on a standard height of 215mm and an allowance of 10mm for mortar. Block size referenced from: http://www.wickes.co.uk/Wickes-Dense-Block-7-3-N-100mm/p/113504

peak water level of 2.33mAOD (recorded at the Aberdeen gauge), this indicates the waves were smaller than 1.27m.

These estimates match anecdotal information supplied by local residents that the largest waves observed within the channel had a height of approximately three feet[7], or one metre[8].

Figure 2-3: Photographs taken during the 15 December 2012 coastal event w ithin the River Carron show ing (top) large w ave propagation, (lower left) draw down betw een wave crests, and (lower right) successive wave crests.

[7] Correspondence between Aberdeenshire Council and Mr Ian G. McDonald. Report supplied by Mr McDonald: 'Wave action in Stonehaven Bay since the early nineteen fifties'.

[8] Correspondence betw een Aberdeenshire Council and William Munro. Email date 12 August 2014.

2.3.3 Similar processes in the River Cowie

Wave propagation is not restricted to the River Carron. Waves have been observed in the River Cowie, situated 450m to the north (See Figure 1-1). Anecdotal observations suggest that during storm conditions wave propagation is more pronounced within the River Cowie than the River Carron, which has been attributed to the more open river mouth. However, due to larger walls constructed around the River Cowie there is a reduced risk that waves could cause overtopping.

2.4 Summary and consideration of the drivers of coastal risk

Based on the information reviewed in this chapter, the cause of wave propagation within the River Carron channel is likely to be due to a number of processes occurring simultaneously. During high tides and extreme sea levels a combination of wave groups and irregular waves can lead to constructive interference of the wave crests, which can be held stationary due to the discharging river flows. As combined waves overcome the river flow and pass into the entrance they are squeezed together due to the mouth geometry. Once a wave reaches the relatively flat channel gradient (i.e. landward of the sewerage conduit) it is then able to propagate freely upstream. A conceptual diagram of these processes is shown in Figure 2-4.

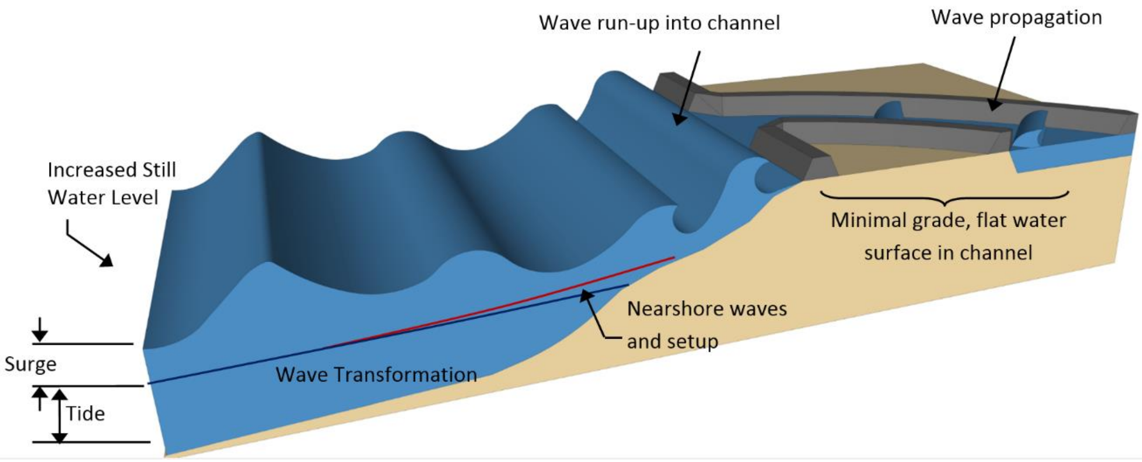

Figure 2-4: Components of wave propagation

3 Data collection

A range of data was used to evaluate the nature of wave propagation, including bathymetry, tide, extreme water level and wave estimates. In addition, new information was collected through a marine data collection campaign, which aimed to collect concurrent offshore, nearshore and channel wave information.

3.1 Bathymetry data

Nearshore bathymetry was obtained from FindMAPS for the study area, based on X, Y, Z survey points derived from surveys undertaken by the Civil Hydrographic Programme, Royal Navy surveys, Centre for Environment, Fisheries and Aquaculture Science (CEFAS) surveys as well as surveys from local port and harbour authorities. The data were supplied as a gridded dataset, processed and output into a 0.5 arc second grid. These data were combined with beach profile topographic survey information provided by Aberdeenshire Council, which was undertaken in May 2013[9]. A seamless Digital Elevation Model (DEM) was created by merging the bathymetry with the topographic survey data, which has been used in wave transformation modelling.

3.2 Available metocean information

3.2.1 Tidal conditions and extreme water levels.

Total tide software was used to extract the astronomical tide information for Stonehaven, based on Admiralty Chart information, and is shown in Table 3-1. This information is shown for information only, with the wave assessments undertaken based on extreme sea levels.

Extreme still water levels (SWL) at Stonehaven for a range of return periods have been obtained from the coastal flood hazard study. They have been updated to represent the expected increase due to the effects of climate change, based on the latest UK Climate Projections (UKCP09)[10]. A medium emissions scenario with a 95th percentile confidence interval is considered to result in a 0.67m rise in sea level by 2115. This climate change scenario reflects other guidance established for England and Wales[11]. The present day and climate change extreme SWLs are shown in Table 3-1.

It is important to note that recent extreme events have changed these statistics, and current studies are being undertaken to re-estimate extreme water levels.

Table 3-1: Tide levels and extreme water levels at Stonehaven

Water level event	Present day Level (mAOD)	Climate change level (2115) (mAOD)
1 in 200-year	3.25	3.92
1 in 100-year	3.19	3.86
1 in 50-year	3.12	3.79
1 in 20-year	3.03	3.70
1 in 10-year	2.97	3.64
1 in 5-year	2.89	3.56
1 in 2-year	2.80	3.47
1 in 1-year	2.73	3.40
Highest Astronomical Tide (HAT)	2.65	3.32
Mean High Water Springs (MHWS)	2.05	2.72
Mean High Water Neaps (MHWN)	1.15	1.82

[9] Canterbury City Council (2013) Topographic Baseline Survey Report 2013

[10] DEFRA, Crown Copyright, (2009), UK Climate Projections.

[11] Environment Agency (2010), Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities.

Mean Sea Level (MSL)	0.17	0.84
Mean Low Water Neaps (MLWN)	-0.75	-0.08
Mean Low Water Springs (MLWS)	-1.85	-1.18
Lowest Astronomical Tide (LAT)	-2.45	-1.78

3.2.2 Extreme waves

Extreme coastal conditions were obtained from the Environment Agency (EA) / Scottish Environmental Protection Agency (SEPA) *Coastal flood boundary conditions for UK mainland and islands* project[12], which includes design swell wave conditions and sea levels around Scotland, England and Wales for a number of directions (see Table 3-2). The table indicates the largest wave heights offshore of Stonehaven originate from the northeast direction (a wave direction of 45°/N). Extreme offshore wave conditions for northeasterly waves are summarised in Table 3-3 with wave periods based on the mid-range trend presented in the *Coastal flood boundary conditions dataset (CFBD)*.

The latest UKCP09 projections indicate the likely reduction in winter swell wave height to the north of the UK, and an increase to the south of the UK. Based on available mapping[13], this indicates little change to the winter wave conditions at Stonehaven, and therefore no allowance has been made for climate change.

Table 3-2: Extreme wave height estimates at Stonehaven for offshore waves from varying directions (Source: CFBD)

Wave direction	Return Period (years)						
	1	2	5	10	50	100	200
North	2.70	2.90	3.15	3.31	3.66	3.8	3.92
Northeast	4.18	4.56	5.07	5.47	6.42	6.84	7.27
Southeast	3.93	4.24	4.63	4.91	5.48	5.71	5.92
South	3.74	4.09	4.53	4.84	5.49	5.74	5.98

Table 3-3: Extreme wave height and period estimates at Stonehaven for offshore waves originating from the northeast

Return Period (year)	Hs (m)	Tm (sec)	Return Period (year)	Hs (m)	Tm (sec)
0.2	3.16	10.43	**10**	5.47	12.00
0.5	3.71	11.05	**20**	5.88	12.00
1	4.18	11.45	**50**	6.42	12.00
2	4.56	11.75	**100**	6.84	12.00
5	5.07	12.00	**200**	7.27	12.00

3.3 Data collection

A data collection campaign was undertaken to support the wave analysis. Metocean data was recorded by Partrac Ltd between 20th May 2014 and 24th June 2014 to provide information on the water level, nearshore and channel waves. A summary of the information collected is provided below, and a full description of the wave and water level monitoring provided in Appendix A.

[12] Coastal flood boundary conditions for UK mainland and islands, Project: SC060064/TR3: Design swell waves. Environment Agency / SEPA, Feb 2011.

[13] UKCP09 (2009), Chapter 5: Marine and Coastal Projections (pp 58), UK Climate Projections Science Report.

3.3.1 Wave and water level conditions

A Nortek 1 mHz Acoustic Waves and Currents (AWAC) recording gauge was deployed approximately 1.5km offshore of the River Carron mouth. The gauge recorded wave and water level information, supplied as a 30-minute interval timeseries. Table 3-4 shows wave statistics for the recorded wave conditions, which spanned four weeks. The variation in significant wave height recorded by the wave buoy is presented in Figure 3-1.

Table 3-4: Recorded w ave statistics for monitoring period

Recorded parameter	Significant wave height, Hs (m)	Largest one-tenth wave height, H10 (m)	Peak period, Tp (seconds)
Minimum	0.12	0.13	2.10
Mean	0.64	0.75	6.10
Maximum	2.21	2.71	11.23

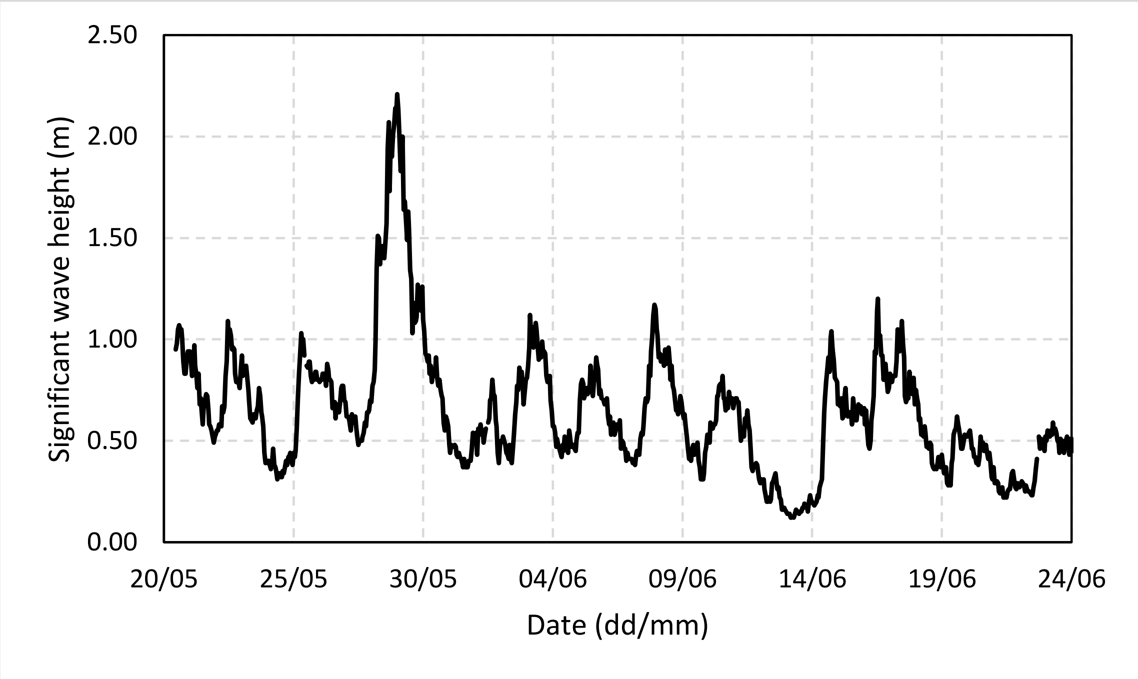

Figure 3-1: Variation in the significant wave height over the monitoring period.

3.3.2 Recorded nearshore water levels

The nearshore water levels were also recorded at the buoy in terms of water depth. Analysis of the recordings indicates that the mean depth at the gauge location was 22.20m. Figure 3-2 shows the variation in water depths over the recording period.

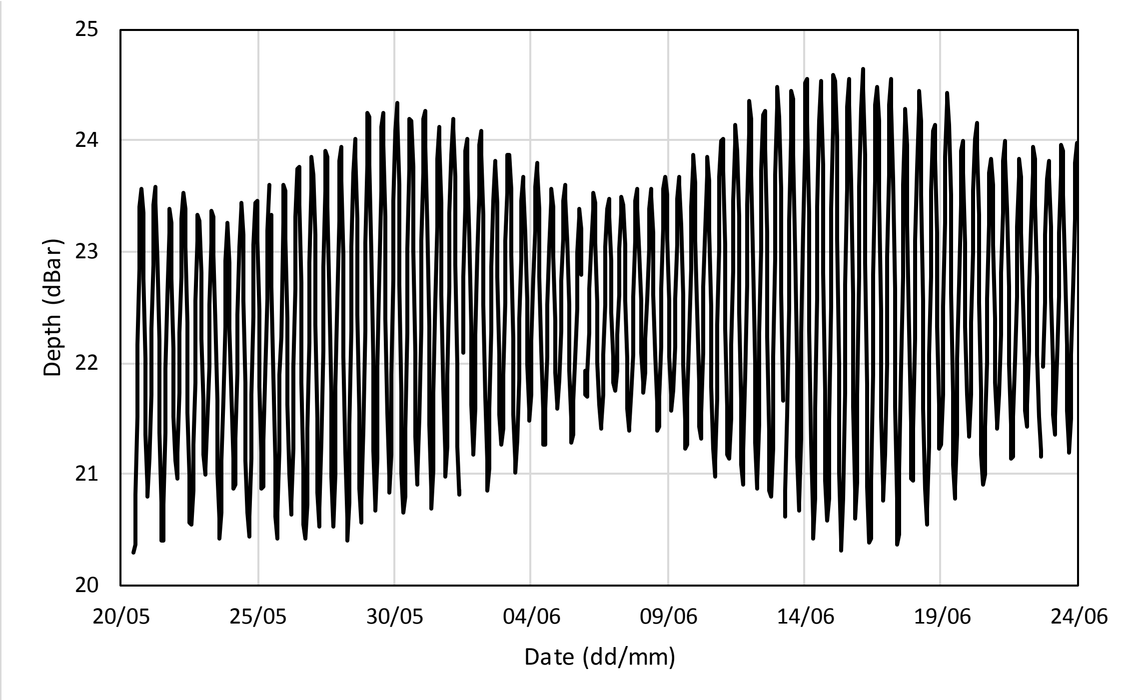

Figure 3-2: Water depth recordings at the wave buoy.

Sea levels at the Aberdeen tide gauge were also obtained from the British Oceanographic Data Centre (BODC) for the duration of the monitoring period. The timeseries of the recorded water level and the recorded surge is shown in Figure 3-3.

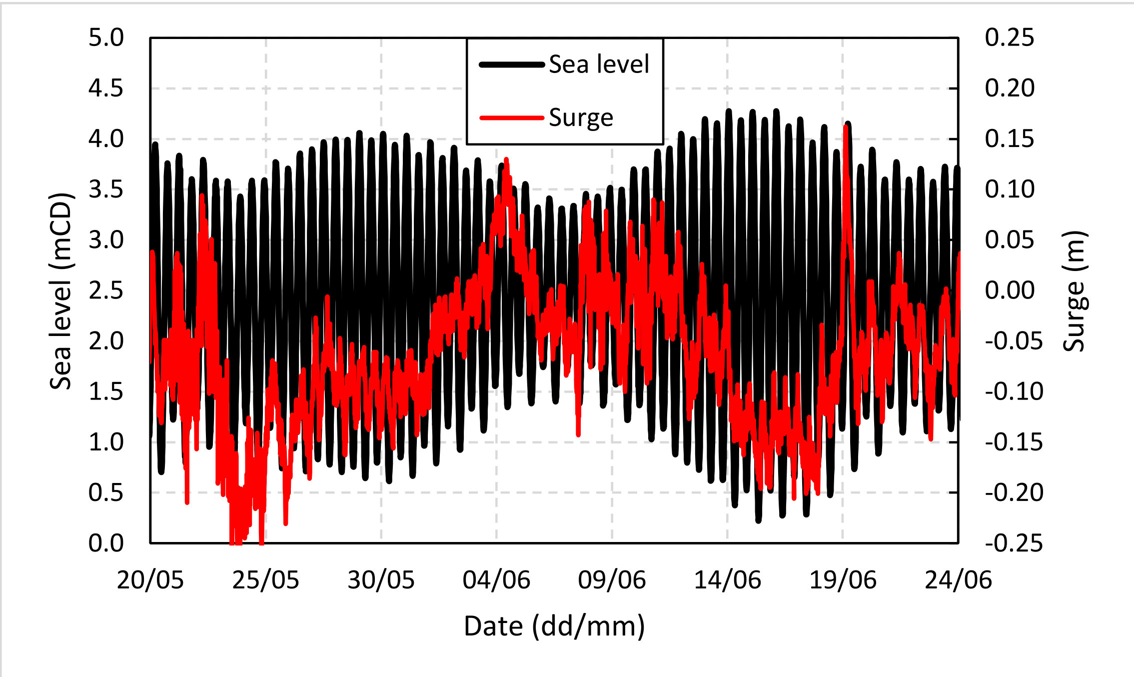

Figure 3-3: British Oceanographic Data Centre tide gauge recordings at Aberdeen.

3.3.3 Recorded channel waves

During the monitoring period a water level gauge was deployed in the River Carron directly upstream of Bridgefield Bridge. During this period the water depth in the channel varied between 0.28m and 0.54m. The peak water level of 0.54m did not coincide with any significant wave event, and is considered to be due to fluvial processes. The largest nearshore wave event captured during the monitoring did not have any effect on the water levels within the river. Based on the data captured, it is clear that there was no wave propagation occurring within the channel during the monitoring campaign. Importantly, this supports the anecdotal information that wave propagation only occurs during high water levels, and is not an everyday occurrence.

4 Quantification of existing wave propagation

4.1 Introduction

To quantify the wave propagation within the River Carron, a number of complex physical and hydrodynamic processes must be understood. The wave conditions experienced within the channel will be influenced by the coastal and fluvial processes, which will include flow, tides, surge, wave height, period and direction. Wave transformation and breaking will be affected by nearshore reefs and headlands, and propagation into the channel will be influenced by the interactions between waves and the rock armour. This chapter describes the methodology used to characterise each of these processes, detailed in the following sections:

- **Approach to quantification:** which summarises the approach used for the assessment, which incorporated numerical modelling to quantify wave conditions and interaction with structures.
- **Wave transformation model and estimate of nearshore wave conditions:** which describes the methodology for estimating the extreme nearshore wave conditions adjacent to the River Carron, which will interact with the river mouth.
- **Boussinesq wave model and consideration of armour efficiency:** which describes the methodology for estimating the efficiency of the existing training walls in reducing wave propagation.
- **Maximum channel wave and water levels:** which considers the potential wave crest elevation under two key scenarios dominated by either coastal or fluvial conditions.

4.2 Approach to quantification

While extreme offshore wave conditions are available at Stonehaven, there is no direct information on the nearshore and channel waves. The nearshore surf zone is a highly dynamic environment, which makes direct measurement of these waves difficult throughout the breaker zone. To quantify channel waves, numerical wave modelling was undertaken. Models were calibrated against a wave recording gauge situated beyond the breaker zone. Unfortunately, there is no single model capable of simulating all the processes occurring as waves propagate into the River Carron. Therefore, a suite of models were used the river channel to capture key wave transformation, breaking and interaction processes.

A spectral **wave transformation model** was developed to assess the wave transformation processes between offshore wave conditions and the nearshore region. The model uses a flexible computational mesh, with a 10m resolution in the nearshore, and includes features such as the nearby headland, the extensive rocky reefs, and up-to-date bathymetry and topographic survey information. This model was calibrated against the nearshore wave and water level recording buoy situated beyond the surf zone.

A **Boussinesq wave model** was developed to investigate how individual waves propagate into the river mouth, and how they transform due to changing channel geometry. The Boussinesq wave model provides an estimation of the changes to waves at the river mouth and other areas where the geometry changes (for example under Bridgefield Bridge). The model has been based on detailed topographic survey, with a spatial resolution of 0.5m.

Finally, wave properties were assessed against flow characteristics which were estimated using a **one-dimensional (1D) flow model**. This included comparing the wave speed against the outflowing currents, and assessing the channel water level during extreme flows.

A schematic of the modelling approach is shown in Figure 4-1, aligned with the key processes identified in Section 2.4. These models are described in more detail within the following sections.

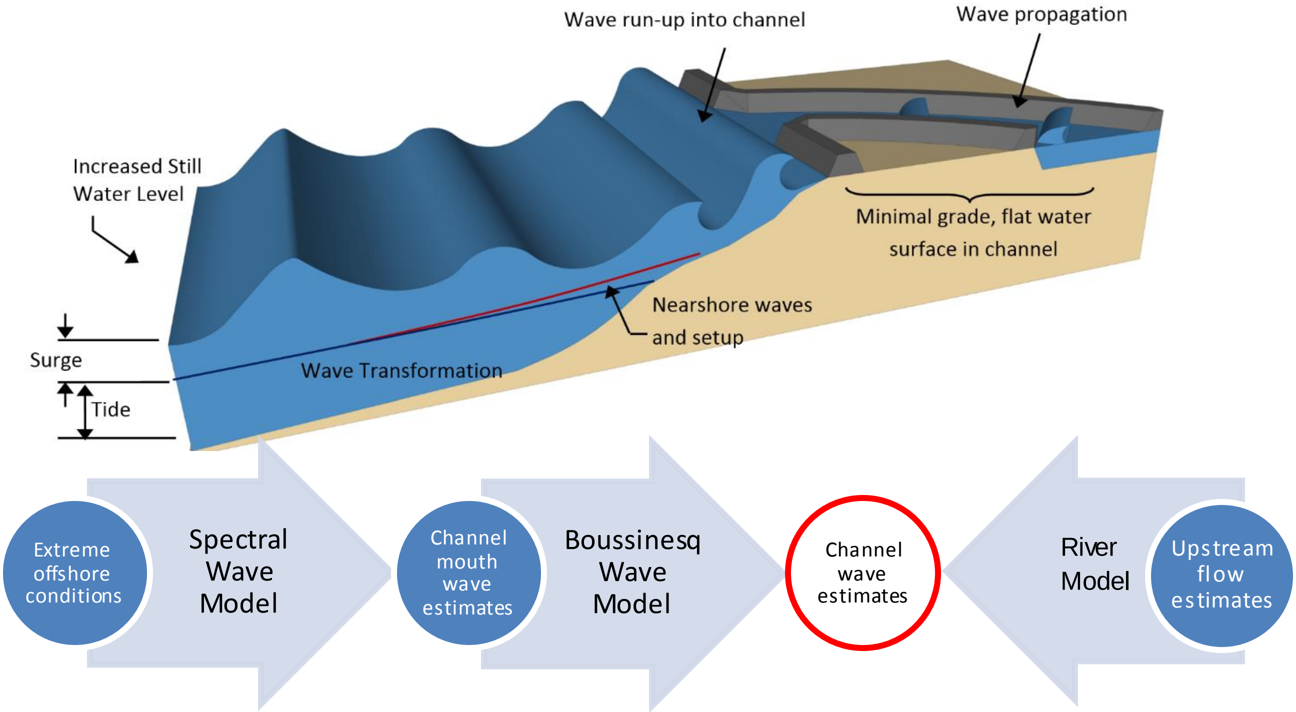

Figure 4-1: Schematic of modelling approach

4.3 Wave transformation model and estimate of nearshore wave conditions

A number of offshore wave and SWL scenarios were simulated using the wave model to determine the worst-case nearshore conditions under design scenarios. These scenarios were developed using a joint probability assessment to determine the likelihood of extreme waves coinciding with extreme SWLs under a range of return periods from 1 in 1 to 200-years. All scenarios were modelled using the industry-standard SWAN (Simulating WAves Nearshore) model. SWAN is a third generation wave model capable of simulating the following nearshore wave transformation processes:

- Wind-wave interactions, which is the transfer of wind energy into wave energy, leading to the growth of waves.
- Shoaling, which is the build-up of energy as a wave enters shallow water, causing an increase in wave height.
- Refraction, which is the change in wave speed as waves propagate through areas of changing depth, causing a change in wave direction.
- Wave breaking, which is the destabilisation of a wave as it enters shallow water, causing broken waves with the characteristic whitewash or foam on the crest.
- Wave dissipation, which limits the size of waves through white-capping, bottom friction and depth-induced breaking.

The wave transformation model was calibrated against wave data recorded between 20th May and 24th June, 2014. The wave model was calibrated using an unsteady (varying water level) approach, which simulated the tidal signature and wave conditions. The calibrated model resulted in a Root-Mean-Square (RMS) error of 0.12m for significant wave height, 0.6s for the mean period and 2.6s for the peak period throughout the simulated period (see Appendix B for a full calibration description).

4.3.1 Joint probability assessment

The wave transformation model was used to calculate nearshore wave conditions based on the available offshore extreme wave conditions and SWL (refer to Section 3.2). These extreme conditions were first conditioned in terms of their joint probability of occurring. The likelihood of two extreme conditions with the same return period coinciding is rare and will have a significantly higher return period (e.g. a 1 in 200-year extreme wave height coinciding with a 1 in 200-year SWL would result in a much higher return period). This is known as the joint probability return period.

This likelihood is determined by the interdependence of the two variables, which is site-specific. Any given joint probability return period can consist of a number of different SWL and wave height combinations.

A number of extreme wave height and SWL combinations were determined for each return period through joint probability analysis. The analysis showed that a 200-year storm event at Stonehaven could consist of a range of wave and water level combinations, such as a 7.82m wave and a 2.49mAOD sea level, or a 2.89m wave and a 3.25mAOD sea level, or a number of combinations in between. Each combination has the same joint probability of occurrence (e.g. 1 in 200 years); however each scenario will result in different conditions at the River Carron mouth, and will interact in different ways.

The joint probability assessment was undertaken using methods described in the Defra best practice guidance[14], with consideration given to recent extreme events and investigations. A key parameter for the joint probability assessment is the level of dependence (ρ) between waves and water levels. Defra guidance suggest a modest correlation along the eastern Scottish coastline ranging between a relatively low correlation of ρ=0.12 and a relatively higher value of ρ= 0.37, with a specific correlation coefficient provided for Aberdeen (ρ=0.21). Variation in the dependence values has a significant impact on the assessment of coastal extremes, with larger coefficients suggesting that larger waves and water levels can occur simultaneously, and lower values suggesting that either extreme sea levels or waves are more likely to occur in isolation.

Recent studies assessed the available extreme wave estimates and the Defra joint probability methodology following the storms in 2012 and 2013[15,16]. The extreme wave estimates were considered to under estimate offshore conditions (for example the offshore waves during the 2012 event were considered far above the 200-year estimates), and the Defra approach underestimates the correlation between wave and SWL conditions. Based on these trends the upper correlation coefficient of ρ=0.37 was adopted for this study to ensure scenarios were not under-predicted. The resulting offshore joint probability wave and sea level combinations shown in Table 4-1 for both present day and climate change scenarios.

Table 4-1: Offshore joint probability combinations of extreme still water levels and offshore wave heights (using a correlation coefficient, ρ = 0.37)

Extreme SWL (mAOD)	Offshore joint probability return period (years)									
Present Day / Climate Change	0.2	0.5	1	2	5	10	20	50	100	200
	Offshore extreme wave heights (m)									
2.49 / 3.16	1.91	2.70	3.50	4.04	4.76	5.31	5.87	6.64	7.22	7.82
2.57 / 3.24	1.56	2.36	3.06	3.60	4.32	4.88	5.42	6.16	6.75	7.35
2.66 / 3.33	-	1.81	2.47	3.02	3.75	4.29	4.84	5.57	6.14	6.73
2.73 / 3.40	-	-	2.04	2.59	3.31	3.85	4.40	5.13	5.68	6.26
2.80 / 3.47	-	-	-	2.15	2.88	3.42	3.97	4.69	5.23	5.80
2.89 / 3.56	-	-	-	-	2.30	2.84	3.39	4.12	4.66	5.21
2.97 / 3.64	-	-	-	-	-	2.40	2.96	3.68	4.22	4.77
3.03 / 3.70	-	-	-	-	-	-	2.52	3.24	3.78	4.34
3.12 / 3.79	-	-	-	-	-	-	-	2.66	3.21	3.76
3.19 / 3.86	-	-	-	-	-	-	-	-	2.77	3.32
3.25 / 3.92	-	-	-	-	-	-	-	-	-	2.89

4.3.2 Nearshore wave conditions

Using the wave transformation model, each of the joint probability scenarios was simulated and the results extracted at the mouth of the River Carron at an elevation of -0.6mAOD (selected as a position directly in front of the river training walls). The resulting nearshore conditions are presented in Table 4-2 and Table 4-3 for present day and climate change conditions respectively. Sensitivity testing was performed to investigate the significance of wind-generated waves along the coastline by considering wind speeds of up to 20m/s from a range of directions. Due to the

[14] 'Defra (2003) 'Joint Probability: Dependence Mapping and Best Practice', Report: FD2308/TR1, Defra/Environment Agency, July 2003.

[15] JBA (2014) UKCMF Factual Report into the Coastal Storms of December 2013 and January 2014 Including Joint Sea Level and Wave Analysis. Undertaken for the Environment Agency

[16] JBA (2014) Stonehaven Coastal Frontage Assessment. Undertaken for Aberdeenshire Council.

depth-limitation of nearshore waves in the shallow nearshore area, the addition of wind results in a negligible change to the wave heights.

Table 4-2: Present day nearshore joint probability combinations of extreme still water levels and nearshore wave heights (using a correlation coefficient, $\rho = 0.37$)

Extreme SWL (mAOD)	Joint probability return period (years)									
Present day water level	0.2	0.5	1	2	5	10	20	50	100	200
	Nearshore extreme wave heights (m) - **present day conditions**									
2.49	1.10	1.27	1.35	1.38	1.41	1.42	1.42	1.43	1.42	1.42
2.57	0.94	1.24	1.35	1.40	1.44	1.46	1.47	1.47	1.47	1.47
2.66	-	1.08	1.29	1.39	1.46	1.49	1.51	1.52	1.53	1.53
2.73	-	-	1.19	1.34	1.46	1.50	1.53	1.56	1.56	1.57
2.80	-	-	-	1.24	1.42	1.50	1.55	1.58	1.60	1.61
2.89	-	-	-	-	1.31	1.45	1.54	1.61	1.63	1.65
2.97	-	-	-	-	-	1.37	1.50	1.60	1.66	1.68
3.03	-	-	-	-	-	-	1.42	1.58	1.64	1.70
3.12	-	-	-	-	-	-	-	1.48	1.61	1.68
3.19	-	-	-	-	-	-	-	-	1.53	1.65
3.25	-	-	-	-	-	-	-	-	-	1.58

Table 4-3: Climate change nearshore joint probability combinations of extreme still water levels and nearshore wave heights (using a correlation coefficient, $\rho = 0.37$)

Extreme SWL (mAOD)	Joint probability return period (years)									
Climate Change water level	0.2	0.5	1	2	5	10	20	50	100	200
	Nearshore extreme wave heights (m) - **including climate change**									
3.16	1.14	1.50	1.67	1.74	1.78	1.81	1.82	1.82	1.82	1.82
3.24	0.92	1.39	1.62	1.71	1.80	1.83	1.85	1.87	1.87	1.87
3.33	-	1.08	1.45	1.64	1.77	1.84	1.88	1.91	1.92	1.92
3.40	-	-	1.22	1.51	1.73	1.82	1.89	1.93	1.95	1.96
3.47	-	-	-	1.28	1.64	1.77	1.87	1.94	1.97	1.99
3.56	-	-	-	-	1.38	1.65	1.80	1.93	1.99	2.02
3.64	-	-	-	-	-	1.44	1.71	1.88	1.98	2.03
3.70	-	-	-	-	-	-	1.51	1.81	1.92	2.02
3.79	-	-	-	-	-	-	-	1.60	1.83	1.95
3.86	-	-	-	-	-	-	-	-	1.65	1.88
3.92	-	-	-	-	-	-	-	-	-	1.71

4.3.3 Estimation of 15 December 2012 nearshore wave conditions

In addition to the extreme joint probability simulations, the wave transformation model was used to estimate the nearshore conditions during the December 2012 event at the exact time the large waves were photographed within the River Carron (see Figure 2-3). Based on photograph metadata, the waves were observed at 14:00hrs. These conditions were simulated based on historic offshore wave conditions from the WWIII model operated by the Met Office, and the sea levels recorded at Aberdeen Harbour by the BODC. The nearshore waves were extracted at the mouth of the River Carron at 14:00hrs which had the following characteristics:

- Water level = 2.55mAOD
- Elevation = -0.61mAOD
- Depth = 2.94m
- Significant wave height: 1.45m
- Peak period = 14.35s
- Spectral period = 12.14s.

Based on the present day return period estimates provided in Table 4-2, the estimated joint probability return period for nearshore waves at this time (e.g. 14:00hrs) was 1 in 10-years. This estimate relates to this time only, and it is possible that larger waves may have occurred that were not captured in photographs, for instance during the peak tide.

4.4 Boussinesq wave model and consideration of armour efficiency

While the spectral wave model captures the regional coastal processes due to headlands and reefs, it does not have the capability to consider the interactions between individual waves and structures in detail. Instead, after the calculation of nearshore wave conditions, a Boussinesq wave model was used to consider the effect of the training walls and channel geometry on individual waves propagating upstream. The MIKE 21 Boussinesq Wave (BW) model was utilised for this study, which is based on the numerical solution of time domain formulations of Boussinesq type equations. Boussinesq models are numerical analogues of physical models and provide detailed information on the combined effects of all important wave phenomena at the study site including refraction, shoaling, diffraction, partial reflection and nonlinear wave-wave interaction.

The BW model outline is presented in Figure 4-2. The model was developed to cover the nearshore region including the SWAN output location, and the model domain was rotated 25 degrees from true north to align the model boundaries to the dominant nearshore swell direction. The model extends throughout the river channel upstream to the White Bridge, with model bathymetry based on topographic survey information. The River Carron channel layout represents the surveyed channel banks, constrictions, bridge abutments and rock armour.

While the BW model is considered a state-of-the-art model for the simulation of complex wave processes, it is important to note that there are inherent limitations in terms of such models. For instance, the model cannot simulate variable water levels (e.g. moving tides), water level gradients or fluvial flows within the River Carron. Furthermore, the model was only able to simulate waves up to 1m in height within the available stability criteria. Therefore more extreme conditions were scaled upwards through a post modelling exercise, as discussed below. As a result of these limitations, and as appropriate in all complex modelling studies, the model results have been used in conjunction with a wider range of supporting information (e.g. anecdotal reports, photographs, surveys, etc.) to estimate wave conditions.

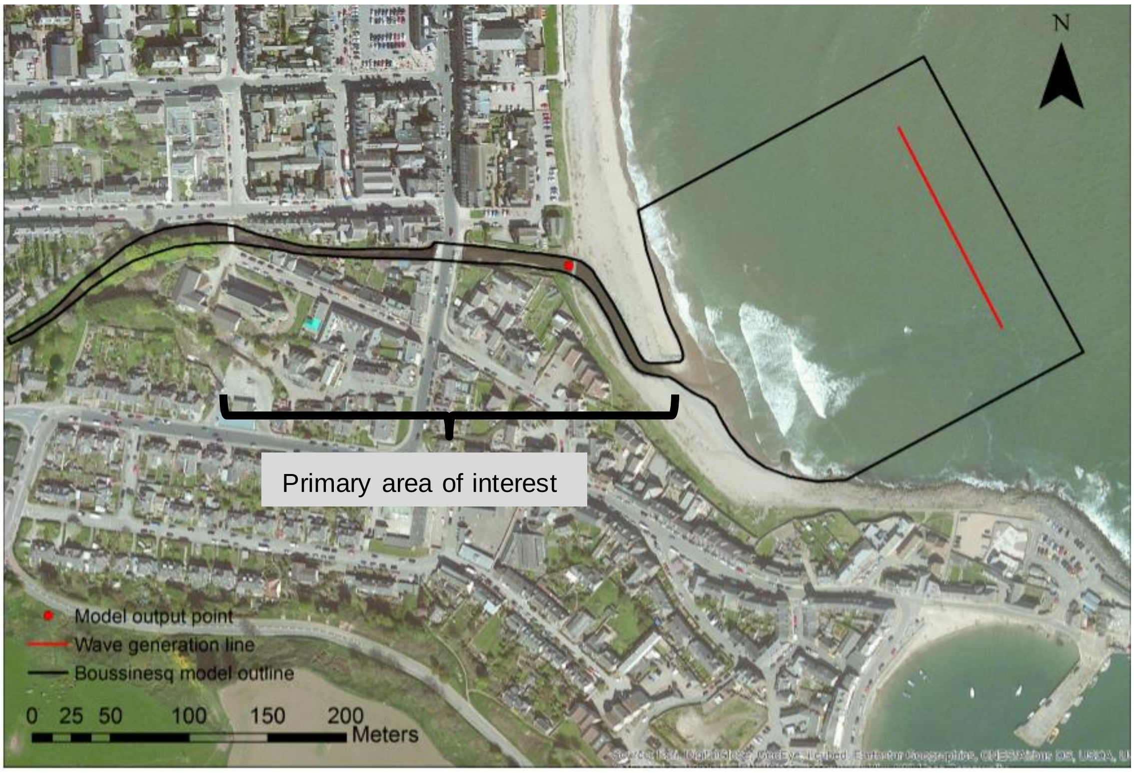

Figure 4-2: Outline of Boussinesq wave model

4.4.1 Model simulations

The BW model was used to simulate an irregular wave train with a significant wave height of 1.0m propagating towards the training wall, with an extreme 200-year (plus climate change) sea level of 3.92mAOD. The changes to the wave properties (e.g. shape and wave height) were simulated throughout the River Carron channel as they undergo transformation, breaking and other non-linear processes, and expressed in terms of the its $Hmo/Hmo_{nearshore}$ ratio. This ratio compares the channel wave height (Hmo) at any point in relation to the nearshore wave height ($Hmo_{nearshore}$).

For instance, a value of 0.5 equates to a 50% reduction of the incident wave height, and values larger than 1.0 indicate an increase. This ratio was then used to scale larger wave events, for instance for a 1.5m or 1.7m wave (the largest 10-year and 200-year nearshore wave respectively). Figure 4-3 and Figure 4-4 show the changes to a 1m wave as it propagates through the channel.

This approach assumes that larger waves will follow a similar trend of wave transformation. While sensitivity testing shows a reduction to the $Hmo/Hmo_{nearshore}$ ratio for larger incoming waves, the trend did not allow certainty in the results. Therefore the use of the 1m incoming wave ratio was used as a conservative estimate of in-stream wave conditions.

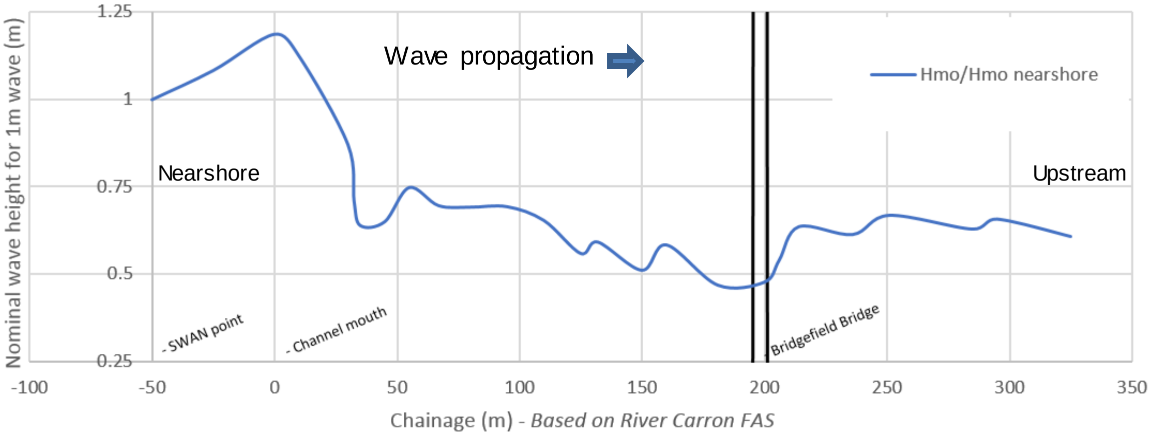

Figure 4-3: $Hmo/Hmo_{nearshore}$ ratio showing changes to wave height for nominal 1m wave

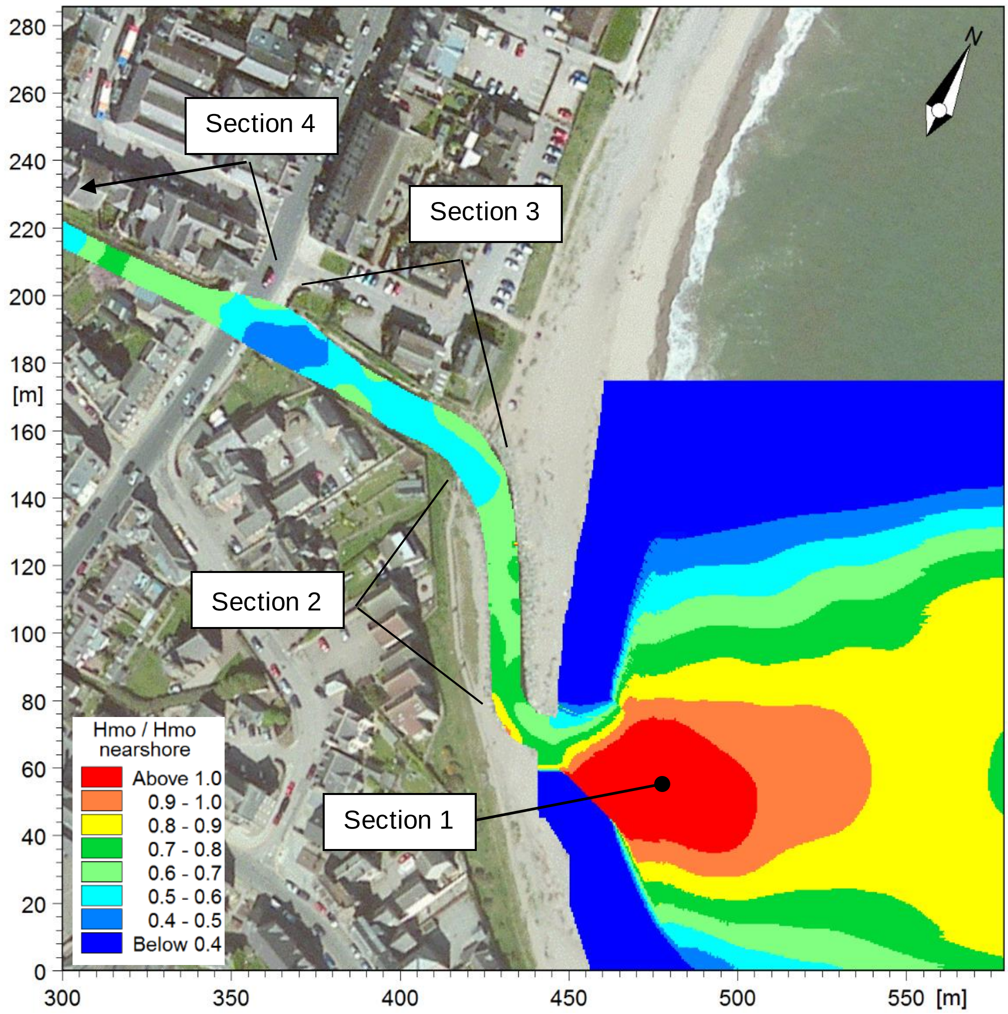

Figure 4-4: Mike 21 BW model validation simulations showing changes to a 1m wave height ($Hmo/Hmo_{nearshore}$)

4.4.2 Model validation

During the calibration period, a high frequency water level recorder was placed within the River Carron to record variations in water level due to either flow or waves. However, during this period no waves were recorded, and as such the BW model could not be formally calibrated. Instead the model was validated against common trends of the channel waves based on observations, anecdotal information and photographs, and supported by estimated conditions during the 15 December 2012 coastal event.

Validation against observed trends

The $Hmo/Hmo_{nearshore}$ ratio presented in Figure 4-3 and Figure 4-3 has been used initially to assess the model simulation against anecdotal information. The following trends were simulated by the model:

- Section 1 (mouth): As waves approach the river mouth they initially shoal (increasing wave height), and then break (decreasing wave height). As they pass into the channel they are funnelled into the channel entrance. There is a sudden increase in wave height as waves are squeezed together and interact with both the structure and themselves due to the partial reflection.
- Section 2: There is a gradual decrease in wave height as waves propagate along the relatively deep, straightened rock armour channel.
- Section 3 (downstream of Bridgefield Bridge): There is a small increase in wave height around the sides of the 130^{o} channel bend, followed by a decrease as the channel widens slightly downstream of the bridge.
- Section 4 (upstream of Bridgefield Bridge): There is an increase in wave height as waves propagate under the road bridge, followed by a sudden increase as the River narrows into the canalised section.

Many of the modelled trends have been observed in the channel. Importantly, the model captures the squeezing of waves into the channel mouth and the sudden increase in wave height following Bridgefield road bridge (refer to anecdotal observation in Section 2.3.2). Further analysis of the BW model results show that as waves propagate along the channel they transform from a sinusoidal shape into a soliton-like shape. This transformation is consistent with anecdotal observations and photographic evidence. Under these conditions the maximum water levels were found to be the sum of the wave height plus the water level, i.e. Hmo + SWL[17]. This trend is consistent with the classical definition of a soliton wave, where the entire displacement of water is above the water level (e.g. there is no preceding wave trough).

Validation against December 2012 images

Using the December 2012 nearshore wave and SWL (refer to Section 4.3.3), the channel wave height and water level were estimated and compared to available photographs. Using the $Hmo/Hmo_{nearshore}$ ratio and the nearshore wave estimate of 1.45m, the simulated significant wave height upstream of Bridgefield Bridge is 0.97m for the December 2012 event. This shows a good match with the observed wave conditions (both photographs and anecdotal reports), which were estimated to be around 1.0m.

The maximum wave crest elevation was calculated as the sum of the wave height and the SWL recorded at Aberdeen (2.55mAOD)[18]. Upstream of Bridgefield Bridge, the simulated maximum wave crest was 3.51mAOD, which shows a good match to the observed maximum level of 3.6mAOD (refer to Section 2.3.2).

While not a formal calibration, in the absence of any wave measurements along the channel, the performance of the BW model is considered satisfactory to support the investigation of wave propagation within the channel. Further confidence could be gained following additional monitoring information or through physical modelling.

[17] This moves away from small amplitude wave theory where the peak water level is the addition of half the wave height (the wave amplitude) plus the SWL.

[18] No translation has been made to account for tide differences at Stonehaven, which is expected to be limited to +-0.1m.

4.5 Maximum channel wave and water levels

The peak water levels within the channel have been estimated under two different scenarios, each with an overall joint probability of 1 in 200-years[19]. The two scenarios consider coastal dominated events (e.g. large waves and SWL) and a fluvial dominated event (e.g. high flows). Each scenario is summarised by its component return-periods in Table 4-4, and are described in the following sections.

Table 4-4: Channel waves scenarios

Scenario	Processes	Return Period (years)		
		Waves	SWL	Fluvial flow
Scenario 1	Coastal dominated, moderate river flow	1-yr to 200-yr (joint probability) inc. climate change*	1-yr to 200-yr (joint probability)* inc. climate change	1-yr inc. climate change
Scenario 2	Fluvial dominated, moderate waves	1-yr (inc. climate change)	1-yr (inc. climate change)	200-yr inc. climate change
*Scenario 1 includes each 200-year joint probability combination shown in Table 4-2 and Table 4-3				

4.5.1 Scenario 1 - Coastal dominated events with moderate flows

Scenario Reference	Processes	Return Period (years)		
		Waves	SWL	Fluvial flow
Scenario 1	Coastal dominated, small flow	1-yr to 200-yr (joint probability) inc. climate change*	1-yr to 200-yr (joint probability)* inc. climate change	1-yr inc. climate change

During an extreme coastal event, extreme waves and SWL are the dominant factor causing inundation to low lying areas such as the River Carron. However, the storm conditions that create these conditions can also lead to rainfall and high flows which may increase risk to estuarine locations, such as the River Carron.

This scenario was developed to consider a joint probability 200-year coastal event coinciding with a moderate river level, including climate change impacts to 2115. A range of 200-year coastal events were considered based on the joint probability combinations shown in Table 4-2 and Table 4-3. There was difficulty in establishing an appropriate fluvial water level, as standard joint probability assessments cannot consider three parameters (e.g. waves, SWL and flow). Instead a second joint probability assessment was undertaken between SWL and flow, which indicates that during 200-year SWL conditions a concurrent 1-year flow is expected. For the River Carron, this flow is considered to be 6.3m^3/s under climate change conditions.

Channel wave heights through the channel were calculated using the scaled $Hmo/Hmo_{nearshore}$ ratio. The largest channel wave heights were observed to be due to larger SWL than extreme offshore waves, as the increased SWL controls the maximum height of the nearshore waves because of depth limitation processes.

The wave height was added to the fluvial water levels, which were calculated using an Infoworks RS 1D hydraulic model developed by JBA for the FAS[20]. The model layout represents the proposed FAS defences (i.e. it includes proposed wall and parapet levels) and was run in an unsteady state. The fluvial water level, wave height and resulting wave crest elevation is shown in Table 4-5. A key assumption of this has been that the flows have negligible effects on wave transformation and continue to propagate un-affected.

[19] The best-practise Defra joint probability approach does not allow for the consideration of three independent variables (e.g. waves, SWL and flows cannot be assessed holistically). This has been addressed by developing a variety of credible combinations of offshore conditions for each joint probability return period, by estimating the magnitude of the third parameter in each scenario.

[20] JBA Consulting (2012) Stonehaven River Carron Flood Alleviation Study, prepared for the Aberdeenshire Council.

Table 4-5: Modelled channel wave height and still water level scenarios for an extreme coastal event with coincident 1-year flows, and estimated peak wave crest level (mAOD) based on a solitary wave form

Cross-section[21] and chainage (m from outlet)	Channel water level (mAOD) / Scaled channel wave height (m)	**Worst case, climate change maximum wave crest level (mAOD)**
CAR_000 (outlet)	3.92 / 1.7	**5.6**
CAR_040	3.91 / 1.1	**5.0**
CAR_117	3.93 / 1.1	**5.0**
CAR_126	3.95 / 1.0	**4.9**
CAR_132	3.95 / 1.0	**5.0**
CAR_169	3.95 / 1.0	**4.9**
CAR_196 (Bridgefield Br.)	3.97 / 0.8	**4.8**
CAR_214	3.98 / 1.0	**5.0**
CAR_221	4.00 / 1.0	**5.0**
CAR_236	4.00 / 1.0	**5.0**
CAR_295	4.05 / 1.1	**5.1**

4.5.2 Scenario 2 - Fluvial dominated events with moderate waves

Scenario	Processes	Return Period (years)		
		Waves	SWL	Fluvial flow
Scenario 2	Fluvial dominated, small waves	1-year (inc. climate change)	1-year (inc. climate change)	200-year inc. climate change

Scenario 2 was developed to consider an extreme 200-year river flow coinciding with moderate wave and SWL conditions, including climate change impacts to 2115. The fluvial conditions have been based on a joint-probability assessment between flow and downstream SWL. As with Scenario 1 there was difficulty in establishing the magnitude of the third parameter, in this case the wave conditions. The parameters used for the scenario included a 200-year flow (77.9m^3/s), a 1-year SWL (3.37mAOD) and coincident 1-year nearshore wave conditions (1.2m), all including climate change impacts to 2115.

Channel wave heights through the channel were calculated using the scaled $Hmo/Hmo_{nearshore}$ ratio. The channel wave heights were smaller than those in Scenario 1 due to smaller SWL. The wave height was added to the fluvial water levels, which were calculated using the Infoworks RS 1D hydraulic model. The fluvial water level, wave height and resulting wave crest elevation is shown in Table 4-6.

Several assumptions have been made to account for physical processes.

1. Under the scenario the river water levels exceed Bridgefield Bridge soffit level. The impact to wave height through the channel was calculated using the Wiegel (1960)[22] power transmission theory for waves propagating past vertical wave barriers. The theory indicates that an obstruction at the still water level will have minimal effect at reducing the wave height, and structures need to extend sufficiently far into the water column to block all wave energy.
2. It is assumed that the flows have negligible effects on wave transformation, and waves will continue to propagate un-affected. While it is likely that the flow conditions will result in high turbulence, the BW model could not account for these processes and as such they were not included (a conservative estimate). The high flows will also result in fast seaward currents, which were assessed using the Infoworks model and compared against the wave

[21] Cross section reference has been taken from the Stonehaven River Carron and Glaslaw Burn Preferred Flood Protection Scheme Report (JBA Consulting, 2013)

[22] Wiegel, Robert L., 1960, "Transmission of waves Past a Rigid Vertical Thin Barrier," J. of the Waterways and Harbors Division, ASCE, Vol. 86, No. WW1, pp. 1-12.

propagation speed (described in Appendix C). The assessment found that the wave speed is faster than the river currents, therefore it is assumed that waves will overcome channel velocity to propagate upstream during extreme flows.

Table 4-6: Modelled channel wave height and water level scenarios for an extreme fluvial event with coincident coastal conditions, including climate change. Estimated peak wave crest level (mAOD) based on a solitary wave form.

Cross-section and chainage (m from outlet)	Channel water level (mAOD) / Scaled channel wave height (m)	**Worst case, climate change maximum wave crest level (mAOD)**
CAR_000 (outlet)	3.40 / 1.2	**4.6**
CAR_040	3.13 / 0.8	**3.9**
CAR_117	4.15 / 0.8	**4.9**
CAR_126	4.55 / 0.7	**5.2**
CAR_132	4.53 / 0.7	**5.3**
CAR_169	4.57 / 0.7	**5.3**
CAR_196 (Bridgefield Bridge)	4.69 / 0.6	**5.3**
CAR_214	4.99 / 0.8	**5.8**
CAR_221	5.11 / 0.8	**5.9**
CAR_236	5.14 / 0.7	**5.9**
CAR_295	5.47 / 0.8	**6.3**

4.6 Summary and consideration of uncertainty

The assessment of wave propagation provided estimates of the maximum wave crest elevation, calculated as the addition of channel wave heights to fluvial flow levels through the River Carron channel. The maximum wave conditions were based on a combination of anecdotal information, photographs and numerical wave modelling. In particular, a spectral wave and Boussinesq wave model were implemented to evaluate changes to waves propagating within the River Carron. While the in-stream water level recorder did not capture any information for a formal calibration of river wave conditions, the models were found to realistically represent the observed conditions during the December 2012 event (which consisted of a wave height of 1m and a wave crest elevation of approximately 3.6mAOD).

Two scenarios were developed to consider the maximum wave crest elevation that might occur during extreme coastal or fluvial events. Scenario 1 considered a 200-year coastal event coinciding with a moderate river levels, including climate change impacts to 2115. Upstream of Bridgefield Bridge the largest wave heights were considered to be 1.1m. When added to the fluvial river levels the maximum wave crest elevation is considered to be 5.1mAOD.

Scenario 2 considered a 200-year river flow coinciding with moderate wave and SWL conditions, including climate change impacts to 2115. Several assumptions were made under these conditions, as the BW model could not represent turbulence, a sloping hydraulic gradient or Bridgefield Bridge soffit. While the scenario resulted in fast currents, as a conservative estimate waves were assumed to overcome the channel velocity based on an analytical solution of wave speed and the flow model results. Upstream of Bridgefield Bridge the largest wave heights were considered to be 0.8m. When added to the fluvial river levels the maximum wave crest elevation is considered to be 6.3mAOD.

The maximum wave crest elevation for each scenario was compared to the River Carron FAS design water levels, shown in Table 4-7. Proposed flood defences have been based on these design water levels and incorporate an additional 0.45m freeboard.

During this assessment the potential for channel waves to exceed Bridgefield Bridge soffit and impact against the parapet was identified. This is likely to result in waves overtopping the parapet, which then has the potential to flow north or south into the low-lying residential area. It is recommended that the parapet upgrade incorporates a wave return wall to direct overtopped water seaward, away from the bridge.

Table 4-7: Modelled channel wave crest height for various joint probability scenarios. Estimated peak wave crest level (mAOD) based on a solitary wave form.

Cross-section and chainage (m from outlet)	Coastal dominated event with coincident flows, inc climate change (mAOD)	Fluvial dominated event with coincident waves, inc. climate change (mAOD)	FAS design water levels (mAOD)
	200-year + CC worst case coastal event, 1-year flows	200-year + CC flow, 1-year + CC SWL 1-year + CC waves	200yr+CC FW+CM+BR2
CAR_000 (outlet)	5.6	4.6	2.7
CAR_040	5.0	3.9	2.8
CAR_117	5.0	4.9	4.1
CAR_126	4.9	5.2	4.5
CAR_132	5.0	5.3	4.5
CAR_169	4.9	5.3	4.5
CAR_196 (Bridgefield Br.)	4.8	5.3	4.6
CAR_214	5.0	5.8	4.9
CAR_221	5.0	5.9	5.1
CAR_236	5.0	5.9	5.1
CAR_295	5.1	6.3	5.4

4.6.1 Uncertainty in results

There remains uncertainty in the wave estimates due to several factors. These are summarised below:

1. The modelling approach. Unfortunately, there is no single model capable of simulating all the processes occurring as waves propagate into the River Carron. While the BW model is considered state-of-the-art for the simulation of complex wave processes, it is important to note that there are inherent limitations in its use in riverine locations. For instance, the model cannot simulate variable water levels (e.g. moving tides), water level gradients or fluvial flows within the River Carron. Furthermore, the model was only able to simulate waves up to 1m in height within the available stability criteria. Therefore more extreme conditions were scaled upwards through a post modelling exercise. As a result of these limitations, and as appropriate in all complex modelling studies, the model results have been used in conjunction with a wider range of supporting information (e.g. anecdotal reports, photographs, surveys, etc.) to estimate wave conditions.
2. The interaction of channel waves and high flows. The influence of flow on wave propagation was considered using small-amplitude wave theory, which indicates wave speed will overcome the channel flow velocity. As a conservative estimate it was assumed that during these conditions waves will not experience any energy loss and will continue to propagate upstream. Additional certainty can be gained in this area by reinstalling the in-stream channel water level gauge, and capturing a wave event during high flow conditions.
3. The maximum wave crest level. The maximum wave crest level has assumed waves propagated as solitons, or solitary waves. This is based on a combination of photographic information and the BW model trends, with the peak water surface calculated as the sum of the wave height plus water level, i.e. Hmo + SWL[23]. Additional certainty on this point can be gained by capturing actual wave data using an in-stream water level gauge.
4. Joint probability assessments. There is uncertainty in the industry as to whether the best-practise Defra methodology accurately predicts the coincidence of extreme wave and SWL

[23] This moves away from small amplitude wave theory where the peak water level is the addition of half the wave height (the wave amplitude) plus the SWL.

processes. Furthermore, this methodology does not allow the assessment of three parameters as required in this study (e.g. flows, waves and SWL). New methods are currently being developed that can increase the reliability of joint probability assessments for coastal applications, based on the application of Heffernan and Tawn[24]. Additional certainty can be gained in this area by adopting the methodology to revise joint probability estimates.

Each of these assumptions adds to the uncertainty behind the maximum wave crest estimate. It is recommended that these elements are taken into consideration when establishing a freeboard level for any proposed defences.

[24] Heffernan, J.E., Tawn, J.A., 2004. A conditional approach for multivariate extreme values (with discussion). J. R. Stat. Soc. Ser. B Stat Methodol. 66 (3), 497–546.

5 Training wall improvement assessment

5.1 Introduction

In the previous chapter, it was shown that the orientation of the existing training wall allows wave propagation past Bridgefield Bridge, and that once in the channel, wave crests have the potential to exceed the River Carron FAS design water levels. In this chapter, different options to mitigate against wave propagation have been assessed, and compared to the existing arrangement. The following options were assessed as part of this study:

1. An option to construct a curved northern training wall and incorporate several short groynes to the south of the River mouth, as identified within a previous report (refer to Section 2.2.3).
2. A detached nearshore breakwater.
3. An extended rock armour training wall.

In addition the expected change to channel waves was assessed for a straight-channel scenario, however it is not expected that this will decrease channel waves.

4. Straight channel

The ability of each design option to reduce channel wave propagation was evaluated to identify a preferred design. This preferred design was assessed further in Chapter 6 to develop engineering plans and a cost estimate of construction.

5.2 Efficiency of existing training wall

Based on site inspections and topographic information, the existing training walls appear to be constructed of rock armour undersized to meet a 200-year storm event, which may lead to damage and deformation during extreme conditions. However, this cannot be verified without information of the existing rock grading. In addition, available topographic survey information show that the defence crest is below a 200-year SWL, which will allow water levels and waves to directly break over the defence. This it is recommended to be upgraded in terms of armour size and crest elevation to minimise wave propagation. To assess the existing training walls, it is assumed that this upgrade has been completed, and the crest elevation is increased to a level above the 200-year SWL.

The BW model was used to assess the efficiency of the upgraded training wall, i.e. assuming the training wall will not deform during an extreme storm or be overwashed, considered in terms of its ratio of channel wave height to nearshore wave height (its $Hmo/Hmo_{nearshore}$ ratio). This was estimated to be 0.67 (refer to Figure 4-3) , meaning that during an extreme event if a nearshore wave of 1.0m propagated towards the existing training walls, a wave as large as 0.67m is expected upstream of Bridgefield Bridge. This location was selected as it is within the proposed River Carron FAS area, and will have a direct impact on flood levels.

This existing $Hmo/Hmo_{nearshore}$ ratio was used as a base-case and compared against each proposed option to compare the changes to channel wave conditions.

5.3 Option 1 - Curved northern training wall

Previous reports[25] propose a number of alternative training walls, primarily as a method of controlling beach sediment transport. Based on the outcomes from these assessments, the most effective solution for beach management was a curved northern training wall in conjunction with downstream rock groynes which will stabilise the southern beach.

Based on available sketches, this option includes a restricted channel width at the downstream footbridge. Whilst this may limit upstream wave propagation, it may also affect the maximum flow of the Carron in flood conditions. As any future solution to wave propagation must not have a negative impact on river flows, this design has been widened to the existing channel width within the BW. The scenario is based on the assumption that the beach has stabilised to the seaward

[25] Stonehaven seawall, Aberdeenshire - Feasibility study of improvements (HR Wallingford, 1998)

end of the short groynes. It is noted that this creates a funnelling shape into the channel, similar to the existing conditions.

Using the BW model the option was assessed and reported in terms of its $Hmo/Hmo_{nearshore}$ ratio, which was calculated to be 1.79. This indicates the option would increase channel wave heights. This increase is attributed to the funnelling of waves, and the lack of any direct protection against incoming wave crests.

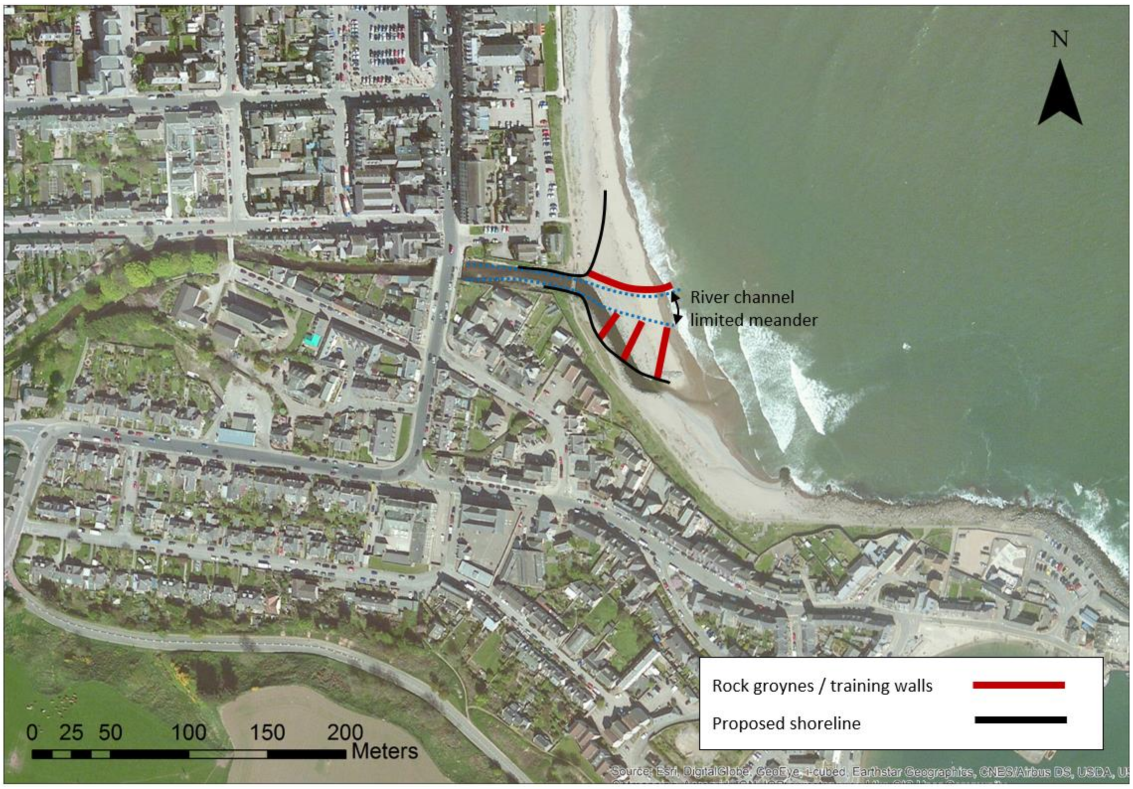

Figure 5-1: Option 1 - Northern training wall and southern groynes.

5.4 Option 2 - Detached breakwater

A nearshore detached breakwater was proposed as a means of directly protecting the river mouth from waves, whilst maintaining the existing training wall arrangement and therefore flow conditions. A number of different breakwater arrangements were proposed, with the final design accounting for several design constraints such as:

- The design of a concave (e.g. horseshoe) shape was not progressed as it could lead to severe wave concentrations in the nearshore, which have the potential to cause severe scour[26].
- The minimum distance from the shoreline was 50m, considered initially to prevent a tombolo forming[27] and having minimal influence on peak flows. If a tombolo was to form in the lee of the breakwater, it may result in heavy siltation of the River Carron mouth potentially affecting flows. Even so, it is expected that some form of salient will form, which will have to be considered in future design stages if this option is taken forward.
- Offshore breakwaters were not considered as they would significantly increase construction costs.

Using the BW model a number of breakwater locations and lengths were assessed in terms of their efficiency. The most efficient design was considered to be a 50m long breakwater, positioned 50m offshore and oriented perpendicular to the dominant wave direction (see Figure 5-2). This design resulted in a $Hmo/Hmo_{nearshore}$ ratio of 0.31, upstream of Bridgefield Bridge. Longer breakwaters were found to be further reduce the channel wave height, however an element of

[26] Refer to British Standard BS 6349 Part 7: Design and construction of breakwaters, Section 2.2.5.

[27] Refer to CIRIA (2007), The Rock Manual: The Use of Rock In Hydraulic Engineering (second edition)

residual wave energy was always found to remain which was able to propagate upstream. The 50m breakwater was selected as it offered the efficient solution, limiting waves to within the 0.45m of the River Carron FAS design water levels, therefore falling within the freeboard allowance. However, if a freeboard above the wave crest was required, the proposed defence crest level would need to be increased.

Figure 5-2: Option 2 - Nearshore breakwater.

5.5 Option 3 - Extended rock armour training wall

A number of changes to the existing training wall alignment were considered to provide additional protection against waves entering the channel. The extent and orientation of the channel modification was limited by the topography, the sewerage conduit and the likely impact on hydraulic and sediment processes. Several design considerations include:

- The channel was not shortened due to the presence of a sewerage conduit close to the existing river mouth. Only extensions to the channel were considered.
- An outlet oriented directly south was not possible as it is likely to direct flows into the shoreline and cause beach erosion and scour patterns. It is also considered that if this was constructed, the natural build-up of sediment would reshape the beach to discharge flows in a more eastward direction (e.g. similar to existing conditions).
- Sharp bends (e.g. 90^{o}) were not considered due to the potential to alter river flow characteristics. Sharp bends may introduce a superelevation of the water surface, which has the potential to alter existing flood levels.
- Potential storm damage was considered based on armour stability. The proposed training wall design has included appropriately sized rock armour to withstand an extreme storm, and has including a roundhead at its seaward end to minimise damage.

Using the BW model a number of extensions, lengths and directions were assessed and reported in terms of its efficiency. The most appropriate design was considered to be a 40m long extension to the rock armour training wall, including a roundhead to stabilise the seaward end of the structure.

Using the BW model the option was assessed and reported in terms of its Hmo/Hmo$_{nearshore}$ ratio, which was calculated to be 1.05. This is a relative increase in wave height from the existing condition, which is considered to be due to the addition of a roundhead (for structural stability), the bathymetry used in the BW model and to the restrictions in redesigning the training wall alignment.

Several causes can be attributed to the worsening conditions. The inclusion of an armour roundhead introduces a smooth entrance to the channel mouth which may funnel waves. The new channel alignment cuts into the existing bathymetry, extending slightly further into the nearshore and towards the southern beach corner. This may allow larger waves to enter the channel due to the greater depth around the new entrance location. Additionally, due to the location of the sewerage conduit any realignment can only straighten the channel further east towards the incoming wave angle, or shift the river mouth to discharge in the same direction. Any further deflection of the outlet to the south (thereby offering more protection) is not considered to be viable in the long-term due to sediment processes, which will naturally re-direct the mouth to discharge towards the sea. Whilst the proposed realignment does introduce an additional bend within the channel, as noted form observed conditions, once a wave enters the channel, there is limited dissipation as it propagates upstream.

Figure 5-3: Option 3 - Extended rock armour training w all.

5.6 Option 4 - Straight channel option

The impact of a straight channel was investigated to consider the significance of the existing channel shape at reducing wave propagation. This option was initially designed as a scenario with no rock armour or training walls, however, based on sediment movement along the beach the complete removal of the training walls is considered to have a detrimental effect. The training walls are designed to stabilise the river outlet, and it is likely that their complete removal could result in the outlet meandering and causing erosion to adjacent infrastructure, or becoming silted up and potentially closing the river, affecting flood levels or creating water quality issues. The current length of the training walls allow a wide beach and stabilises sediment transported from the north. In order to maintain the current beach this option has considered an extended, straight channel.

The channel was extended from the corner of Salmon Lane following an easterly orientation to the shoreline, as shown in Figure 5-4. Using the BW model the option was assessed and reported in terms of its $Hmo/Hmo_{nearshore}$ ratio. This has resulted in a ratio of 1.51, meaning that during an extreme event if a nearshore wave height was 1.0m, it is expected that the waves could be approximately 1.5m upstream of Bridgefield Bridge. This increase is considered to be due to the minimal protection offered the straightened channel, and the subsequent increase in wave height as waves are squeezed into the narrowed upstream channel.

Figure 5-4: Option 4 - Straight channel.

5.7 Summary of training wall assessment

The efficiency of the existing training wall was assessed and new options proposed to minimise wave propagation. Based on site inspections and topographic information, the current design of the existing training wall appears to be constructed of rock armour undersized to meet a 200-year storm event with a crest level below the 200-year SWL. It is recommended that this is upgraded in terms of armour size and crest elevation to minimise wave propagation and structural damage.

Assuming these upgrades are complete, i.e. the training wall will not deform during an extreme storm or be overwashed, the efficiency of the existing training wall has been considered in terms of its ratio of channel wave height and nearshore wave height (its $Hmo/Hmo_{nearshore}$ ratio). This is estimated to be 0.67, meaning that during an extreme event if a nearshore wave of 1.0m propagated towards the existing training walls, a wave as large as 0.67m is expected upstream of Bridgefield Bridge.

A number of potential changes to the River Carron mouth are proposed, incorporating a new wall and rock groynes, a detached breakwater and an extended training wall. Using the BW model each design option was assessed and reported in terms of its $Hmo/Hmo_{nearshore}$ ratio, as shown in Table 5-1. This assessment indicates Options 1, 3 and 4 result in a decreased efficiency compared to the existing case (e.g. larger channel wave heights), with the most effective option considered to be the nearshore breakwater. This option has been further explored in Chapter 6.

It is important to understand the approach used to quantify the efficiency of each proposed option. The assessment has used the BW model only, which focusses on important wave phenomena. However, the model does not consider the effect of varying water levels (e.g. tides), currents, hydraulic gradients due to flows, sediment transport or changes to the bathymetry. As such it presents a snapshot of a single wave event, suitable to comparatively assess each option in terms of its efficiency. It is known that channel bathymetry will also influence the channel wave height, and small changes have the potential to significantly change the upstream wave conditions. A sensitivity analysis was performed in which the width of the entrance channel was increased and decreased by one metre, and the depth was increased and decreased by half a metre. The variations in channel geometry resulted in changes to the upstream wave height between 37% and 75%, showing the sensitivity to channel conditions. It is also important to note that the channel wave heights are dependent on offshore conditions used within the wave/SWL joint probability assessment. Whilst based on the best available data, there is always the potential for larger waves to occur.

Based on these factors, the $Hmo/Hmo_{nearshore}$ ratio should be used with caution when predicting the maximum wave heights to occur from each proposed option. Whilst the results serve as a useful guide to the relative performance of different options, further assessment is required prior to the detailed design of the final engineering solution.

Table 5-1: Efficiency of proposed options in terms of $Hmo/Hmo_{nearshore}$ ratio upstream of the Bridgefield Bridge.

	$Hmo/Hmo_{nearshore}$ ratio	Comments
Existing training wall arrangement	0.67	Base case
Option 1 - Curved northern training wall	1.79	Worsened conditions
Option 2 - Breakwater	0.31	Improved conditions
Option 3 - Extended rock armour training wall	1.05	Minimal change
Option 4 - Straight channel	1.51	Worsened conditions

6 Preferred design and cost effectiveness analysis

6.1 Introduction

The assessment of channel waves undertaken in Chapter 4 shows the potential for wave crests to increase the River Carron FAS design water levels. An options appraisal undertaken in Chapter 5 show that a nearshore detached breakwater would offer the best protection from waves, reducing them by approximately 70%. In this chapter, the cost effectiveness of constructing the breakwater is considered and compared to either raising the River Carron FAS wall height or adopting a *do nothing* option.

The capital costs of each of these options are explored in further detail in the following sections.

6.2 Do nothing

To adopt a *Do Nothing* approach the impact of allowing waves to enter the channel must be understood. It was concluded from the wave crest assessment that waves can potentially propagate upstream during extreme flow events, as their wave speed will overcome the flow velocity and Bridgefield Bridge would not extend sufficiently far into the water column to block all wave energy. Based on modelling undertaken in this study, which could not be verified against recorded channel wave properties, upon entering the channel waves will transform into a soliton shape, with a frequency passing any given point at around three to four waves per minute. Based on captured photographic information these waves will have a long wave crest. This is supported by linear wave theory which results in an estimation of a 16 second wave in 2m water depth having a wavelength of approximately 70m. The overtopping that will result from these waves will be periodic, limited to times of high tides or extreme sea levels, and occurring only as a wave crest propagates upstream.

6.3 Wall raising

The River Carron FAS design wall levels may be increased to cater for the additional wave crests. The wall level would either be designed to include wave crests within the freeboard allowance or to include a freeboard above the wave crest level. The current freeboard allowance for the FAS is 0.45m.

The analysis in Section 4 shows that the maximum wave crest elevation during the River Carron FAS design event is up to 0.8m higher than the design water levels, however, if climate change is included on the downstream boundary the wave crest elevation could be up to 0.9m larger. Anecdotal observations of the channel waves indicate they are able to propagate up to and beyond White Bridge, approximately 300m upstream of the river mouth. The interactions between river gradient, flow and wave propagation do not allow the wave characteristics to be estimated past this point using numerical modelling, and direct measurements are required. However, based on the rising wall heights, and the continuous energy dissipation the waves would experience to this point, White Bridge was used as the upstream limit of wall raising. This will require further analysis during a detailed design phase, which will include an assessment of tie in details and wall transitions.

If waves were included within the FAS freeboard allowance the wall increase would be restricted to 0.4m. If a 0.45m freeboard above the wave crests was required this would increase to 0.9m extension. A high-level cost assessment of the associated construction costs up to White Bridge is presented in Table 6-1, which shows options including or excluding an additional freeboard allowance. The costing is based on unit rates developed in the River Carron FAS design estimates, and include a bias of 60%. The wave crests are based on the worst case 200-year wave estimates (refer to Scenario 3 in Section 4.5.2) which include an extreme 200-year + CC fluvial event, with coincident 1-year + CC SWL and nearshore waves.

The total estimated cost for these options is £1,500,000 for the 0.4m wall raising and £3,230,000 for the 0.9m wall raising.

Table 6-1 Breakdown of costs for raising the walls along the River Carron to the White Bridge

Section	Unit-cost	0.4m wall pro-rata increase	0.4m wall pro-rata value	0.9m wall pro-rata increase	0.9m wall pro-rata value
01-01	£488,600	19%	£92,834	42%	£205,212
01-02	£127,250	12%	£135,270	27%	£304,358
01-03	£1,866,150	17%	£317,246	37.5%	£699,806
04-01	£216,500	24%	£51,960	54%	£116,910
04-02	£824,500	24%	£197,880	54%	£445,230
04-03	£15,000	67%	£10,050	150%	£22,500
Total			£805,240		£1,794,016
Preliminaries & general items (12.5%)			£100,665		£224,252
Total			£905,905		£2,018,268
Optimism bias			60%		60%
Total			£1,449,500		£3,229,000

6.4 Nearshore breakwater

Based on the assessment undertaken in Chapter 5, the construction of a nearshore breakwater will offer the best protection against channel waves. The breakwater design was optimised in terms of its location, orientation and length through an iterative process. In this way, the distance offshore, the angle and the length of the breakwater was varied to identify the layout that offered the best reduction in nearshore energy, whilst minimising the defence footprint.

A number of breakwater positions, lengths and orientations were tested using the BW model. Large breakwaters positioned close to the coastline were found to offer more protection than small breakwaters positioned offshore. However, the final optimised design was developed to consider other practical aspects such as:

- Ease of construction
- Volume of required materials (and therefore cost)
- Health and safety
- Engineering judgement.

This resulted in a preferred design of a 50m long rock armour breakwater positioned perpendicular to the dominant wave direction approximately 50m offshore. The function of the breakwater will be to allow the waves to break on the structure itself, and therefore prevent unbroken wave trains propagating into the channel. However, even after construction it is expected that an element of residual wave energy will remain, which is able to propagate upstream. It is estimated that these waves will be limited to approximately 0.3-0.4m upstream of Bridgefield Bridge, as shown in Table 6-2. If waves were included within the FAS freeboard allowance there would be no need to increase the wall height. However, if a 0.45m freeboard above the wave crests was required this would result in a 0.4m extension.

Table 6-2: Modelled channel wave crest height for fluvial dominated scenarios including a nearshore breakwater. Estimated peak wave crest level (mAOD) based on a solitary wave form.

Cross-section and chainage (m from outlet)	Wave crest elevation, fluvial dominated events (mAOD) including breakwater		FAS design water levels (mAOD)
	200-year + CC flow, 1-year + CC SWL 1-year + CC waves	River Carron FAS: 200-year + CC flow, 1-year SWL, 1-year waves	200yr+CC FW+CM+BR2
CAR_000 (outlet)	3.6	2.9	2.7
CAR_040	3.4	3.1	2.8
CAR_117	4.4	4.4	4.1
CAR_126	4.8	4.7	4.5
CAR_132	4.8	4.7	4.5
CAR_169	4.9	4.8	4.5
CAR_196 (Bridgefield Br.)	4.9	4.8	4.6
CAR_214	5.3	5.3	4.9
CAR_221	5.5	5.4	5.1
CAR_236	5.4	5.4	5.1
CAR_295	5.8	5.8	5.4

The breakwater costs are based on a 3-6tonne armourstone breakwater formed in a double interlocking layer. The breakwater has a 1:2 sloped front face and a 1:1.5 sloped rear face with a 3.6m wide crest with an elevation of at least 4.7mAOD. On the rear side of the embankment, the slope angle has been increased to 1:1.5 to reduce total material quantities. Appendix D provides further information on the breakwater design.

A breakdown of high-level construction costs for the nearshore breakwater is presented in Table 6-3. The total estimated cost is £1,785,000. If the FAS wall height required a freeboard above the wave crests the estimated costs would increase by £1,450,000 (see Table 6-1). The total cost of the breakwater plus increased freeboard option is therefore estimated to be £1,785,000.

Table 6-3 Breakdown of costs for a nearshore detached breakwater

Element	Value
Breakwater Armour	
Length of breakwater	85.800m
Cross sectional area	56.856m^2
Volume of breakwater	5,366m^3
Packing density of rock	2.200t/m^3
Weight of rock	11,805t
Unit price of rock	72.000£/t
Optimism bias	60%
Cost of rock	£1,359,977
Quarry Run Core	
Length of breakwater	85.800m
Cross sectional area	23.410m^2
Volume of core	2,209m^3
Packing density of fill	2.000t/m^3

Element	Value
Breakwater Armour	
Weight of rock	4,419t
Unit price of fill	60.000£/t
Optimism bias	60%
Cost of quarry run core	£424,212
Total cost of structure	£1,784,000

7 Conclusions and recommendations

7.1 Summary

This study was undertaken by JBA, on behalf of Aberdeenshire Council, to investigate the magnitude of wave propagation within the River Carron, Stonehaven. The study had three key aims:

1. To estimate the degree to which wave propagation will increase water levels upstream based on the current armourstone alignment.
2. To assess the efficiency of the current orientation of the armourstone training structures in decreasing the ability of waves to propagate upstream and make recommendations for improvement.
3. To provide an outline design for the alignment of the rock armour to minimise the opportunity for propagation whilst ensuring maximum discharge from the River Carron.

7.1.1 Wave propagation

The assessment of the largest wave crest elevation was based on a combination of anecdotal information, photographs and numerical wave modelling. A spectral wave model was used to assess the wave conditions reaching the river mouth, which was calibrated against a nearshore wave buoy. A Boussinesq wave (BW) model was then used to assess the interaction of waves with the existing training walls, and to simulate waves propagating within the river. While the in-stream water level recorder did not capture any information suitable for a formal calibration of river wave conditions, the model was found to realistically represent the observed trends during the December 2012 event. This consisted of a wave height of 1m and a wave crest elevation of approximately 3.6mAOD.

In order to investigate wave propagation during different combinations of storm-driving conditions, two scenarios were considered to address extreme coastal and fluvial events. These were compared to the design water levels adopted for the River Carron FAS.

Scenario 1 considered a 200-year coastal event coinciding with moderate river levels, including climate change impacts to 2115. Upstream of Bridgefield Bridge the largest wave heights were considered to be 1.1m. When added to the fluvial river levels the maximum wave crest elevation is considered to be 5.1mAOD.

Scenario 2 considered a 200-year river flow coinciding with moderate wave and SWL conditions, including climate change impacts to 2115. Several assumptions were required under these conditions, because the BW model could not represent turbulence, a sloping hydraulic gradient or Bridgefield Bridge soffit. While the scenario resulted in fast currents, as a conservative estimate waves were assumed to overcome the channel velocity, as indicated by the comparison of theoretical wave speed and the flow model results. Upstream of Bridgefield Bridge the largest wave heights were considered to be 0.8m. When added to the fluvial river levels the maximum wave crest elevation is considered to be 6.3mAOD.

The maximum wave crest elevation was compared with the River Carron FAS design water levels, which show the potential for wave crests during the fluvial-dominated event to be up to 0.8m higher, upstream of Bridgefield Bridge.

7.1.2 Efficiency of training wall options

The efficiency of the existing training wall was assessed and new options proposed to minimise wave propagation. The efficiency of the existing training wall in limiting wave propagation is considered to decrease wave heights by approximately 30%.

A number of potential changes to the River Carron mouth were proposed, and assessed in terms of their efficiency to reduce wave propagation. The options of a curved northern training wall, southern extension or a straightened channel were not found to improve conditions.

The most effective option was considered a detached breakwater, which was able to decrease wave heights by approximately 70% of the neashore wave height. Due to the low-lying nature of the river outlet, climate change impacts and the necessity to maintain river hydraulic efficiency, it is not expected that wave propagation could be stopped completely.

7.1.3 Cost effectiveness analysis

Cost effectiveness analysis was undertaken to compare the costs of constructing a nearshore breakwater versus raising the FAS wall height to include an allowance for waves.

Based on a high-level assessment, the costs for constructing a breakwater are estimated to be in the order of £1,800,000. However, after construction it is expected that some wave energy will remain, with small waves still able to propagate within the channel during an extreme event. If a freeboard was required above the wave crests this would result in a 0.4m increase to the wall height (considered up to White Bridge), at an estimated to cost in the order of £1,450,000. The total cost of the breakwater plus freeboard option is therefore estimated to be of the order of £3,250,000.

Alternatively, the occurrence of waves could be mitigated by increasing the height of the FAS walls alone. If a freeboard was required above the wave crests an increase to the wall height of 0.9m would be required. Based on unit-cost estimates up to White Bridge this would cost approximately £3,230,000.

Information relating to the likely wave impact is provided to consider the implications of a *do nothing* approach. It is expected that any overtopping will be periodic, limited to times of high tides and extreme sea levels, and will only occur as a wave crest propagates upstream (e.g. three-four times per minute).

7.2 Uncertainty in results

There is a high degree of uncertainty in the maximum wave crest level estimated in this study due to several factors, as described in Sections 4.6.1 and 5.7. In particular the interaction of different physical processes such as tides, surges, waves, river levels and flows are unable to be simulated in a single numerical model. Instead this study used the outputs of several wave and flow models to best represent the highly complex environment, each introducing a level of uncertainty within the calculations. The models have been used to simulate a sequence of events; first transforming offshore wave conditions to nearshore, then into the river mouth and upstream of Bridgefield Bridge. However, there is uncertainty as to whether the offshore wave conditions accurately represent extreme conditions, and if the Defra joint-probability methodology correctly predicts the coincidence of extreme waves and sea levels. Furthermore, the Defra methodology does not allow for an assessment of three parameters (e.g. flows, waves and SWL) which was a limitation of this study. New methods are currently being developed that can increase the reliability of joint probability assessments for coastal applications, and additional certainty can be gained in this area by adopting such a methodology to revise joint probability estimates.

As a result of this uncertainty, and as appropriate in all complex modelling studies, the model results have been used in conjunction with a wider range of supporting information (e.g. anecdotal reports, photographs, surveys, etc.) to estimate wave conditions.

7.3 Recommendations

Several key recommendations were made to address either the uncertainty in the modelling, or to assist in using the study conclusions in future planning. These include:

1. The options of a curved northern training wall, southern extension or a straightened channel should not be considered further as they were found to offer no improvement to upstream wave conditions.
2. As the construction costs for the breakwater and wall raising options are quite high, the *do nothing* option may be considered the most appropriate until a coastal protection scheme is considered to address more general wave overtopping issues.
3. The assessment shows the potential for waves to reach the soffit of Bridgefield Bridge during high water levels, therefore able to break against the bridge and parapet (which is recommended to be infilled). Any overtopped water will bypass the River Carron FAS, with the potential to flow north or south into the low-lying areas. It is recommended that the parapet upgrade incorporates a wave-return wall to direct overtopped water seaward, away from the bridge.
4. It is recommended that a revised joint probability assessment is undertaken to increase the reliability of nearshore and channel wave estimates, adopting a methodology such as

that proposed by Heffernan and Tawn[28]. This new assessment should be specifically developed to assess the joint probability between waves, coastal still water levels and river flow, and should be conducted prior to any detailed design or construction.

5. Further physical data collection is recommended from within the River Carron channel, which could be used to understand the interaction with waves and flow, and to validate future physical models. Ideally this information will capture a wave event during high flow conditions.
6. The modelling results presented in this report are considered conceptual. As with any numerical models, the results are a simplification of complex physical processes. While the modelling results serve as a useful indicator of the wave trends, it is recommended that detailed physical modelling involving waves and flow is carried out prior to any detailed design or construction.

[28] Heffernan, J.E., Tawn, J.A., 2004. A conditional approach for multivariate extreme values (with discussion). J. R. Stat. Soc. Ser. B Stat Methodol. 66 (3), 497–546.

A Appendix A - Data collection report

(Supplied separately)

B Appendix B - Model calibration

B.1 Wave transformation model

A spectral wave model was developed to evaluate the wave transformation processes occurring as waves propagate towards the shoreline. These nearshore wave characteristics were used to investigate the interactions with the River Carron rock armour, and calculate the propagation into the river channel using a Boussinesq wave model.

B.2 Model setup

All storm scenarios were modelled using the industry-standard SWAN (Simulating WAves Nearshore) model. SWAN is a third generation wave model capable of simulating the following nearshore wave transformation processes:

- Wind-wave interactions, which is the transfer of wind energy into wave energy, leading to the growth of waves.
- Shoaling, which is the build-up of energy as a wave enters shallow water, causing an increase in wave height.
- Refraction, which is the change in wave speed as waves propagate through areas of changing depth, causing a change in wave direction.
- Wave breaking, which is the destabilisation of a wave as it enters shallow water, causing broken waves with the characteristic whitewash or foam on the crest.
- Wave dissipation, which limits the size of waves through white-capping, bottom friction and depth-induced breaking.
- Diffraction, which is the spreading of wave energy behind structures, headlands and islands, which causes waves to change direction.

B.2.1 Computational grid and bathymetry

A computational unstructured grid was developed for Stonehaven Bay as shown on Figure B-1 and Figure B-2. The computational mesh extends approximately 15km offshore, to a location where extreme wave characteristics and Met Office wave forecasts are available are available. The grid resolution ranged from 2000m at the offshore eastern boundary, where depths vary between 50m to 110m and increased towards the study area to ensure a resolution of no less than 10m in the nearshore zone adjacent the mouth of River Carron.

The model bathymetry was generated using three sources. Detailed survey data was used to represent the River Carron, based on field survey. More widespread beach foreshore information was based on topographic survey information provided by the Aberdeenshire Council, which was undertaken in May 2013[29]. The regional bathymetry was obtained from FindMAPS for the study area, based on X, Y, Z survey points derived from surveys undertaken by the Civil Hydrographic Programme, Royal Navy surveys, Centre for Environment, Fisheries and Aquaculture Science (CEFAS) surveys as well as surveys from local port and harbour authorities. This data was combined to produce a seamless Digital Elevation Model (DEM). The computational model boundary and bathymetry at the study site are shown in Figure B-3.

[29] Canterbury City Council (2013) Topographic Baseline Survey Report 2013

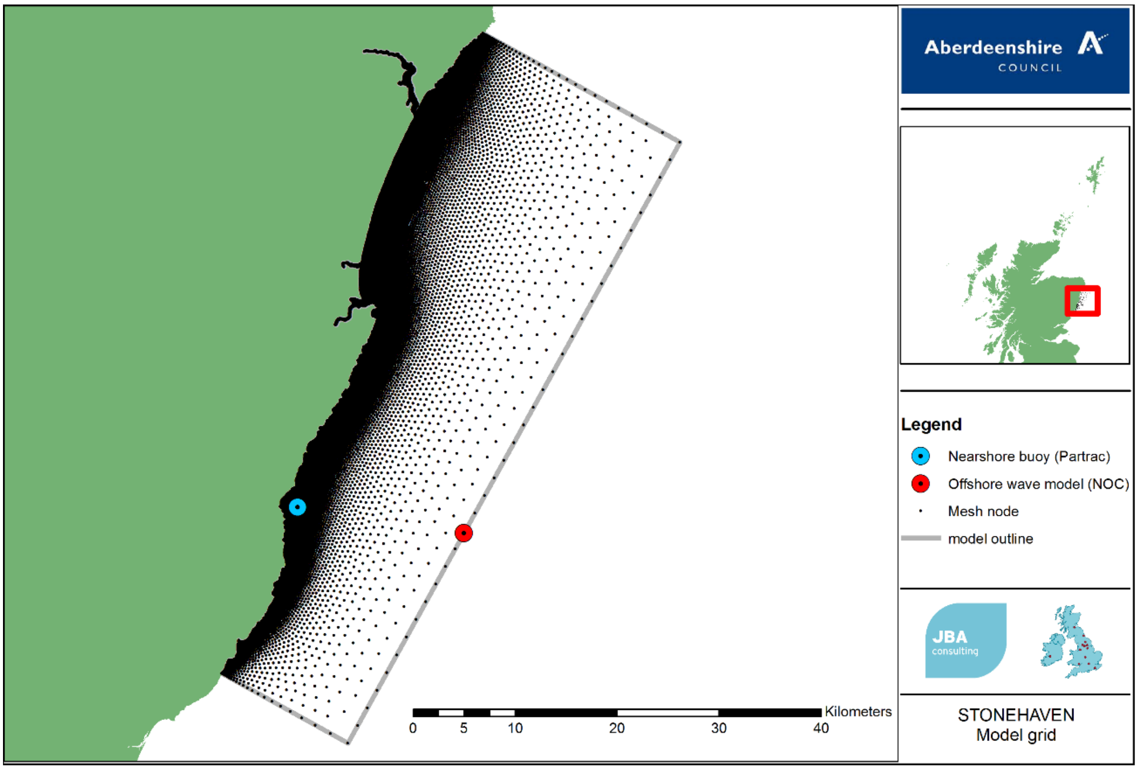

Figure B-1: Model grid and extents, showing offshore Met Office wave point and nearshore wave buoy

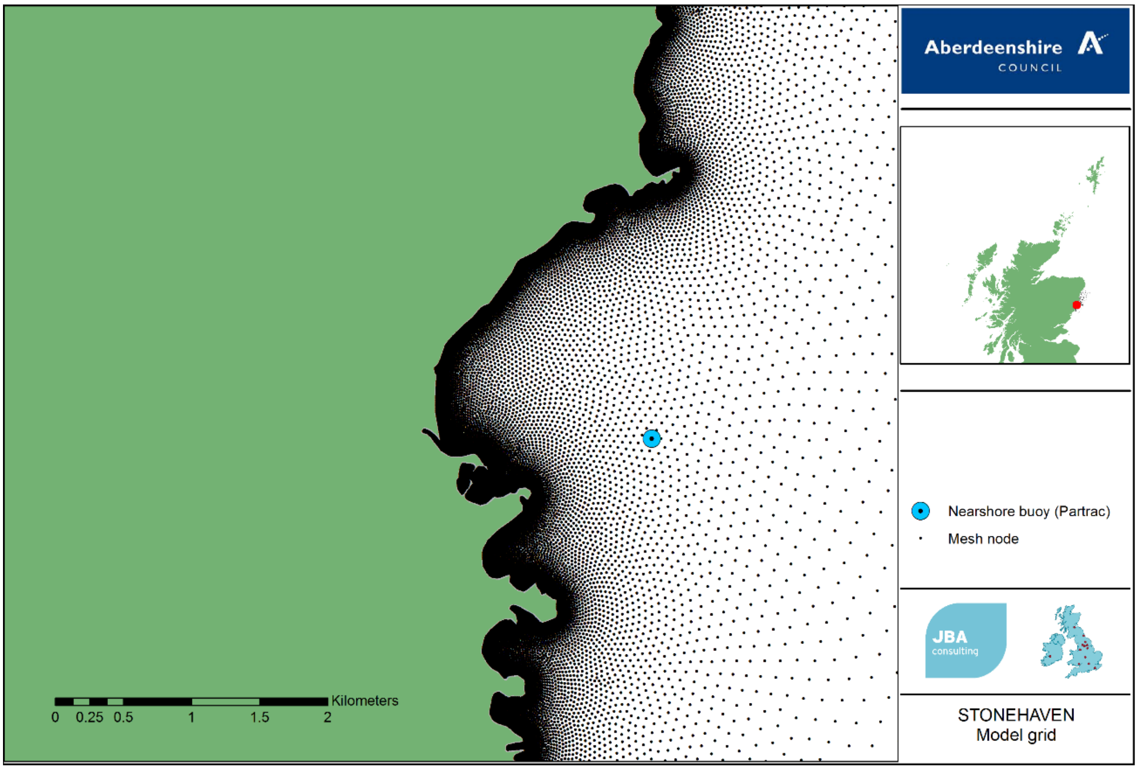

Figure B-2: Location of nearshore wave buoy

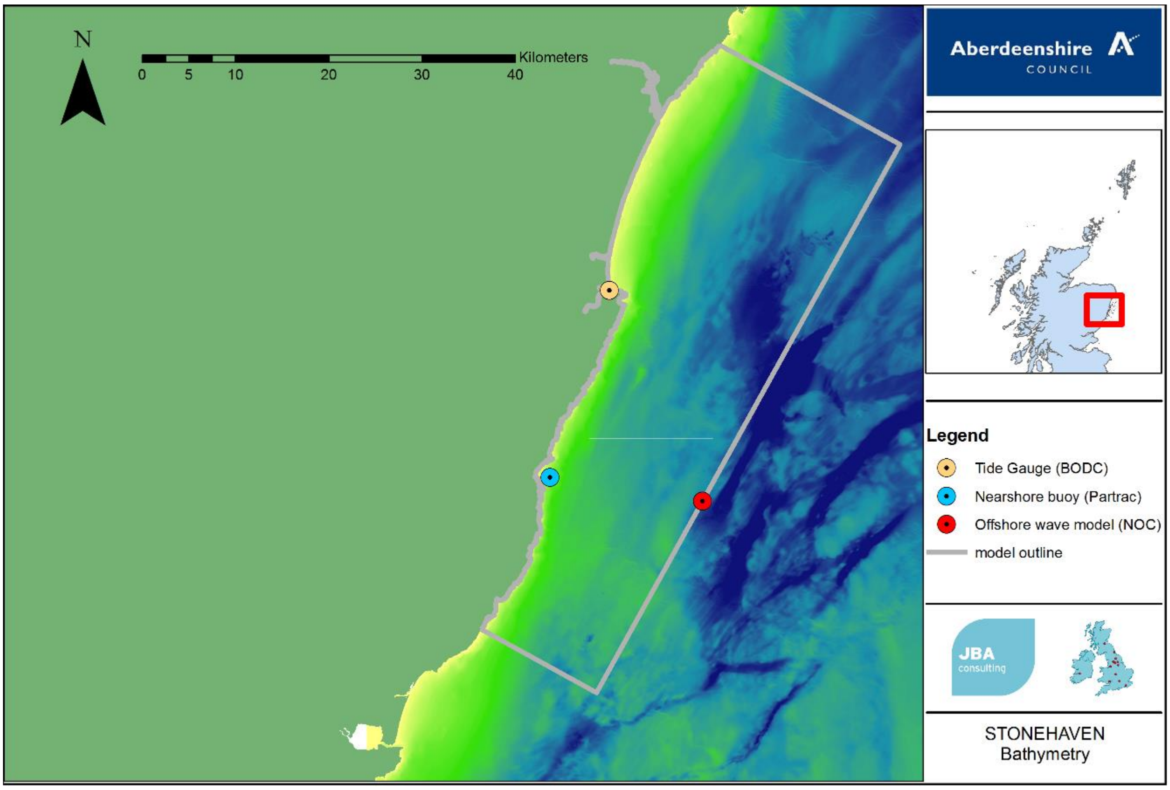

Figure B-3: Model domain with bathymetry showing offshore Met Office wave point and nearshore wave buoy

B.2.2 Model calibration

A long-term record of nearshore or offshore waves is not available at Stonehaven to calibrate a wave model. Instead, specific data was recorded for this project, as described in Appendix A. Nearshore wave data was recorded between 20th May 2014 and 24th June 2014, and was split for use in calibration and validation of the model.

The wave model was calibrated over 10 days of recorded data between 20th May and 31st May. During this period a range of wave heights, periods and directions were observed, ensuring the model domain was capable of representing a broad range of conditions. The model boundary conditions used simulated offshore wave and wind data from the Met Office WWIII model, provided by the Met Office. Tidal water level information was obtained from the British Oceanographic Data Centre (BODC) from the water level gauge at Aberdeen Harbour, approximately 15 miles north of the study site. The offshore conditions were used to force the wave model, which was calibrated against nearshore wave buoy data collected for this study, located 1,500m east of the mouth of River Carron.

The calibration period was simulated using the wave model for nine separate model setups. Each setup consisted of a unique permutation, formed by selecting one of three separate bed friction schemes (Collins, Madsen, JONSWAP) and three white-capping schemes (Komen et al, Van der Westhuysen, Janssen). These parameters form part of the calculation process in the model and are devised from referenced methodologies in the SWAN technical manual[30]. Each approach uses a different calculation to resolve wave growth and dissipation, and by testing each method the most accurate solution can be found for the study site. This was selected by comparing the modelled wave heights against the records from the nearshore wave buoy.

B.3 Results

The best performing scheme was selected based on its performance at the wave buoy. This was achieved using a Collins bed friction scheme and Westhuysen white capping parameters[31]. The calibrated model setup resulted in a Root-Mean-Square (RMS) error of 0.12m for the significant

[30] SWAN, Scientific and technical documentation, Delft University of technology, 2009

[31] Refer to SWAN Scientific and technical documentation for a full description of these parameters.

wave height, 0.6s for the mean period and 2.6s for the peak period throughout the simulated period (see Figure B-4, Figure B-5 and Figure B-6).

The model represented the wave conditions at the buoy very well for the majority of the simulation, although underpredicted the nearshore wave conditions for one event on the 29th May. Several reasons were considered for the discrepancies between modelled and recorded data for this event.

1. The offshore wave boundary conditions supplied by the Met Office are derived from simulated conditions from the WWIII model. Therefore, any inaccuracies in the WWIII model will be transferred through to new model developed for this study. During the majority of the simulation the offshore wave heights were observed to be consistently higher than the nearshore buoy conditions - due to wave transformation and dissipation processes as the waves propagate into the nearshore. However, this is not the case for the May 29 event where the WWIII model indicated that the offshore waves were smaller than the measured buoy data. As the buoy data has been captured using specialist equipment specifically for this study, more faith is given in these nearshore results than the offshore wave conditions, which are subsequently considered to be underpredicted by the WWIII model.
1. The model bathymetry is likely to contain some errors, and this can be associated with loss of detail due to interpolation. There is a discrepancy between the DTM model and the topographic survey at some locations whereby the topographic survey results indicate that bed levels should be lower than reflected by the DTM model. However, the topographic survey does not extend as far as the buoy location.

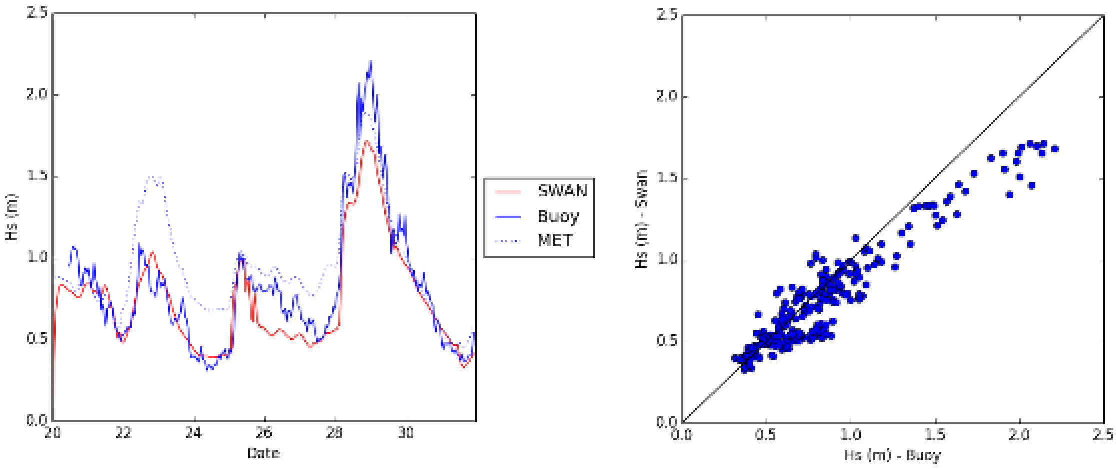

Figure B-4: Significant wave heights as recorded by the buoy and predicted by the SWAN model. Mean RMS Error = 0.12m

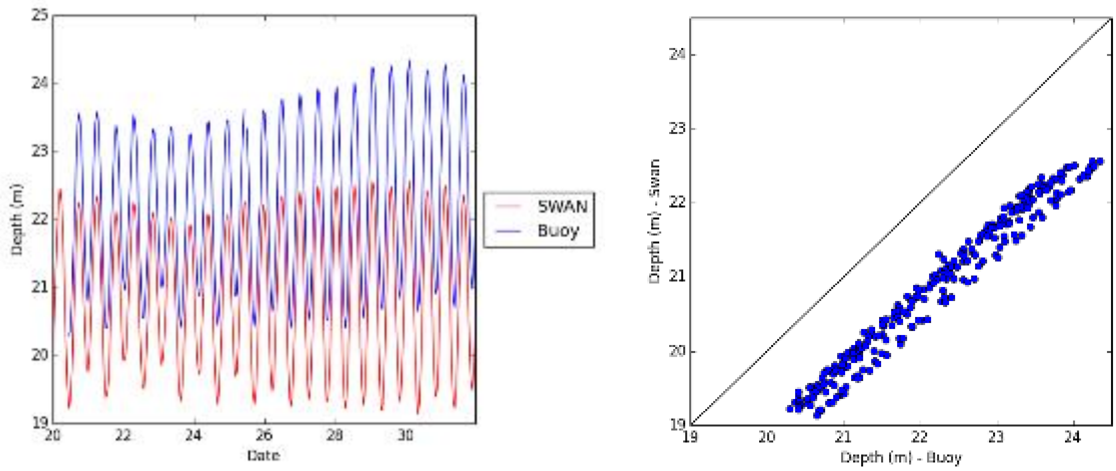

Figure B-5: Water depths as recorded by the buoy and predicted by the SWAN model. Mean RMS Error = 1.35m

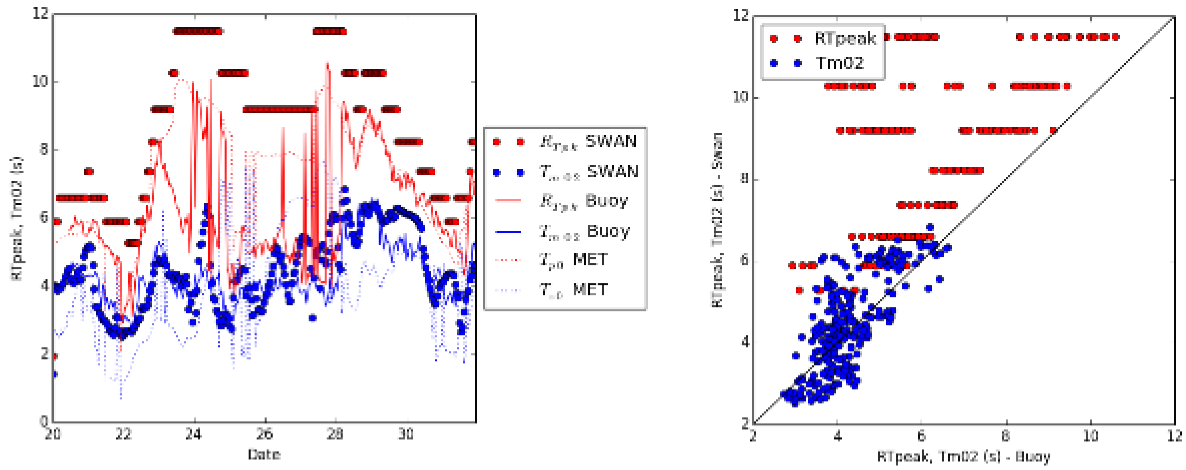

Figure B-6: Peak and mean wave periods as recorded by the buoy and predicted by the SWAN model. Mean RMS Error = 2.6 s and 0.6 s respectively.

B.3.3 Sensitivity testing

Sensitivity testing was undertaken as part of the model calibration to understand the influence of model parameters and boundary conditions. This testing is performed to understand any error that may arise due to difference in the calculated input values and the variations in model results due to various model setups. Using the best performing model setup, the influence of the offshore wave boundary was tested to reflect the potential uncertainty of offshore conditions

Changes to the offshore wave height were made reflecting +/-15% of the supplied WWIII conditions. The wave conditions were compared at the buoy, which resulted in an RMS error of 0.19 and 0.10m. These results indicate a slight improvement to the calibration by using a larger wave height. This may be due to the under prediction of the WWIII model during the 29 May event.

The nearshore conditions at the mouth of the River Carron was compared for each scenario reflecting the supplied WWIII conditions and waves increased by 15%. The model results were extracted for a number of depths and the wave conditions compared. The results in Table B1 show that at the channel entrance (mean depth of 0.67m) the average wave conditions for both scenarios with within 1cm. Larger variations to the wave height are observed with increasing depth, which supports the assumption that the waves directly adjacent to the channel mouth are significantly depth limited.

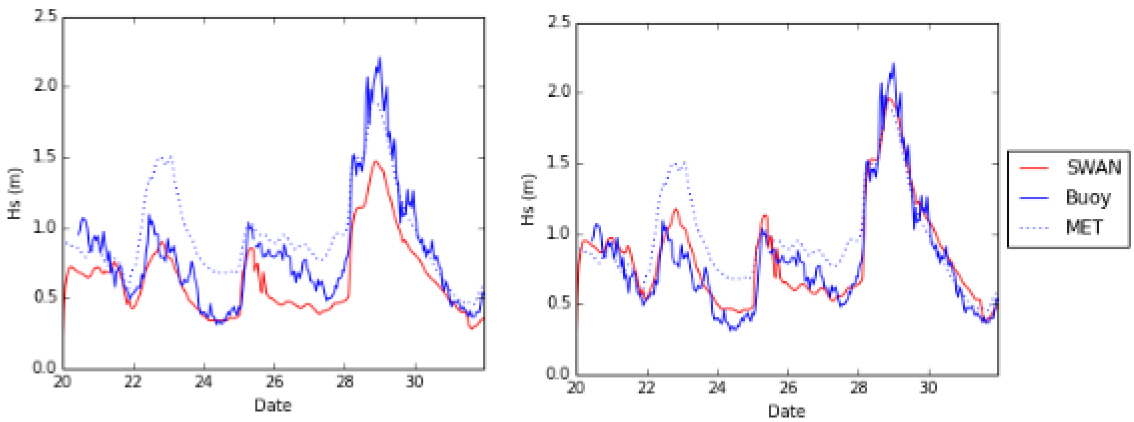

Figure B-7: Significant wave heights as recorded by the buoy and predicted by the SWAN model. Significant wave heights at the boundary were decreased by 15% (left) and increased by 15% (right) respectively. Mean RMS Errors = 0.19 m and 0.10 m respectively

Table B1: Comparison of nearshore wave conditions at the mouth of the River Carron during varying offshore wave conditions

Run scenario	Depth at model output point (mAOD)	Mean wave height (m)	Variation from basecase (m)
Basecase (supplied WWIII wave conditions)	0.67	0.44	
Waves decreased by 15%	0.67	0.45	0.01
Waves increased by 15%	0.67	0.43	-0.01
Basecase (supplied WWIII wave conditions)	0.89	0.52	
Waves decreased by 15%	0.89	0.53	0.02
Waves increased by 15%	0.89	0.49	-0.03
Basecase (supplied WWIII wave conditions)	1.20	0.56	
Waves decreased by 15%	1.20	0.59	0.03
Waves increased by 15%	1.20	0.51	-0.04
Basecase (supplied WWIII wave conditions)	1.70	0.59	
Waves decreased by 15%	1.70	0.63	0.05
Waves increased by 15%	1.70	0.53	-0.05

B.4 Validation simulation

A validation was performed by applying the model settings selected following the model calibration by applying the model to a further 20 day period of recorded data (from the 1st June 2014 to the 20th June 2014). Errors in significant wave height (see Figure B-8) were similar to those observed from the calibration with a root mean square error of 0.11m (compared to 0.12m from the calibration). Peak wave periods (see Figure B-9) were predicted more accurately with an RMS error of 1.3s (compared to 2.6s) and Tm02 periods had similar errors to those observed previously with an RMS error of 0.7s (compared to 0.6s).

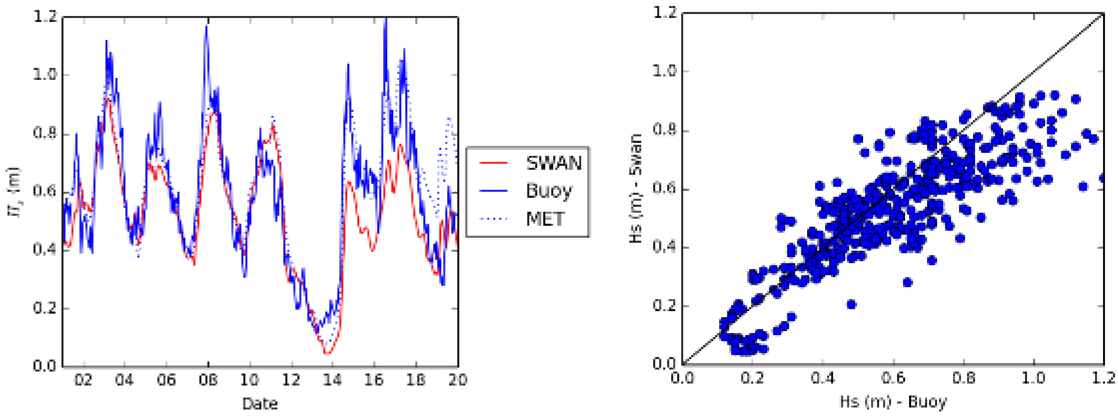

Figure B-8: Significant wave heights as recorded by the buoy and predicted by the SWAN model. Mean RMS Error = 0.11m

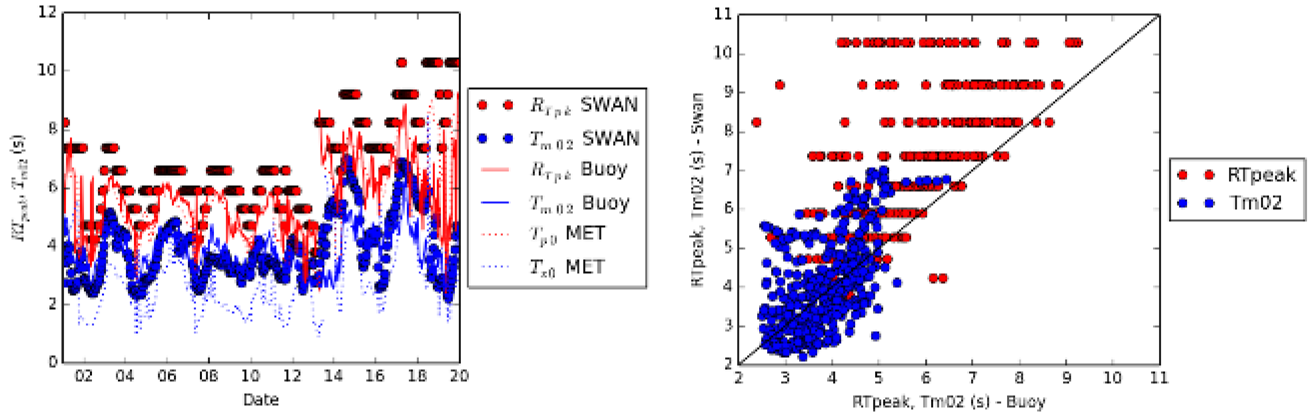

Figure B-9: Peak and mean wave periods as recorded by the buoy and predicted by the SWAN model. Mean RMS Error = 1.3 s and 0.7 s respectively.

B.5 Summary of calibration

A SWAN wave transformation model was set up to determine nearshore wave conditions close to the mouth of the River Carron. The model was calibrated and validated against recorded wave buoy data collected close to the river mouth. The best performance was achieved by implementing a Collins bed friction scheme combined with a Westhuysen white capping scheme. RMS errors in significant wave heights were approximately 0.1m. There was some degree of under-prediction of larger wave conditions, which was attributed to errors in the forecast data used to characterise the boundary conditions of the wave transformation model. However, to ensure the model does not under-predict the nearshore wave conditions, and in response to recent observations of offshore conditions greatly exceeding published extreme datasets, it is recommended that the conditions at the offshore wave boundary are increased by 15% for all model simulations.

C Appendix C - Wave speed during high channel flows

C.1 River flow model and consideration of wave propagation

While waves have been witnessed within the channel under extreme sea levels and low flows, it is not known how they will be influenced by strong currents being created during high flows in the River Carron. This has been assessed in terms of the flow depth and current velocity during extreme fluvial events, which was compared against shallow water wave processes.

The physical processes involving wave propagation is complex, differing between short waves (with periods from 0.1 sec) and long waves (with periods up to hours). This study has focussed on waves between these two extremes, made up of sea and swell conditions with a period between 1 to 20 seconds. The most basic form of a wave follows a sinusoidal form, although this becomes distorted in very shallow water due to the interaction of the fluid motion with the bathymetry. However, at this conceptual level small amplitude wave theory has been used to broadly consider the wave properties at the River Carron. The velocity of waves traveling through shallow water is dependent on the wavelength (the distance over which the wave crests repeat themselves) and the depth of the water. This is also common for long waves, which includes tides, seiches, surges and tsunamis. Waves propagate with a velocity (Celerity), C, where C = wavelength / period.

Using these basic wave properties the wave speed has been compared against channel velocity during observed wave events and an extreme 1 in 200-year event. If the channel velocity exceeds the wave speed, wave propagation is not considered to have a dominant effect.

C.2 Validation of approach - assessment of conditions during 15 December

Waves are known to have entered the River Carron on 15 December 2012. The flow velocity during this event has been calculated using an Infoworks RS 1D hydraulic model developed by JBA Consulting to assess existing flood risk to Stonehaven[32]. The model layout represents the proposed Stonehaven Protection Scheme (i.e. it includes proposed wall and parapet levels) and was run in an unsteady state.

Data from the River Carron gauge shows that flows in the Carron peaked at approximately 6.5 m^3/s at around 0300hrs on 15 December, however by the time the high tide of 2.59 mAOD at 1400hrs, flows in the Carron had reduced to approximately 1.5 m^3/s. Photographs of waves propagating up the channel were taken at approximately 1445hrs[33]. The Infoworks model was run using the recorded flow gauge data, contributions from the Glaslaw Burn which converges with the River Carron downstream of the gauge and a downstream tidal hydrograph, which was scaled to match the recorded peak of 2.59 mAOD. Table B-10 shows the simulated water levels and velocities within the downstream section of the channel at 1445hrs. Using linear wave theory, the wave speed has been calculated based on the nearshore conditions at this time (refer to Section 4.3.3) and the water depth in the channel, and is considered to be approximately 3.3 m/s. As the fluvial velocity is lower than the estimated wave velocity, waves are perceived to have been able to propagate within the channel, therefore matching observed conditions and offers a degree of validation to the methodology.

Table B-10: Simulated water level and flow velocity within the River Carron at 1445hrs 15 December 2012.

Cross section reference	Water Level (mAOD)	Velocity (m/s)	Could wave propagate?
Mouth (ref:CAR_000)	1.99	0.1	Yes
Downstream footbridge (XS ref: CAR_132)	2.03	0.43	Yes
Bridgefield Bridge (XS ref: CAR_214)	2.33	0.82	Yes

[32] JBA Consulting (2012) Stonehaven River Carron Flood Alleviation Study, prepared for the Aberdeenshire Council.

[33] Based on the date/time stamp on the photographs, considered correct by the photographer

C.3 Flows during an extreme event

Using the approach described above, an analysis has been undertaken to consider the relative velocities of flow and wave velocities under extreme conditions. In order to achieve this, a joint probability analysis was conducted comparing tidal levels to flows during a 200-year event. Table B-11 shows the range of flow and tide combinations for the joint probability assessment, which includes allowances for climate change (CC).

Table B-11: Joint probability flow and tide conditions within the River Carron[34]

200-year scenarios		200 year + CC scenarios	
Flow (m^3/s)	Tide (mAOD)	Flow (m^3/s)	Tide (mAOD)
4.7	3.25	6.3	3.92
11.7	3.20	14.8	3.87
17.0	3.16	22.6	3.83
23.9	3.07	31.8	3.74
29.1	3.01	38.7	3.68
35.9	2.91	47.7	3.58
43.1	2.84	57.3	3.51
47.1	2.80	62.6	3.47
50.3	2.77	66.9	3.44
58.6	2.70	77.9	3.37

Each of the joint probability scenarios has been simulated using the Infoworks RS model, with the results shown in Table B-12 and Table B-13 for the present day and climate change scenarios respectively. The shallow water wave speed has been calculated, which suggests waves are able to propagate within the channel during large fluvial events, although may be restricted during high flow/low tide combinations. In addition, the modelling shows that during large fluvial events there is limited clearance under the Bridgefield Bridge which has a soffit level of 3.78mAOD.

Table B-12: Simulated water level and flow velocity within the River Carron for 200-year events

Cross section reference	Water Level Range (mAOD)	Velocity Range (m/s)	Wave velocity (m/s)	Could wave propagate?
Mouth (ref:CAR_000)	2.70 to 3.25	0.51 to 3.76	4.17 to 4.76	Velocity: Primarily yes, potentially limited for high flow/low tide combinations.
Downstream footbridge (XS ref: CAR_132)	3.27 to 4.44	1.08 to 1.49	4.17 to 4.76	Velocity: Yes Clearance: high flow/low water level combinations affected by bridge soffit.
Bridgefield Bridge (XS ref: CAR_214)	3.31 to 4.90	1.31 to 2.60	4.17 to 4.76	Velocity: Yes Clearance: high flow/low water level combinations affected by bridge soffit.

[34] Note: Flow values used are considered to be conservative and assume the River Carron and Glaslaw Burn peak at the same time. A climate change uplift of 33% has been applied for fluvial flows and an increase of 0.67m used for tidal levels.

Table B-13: Simulated water level and flow velocity within the River Carron for 200-year + CC events

Cross section reference	Water Level Range (mAOD)	Velocity Range (m/s)	Wave velocity (m/s)	Could wave propagate?
Mouth (ref:CAR_000)	3.37 to 3.92	0.80 to 4.66	4.88 to 5.39	Velocity: Primarily yes, potentially limited for high flow /low tide combinations.
Downstream footbridge (XS ref: CAR_132)	3.91 to 5.02	1.24 to 1.49	4.88 to 5.39	Velocity: Yes Clearance: high flow /low water level combinations affected by bridge soffit.
Bridgefield Bridge (XS ref: CAR_214)	3.96 to 5.65	1.60 to 2.69	4.88 to 5.39	Velocity: Yes Clearance: high flow /low water level combinations affected by bridge soffit.

D Appendix D - Breakwater design information

(Supplied separately)

D.1 Appendix D1 - Design Technical Note

D.2 Appendix D2 - Designers Hazard Inventory

D.3 Appendix D3 - Detached Breakwater Plan View

D.4 Appendix D4 - Detached Breakwater Cross Section

Offices at

Coleshill

Doncaster

Edinburgh

Exeter

Haywards Heath

Limerick

Newcastle upon Tyne

Newport

Saltaire

Skipton

Tadcaster

Thirsk

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Registered Office
South Barn
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SKIPTON
North Yorkshire
BD23 3AE

t:+44(0)1756 799919
e:info@jbaconsulting.com

Jeremy Benn Associates Ltd
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