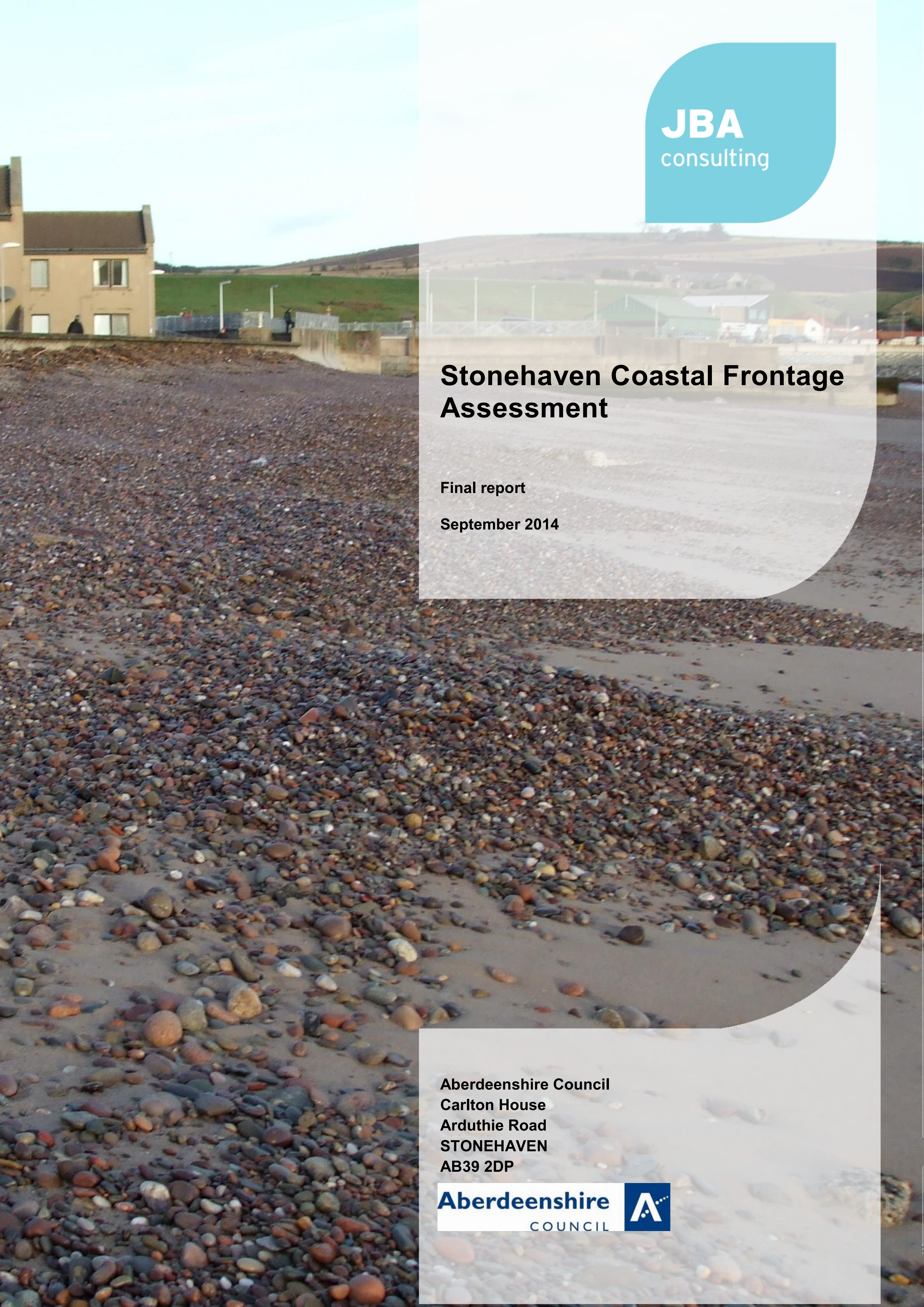

JBA consulting

Stonehaven Coastal Frontage Assessment

Final report

September 2014

Aberdeenshire Council
Carlton House
Arduthie Road
STONEHAVEN
AB39 2DP

Aberdeenshire COUNCIL

JBA Project Manager

Graham Kenn
JBA Consulting
Aberdeen House
South Road
Haywards Heath
West Sussex
RH16 4NG

Revision History

Revision Ref / Date Issued	Amendments	Issued to
V1.0 17 April 2014		Joanna Cubbage
V1.1 28 May 2014	Revised beach recharge cost estimates	Joanna Cubbage
V2.0 28 July 2014	Revised wave overtopping calculations and cost-benefit	Joanna Cubbage
V2.1 5 September 2014	Following JC review	Joanna Cubbage

Contract

This report describes work commissioned by Aberdeenshire Council, by a letter dated 20 February 2014. Aberdeenshire Council's representatives for the contract were Joanna Cubbage and Liam Rockford. Rami Malki, Josh Harris and Daniel Rodger of JBA Consulting carried out this work.

Prepared by ....Rami Malki MEng PhD

Coastal Engineer

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

Senior Coastal Engineer

Reviewed by ....Graham Kenn BSc MSc CEng MICE CEnv MCIWEM C.WEM

Principal Engineer

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

Head of Marine and Coastal Risk Management

Purpose

This document has been prepared as a final report for Aberdeenshire Council. JBA Consulting accepts no responsibility or liability for any use that is made of this document other than by the Client for the purposes for which it was originally commissioned and prepared.

JBA Consulting has no liability regarding the use of this report except to Aberdeenshire Council.

Acknowledgements

JBA would like to thank the staff at Aberdeenshire Council for their assistance whilst carrying out this work.

Copyright

Carbon Footprint

A printed copy of the main text in this document will result in a carbon footprint of 198g if 100% post-consumer recycled paper is used and 252g if primary-source paper is used. These figures assume the report is printed in black and white on A4 paper and in duplex.

JBA is aiming to reduce its per capita carbon emissions.

Executive Summary

This study was undertaken by JBA Consulting for Aberdeenshire Council as a high level overview to help inform the future management of coastal flooding in Stonehaven. It has investigated the magnitude of recent storm events in terms of offshore waves, sea levels and expected overtopping (such as in December 2012). Using this information the standard of protection of the coastal frontage was considered, and potential improvements to the beach and rear seawall assessed that could reduce the level of coastal flood risk.

An assessment of historic storms between 2002 to 2012 indicates the December 2012 coastal event has a return period of under 1-year for the observed water levels, over 200-years for offshore waves, and over 200-years for the joint-probability wave and water level conditions experienced. Using a simplified depth-limited approach the nearshore wave height and overtopping was estimated for historic events. The December 2012 event was considered to have the highest rate of overtopping since 2002, however a more detailed assessment is required in order to characterise the rate of overtopping between individual defence profiles.

Potential changes to the existing beach width and elevation was investigated to decrease the existing rate of overtopping. In order to meet a nominal present day 5l/s/m overtopping target, the following beach characteristics are required.

- Profile SH05: A widened beach between 10m to 15m.
- Profile SH12: Currently meets the nominal overtopping target under the simplified depth-limited assessment method (which is not considered reflective of existing conditions).
- Profile SH17: A 0.5m increase to the existing defence crest level and a 15m wide beach.

An assessment of the increased overtopping rate under the influence of climate change indicates all defences will require an increased crest level in the future, ranging between 0.5 to 2m to meet the 5l/s/m target.

A review of the beach recycling activities and sediment changes between 2008 and 2013 show a general accumulation of sediment throughout the bay, although an ongoing loss of sediment is observed to the South of the River Carron. These trends support previous recommendations for the addition of two 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.

A high-level cost-benefit analysis was undertaken for two beach recharge scenarios reflecting a mid- and high-level beach recharge scheme. The results suggest that the economic viability of the scheme is only viable if a low-cost local sediment supply could be identified. While the cost-benefit results are considered to be above parity, they are considered finely balanced and very sensitive to the assumptions and limitations used throughout this study. Any future review of this assessment is therefore recommended to use detailed numerical modelling, to consider a more holistic defence strategy incorporating different defence requirements in different parts of the bay (e.g. wall raising vs. recharge), and consider the desired standards of defence (e.g. allowing 10l/s/m overtopping during a 200-year event). By considering these elements it is considered that a more cost effective strategy can be developed for the bay than beach recharge alone, which could include the use of property level protection (PLP) to assist in reducing and managing the consequences of coastal flooding.

Contents

Executive Summary ..... **iii**

1 Introduction **1**

1.1 Project background 1
1.2 Report structure 1

2 Review of coastal processes **3**

2.1 Introduction 3
2.2 Drivers of coastal flood risk 3
2.3 Wave overtopping and tolerable thresholds 3

3 Historic assessment of storms **6**

3.1 Introduction 6
3.2 Coastal extremes 6
3.3 Joint probability analysis 7
3.4 Historical event analysis 8
3.5 Wave overtopping analysis for historical events 11
3.6 Summary of historic assessment 15

4 Wave overtopping analysis **16**

4.1 Extreme overtopping assessment 16
4.2 Modifications to existing beach 17
4.3 Summary of overtopping analysis 23

5 Coastal management advice **24**

5.1 Introduction 24
5.2 Historic beach management activities 24
5.3 Sediment changes 25
5.4 Maintaining minimum beach widths 27
5.5 High level assessment of beach recharge suitability 28
5.6 Flood damages avoided by proposed options 31
5.7 Consideration of beach recharge economics 33
5.8 Logistics of implementing a local coastal flood warning system 35
5.9 Resistance and resilience measures 37

6 Study limitations **39**

7 Summary and recommendations **40**

Appendices **42**

A Offshore extreme wave and water level data **43**

B Joint probability Analysis **46**

List of Figures

Figure 1-1: The study site at Stonehaven. 1
Figure 2-1: Components of wave overtopping 3
Figure 2-2: Schematisations of a typical beach profile for analysis using the Neural Network overtopping tool 4
Figure 3-1: Observed and predicted coastal flooding events and how they relate to joint probability return periods accounting for extreme still water level and offshore wave height 10
Figure 3-2: Locations of the beach profiles used to assess the vulnerability of Stonehaven to coastal flooding 12
Figure 3-3: Surveyed ground elevations and schematised profiles used to represent beach sections in the overtopping analysis 13
Figure 4-1: Predicted overtopping rates for various beach elevations under 1 in 200-year extreme sea-level and depth-limited nearshore wave conditions under a present-day scenario. 18
Figure 4-2: Predicted overtopping rates for various beach widths under 1 in 200-year extreme sea-level and depth-limited nearshore wave conditions under a present-day scenario. 19
Figure 5-1: Beach sections 1 to 4 assessed through 2008 and 2014 volumetric surveys. 26
Figure 5-2: Predicted erosion for various beach elevations under 1 in 200-year extreme sea-level and depth-limited nearshore wave conditions under a present-day scenario. 28
Figure 5-3: Existing beach profile SH05 and required recharge under Scenario 1 (existing beach) and 2 (raised beach). 29
Figure 5-4: Existing beach profile SH17 and required recharge under Scenario 1 (existing beach) and 2 (raised beach). 30
Figure 5-5: FWS operational procedure 36
Figure 7-1: Location of the extraction points used extreme still water levels and wave heights offshore of Stonehaven 43
Figure 7-2: Dependence information for wave height and SWL (Source: Defra 2003) 46
Figure 7-3: Lower limit bands for joint probability return periods accounting for extreme still water level and wave height. 48

List of Tables

Table 2-1: Limits for overtopping for pedestrians (source: EurOtop) 4
Table 2-2: Limits for overtopping for vehicles (source: EurOtop) 4
Table 2-3: Limits for overtopping for property and damage to the defence (source: EurOtop) 4
Table 3-1: Tide levels at Stonehaven 6
Table 3-2: Extreme wave estimates at Stonehaven for offshore waves from varying directions (Source: CFBD) 7
Table 3-3: Extreme wave estimates at Stonehaven for offshore waves originating from the northeast 7
Table 3-4: Extreme water levels at Stonehaven for different return periods 7
Table 3-5: Combinations of extreme still water levels and wave heights required to

achieve various joint probability return periods for a high correlation coefficient (ρ = 0.37) 8

Table 3-6: Water levels and wave heights during observed storm events 9

Table 3-7: Water level, offshore wave height and joint probability (ρ = 0.37) return periods for historic events, ranked by highest return period 11

Table 3-8: Predicted overtopping rates for the three beach sections for historic events, using the depth-limited estimated nearshore wave heights. 14

Table 4-1: Predicted overtopping rates for extreme sea-level and depth-limited nearshore wave conditions under present-day and climate change scenarios. 16

Table 4-2: Predicted overtopping rates for various beach elevations under a present day 1 in 200-year extreme sea-level and depth-limited nearshore wave scenario. 18

Table 4-3: Predicted overtopping rates for various beach widths under a present day 1 in 200-year extreme sea-level and depth-limited nearshore wave scenario. 19

Table 4-4: Predicted overtopping rates for Profile SH05 for various beach widths and defence crest levels under 1 in 200-year extreme sea-level and depth-limited nearshore wave conditions under present-day and climate change scenarios. 20

Table 4-5: Predicted overtopping rates for Profile SH12 for various beach widths and defence crest levels under 1 in 200-year extreme sea-level and depth-limited nearshore wave conditions under present-day and climate change scenarios. 21

Table 4-6: Predicted overtopping rates for Profile SH17 for various beach widths and defence crest levels under 1 in 200-year extreme sea-level and depth-limited nearshore wave conditions under present-day and climate change scenarios. 22

Table 5-1: Beach recycling operations carried out at Stonehaven. (source: HR Wallingford 2009 and Aberdeenshire Council) 25

Table 5-2: Cost estimates for beach recharge and defence upgrades 30

Table 5-3: Do Nothing damages for a range of flooding onset and write off assumptions (£k) 33

Table 5-4: Cost benefit results based on the WAAD methodology 34

Table 5-5: Cost benefit results based on the write-off methodology 34

Table 7-1: Probability of occurrence of offshore wave period for different wave heights 43

Table 7-2: Extreme wave estimates for waves originating from the northeast. 44

Table 7-3: Tide levels at Stonehaven 44

Table 7-4: Extreme water levels at Stonehaven for different return levels 45

Table 7-5: Combinations of extreme still water levels and wave heights required to achieve various joint probability return periods for a low correlation coefficient (ρ = 0.12) 47

Table 7-6: Combinations of extreme still water levels and wave heights required to achieve various joint probability return periods for a mid-range correlation coefficient (ρ = 0.21) 47

Table 7-7: Combinations of extreme still water levels and wave heights required to achieve various joint probability return periods for a high correlation coefficient (ρ = 0.37) 47

Abbreviations and Definitions

BODC............................. British Oceanographic Data Centre

CFAT.............................. Coastal Flood Alert Tool

CFBD .... Coastal Flood Boundary Dataset

EA .... Environment Agency

FEWS............................. Flood Early Warning System

FWS............................... Flood Warning System

MCM .... Multi Coloured Manual

NN.................................. Neural Network

NOC............................... National Oceanographic Centre

OB.................................. Optimism Bias

SAA................................ Scottish Assessors Associated

SEPA .... Scottish Environment Protection Agency

SoP .... Standard of Protection

SWL .... Still Water Level

UKCMF .... United Kingdom Coastal Monitoring and Forecasting service

UKCP09......................... UK Climate Projections

WAAD............................ Weighted Annual Average Damage

WW3.............................. Wave Watch 3

1 Introduction

1.1 Project background

This study was undertaken by JBA Consulting, on behalf of Aberdeenshire Council, to investigate the magnitude of recent storm events in terms of offshore waves, sea levels and wave overtopping (such as on December 2012). Using historic and extreme conditions the existing Standard of Protection (SoP) of the coastal frontage was estimated, and potential improvements identified that could be made to the beach to reduce the level of coastal flood risk.

Stonehaven lies 15 miles south of Aberdeen along Stonehaven Bay, which is situated between the River Carron and the River Cowie (see Figure 1-1). Coastal flooding in Stonehaven is typically the result of extreme sea levels and extreme wave conditions, although some protection to waves is offered due to the southern headland. Each of these processes can occur in isolation or combination, with the latter resulting in more severe floods. The coastal frontage is protected in part due to a rear seawall, stepped revetment and a managed beach.

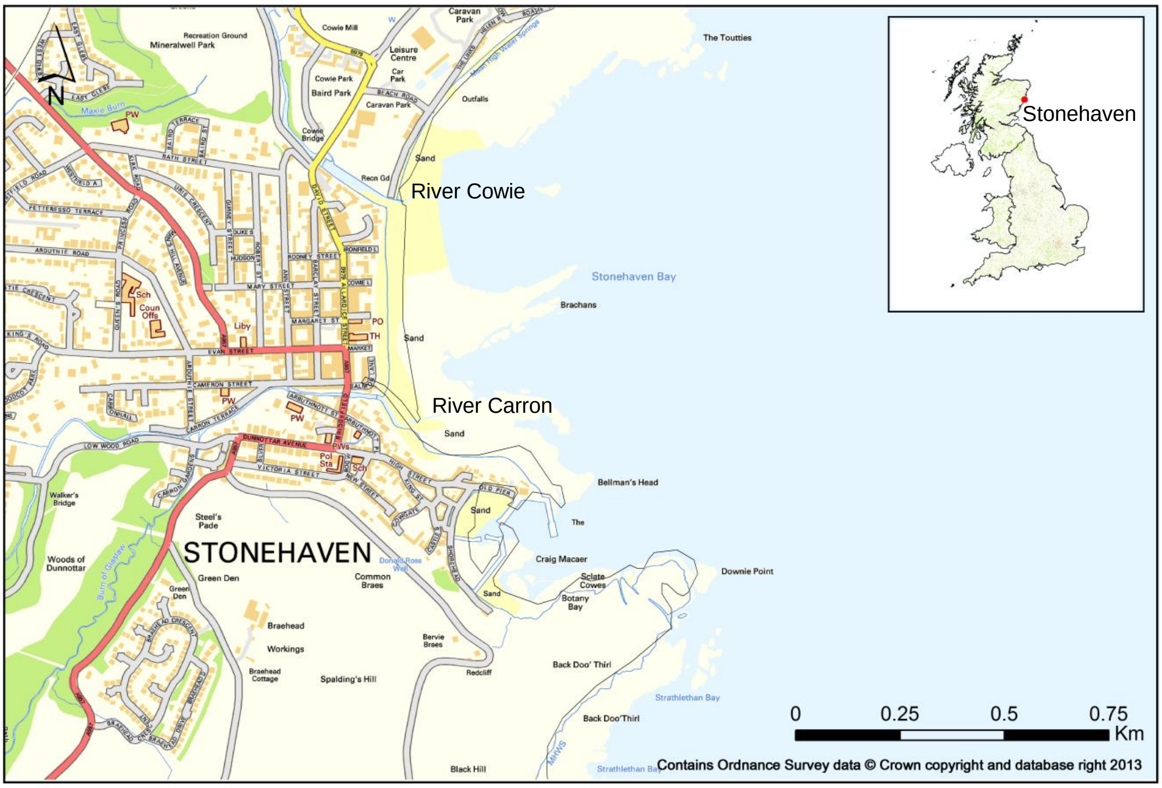

Figure 1-1: The study site at Stonehaven.

1.2 Report structure

This report consists of the following sections:

- **Chapter 2 (Review of coastal processes)** reviews the processes that create a coastal flood risk at Stonehaven.
- **Chapter 3 (Historic assessment of storms)** reviews the wave and water level conditions associated with recent and historic storm events.
- **Chapter 4 (Wave overtopping analysis)** evaluates the expected wave overtopping due to extreme events and outlines solutions for reducing this wave overtopping.
- **Chapter 5 (Coastal management advice)** considers pragmatic actions that could be made to reduce the present day coastal flood risk, including: design beach widths, allowances for storm erosion, required beach recharge volumes and the potential for a coastal flood warning system.

- **Chapter 6 (Study limitations)** summarises the limitations to the study and presents recommendations for further work.

2 Review of coastal processes

2.1 Introduction

The first stage in the development of any coastal protection scheme involves consideration of the local mechanisms of coastal flood 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 principal mechanisms of coastal flood risk are discussed to provide a conceptual understanding for the development of new mitigation strategies. Following this review, the available guidance on tolerable wave overtopping discharge is summarised, as these provide a basis for future designs.

2.2 Drivers of coastal flood risk

Stonehaven lies on the east coast of Scotland with direct exposure to storm surges and extreme wave conditions from the North Sea. The most likely cause of coastal flooding at Stonehaven is due to waves overtopping the existing defences. Wave overtopping can occur when waves propagate to a shoreline and break over the coastal defences (seawalls, revetments etc.), spreading water behind the coastal frontage. As this occurs, the waves have the potential to cause damage to any infrastructure located behind the foreshore, either through scour, inundation or high flows. Wave overtopping is a complex process controlled by the state of the sea (water depth, wave properties) and the geometry of the beach and foreshore, as shown in Figure 2-1.

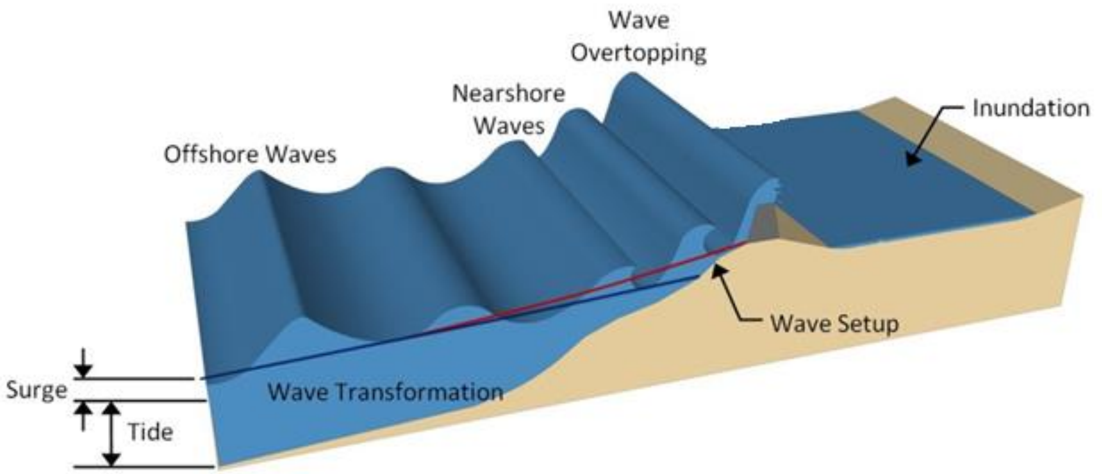

Figure 2-1: Components of wave overtopping

2.3 Wave overtopping and tolerable thresholds

The complexity of the physical processes leading to wave overtopping introduces a high degree of uncertainty into its quantification. As a result, the overtopping caused by individual waves is not typically calculated; instead the average overtopping rate for a particular sea-state is estimated using empirical or physical models. An example is the Neural Network tool. This empirical-based model is described in the industry standard EurOtop[1] manual as the most suitable methodology for evaluating wave overtopping for composite defences such as seawall structures and armour. Even so, as with all calculation approaches, the Neural Network tool has several limitations. Estimates are given based on a dataset of small-scale physical model tests which are affected by model and scale effects, the accuracy of measurement equipment and wave generation techniques. There is also the potential for limited data for particular schematisations, for example overtopping across wide (say 30m wide) beaches, as few model tests are available within the database. As a result, it is important that the results of the Neural Network are used with a degree of engineering judgement and caution.

The Neural Network tool can be applied to different beach profiles, the geometric properties of which are characterised using 15 parameters including: crest height (Rc); armour height (Ac);

[1] EurOtop (2010) "Wave Overtopping of Sea Defence and Related Structures: Assessment Manual", Overtopping Course Edition, November 2010. HR Wallingford.

armour width (Gc); berm elevation (hb); berm width (B); upper slope (α_u); lower slope (α_d); and roughness (γf) (see Figure 2-2).

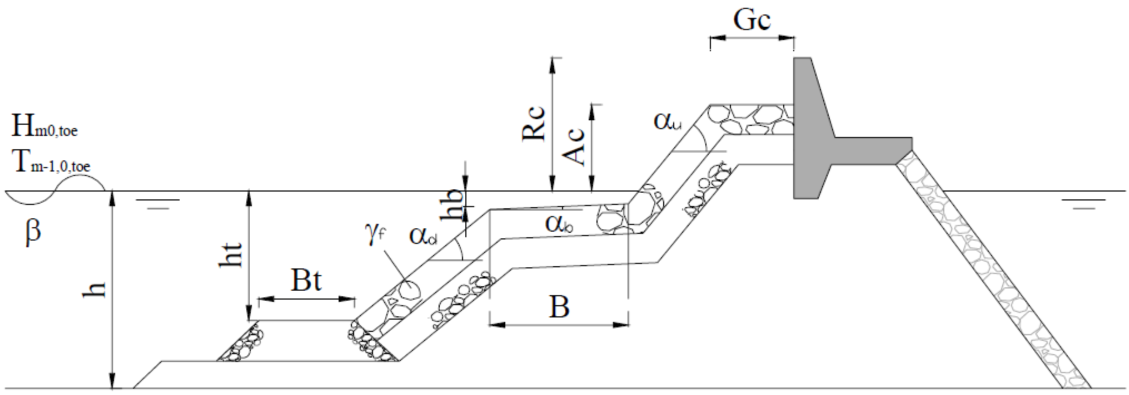

Figure 2-2: Schematisations of a typical beach profile for analysis using the Neural Network overtopping tool

Using the Neural Network model, the average rate of overtopping can be calculated for a beach or defence cross-section. These can then be related to guidance given in the EurOtop manual which relates hazardous situations to overtopping rates and volumes. The tolerable limits for pedestrians and vehicles are given in Table 2-1 and Table 2-2 respectively. As discussed within this report, these tolerable limits provide a basis for the design of mitigation strategies.

Table 2-1: Limits for overtopping for pedestrians (source: EurOtop)

Hazard type and reason	Mean discharge	Max volume
	q (l/s/m)	Vmax (L/m)
Trained staff, well shod and protected, expecting to get wet, overtopping flows at lower level only, no falling jet, low danger of fall from walkway.	1-10	500 at low level
Aware pedestrian, clear view of sea, not easily upset or frightened, able to tolerate getting wet, wider walkway.	0.1	20-50 at high level or velocity

Table 2-2: Limits for overtopping for vehicles (source: EurOtop)

Hazard type and reason	Mean discharge	Max volume
	q (l/s/m)	Vmax (L/m)
Driving at low speed, overtopping by pulsating flows at low flow depths, no falling jets, vehicle not immersed.	10 - 50[2]	100 – 1,000
Driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets.	0.01 – 0.05[3]	5 – 50 at high level or velocity

Table 2-3: Limits for overtopping for property and damage to the defence (source: EurOtop)

Hazard type and reason	Mean discharge
	q (l/s/m)
Damage to building structural elements	1[4]
Damage to equipment set back 5-10m	0.4[5]

[2] Note: These limits relate to overtopping defined at highways.
[3] Note: These limits relate to overtopping defined at the defence, assumes the highway is immediately behind
[4] Note: This limit relates to the effective overtopping defined at the building
[5] Note; This limit relate to overtopping defined at the defence

No damage to embankment/seawalls if crest and rear slope are well protected	50-200
No damage to embankment / seawall crest and rear face of grass covered embankment of clay	1-10
Damage to paved or armoured promenade behind a seawall	200
Damage to grassed or lightly protected promenade	50

3 Historic assessment of storms

3.1 Introduction

On the 15th December 2012, a large storm caused wave overtopping along the lower seawall immediately south of the River Cowie, which inundated a number of properties. This event is considered to be the largest in recent years, with the most significant rate of overtopping observed historically. This event was investigated to estimate the associated return period, which has been undertaken in the following sections:

- **Coastal extremes:** This section presents the offshore extreme wave conditions and sea levels for a range of return periods.
- **Joint probability analysis:** This section assesses the likelihood for extreme wave heights and water levels occurring simultaneously.
- **Historic event analysis:** This section considers the return period of historic events based on their offshore conditions and presents their estimated overtopping rate.

3.2 Coastal extremes

Stonehaven is a secondary non-harmonic port, with its tidal predictions being based on the primary port of Aberdeen, located approximately 21km to the north. The region experiences a meso-tidal climate, with an astronomic (mean spring) tidal range of 4.5m, as shown in Table 3-1. The highest astronomical tide level at Stonehaven is 2.65mAOD.

Extreme wave and water level conditions for Stonehaven are available as part of several government reports on coastal extremes. A full description of the data obtained for Stonehaven is provided in Appendix A, and is summarised below.

Extreme coastal conditions have been obtained from the Environment Agency (EA) / SEPA *Coastal flood boundary conditions for UK mainland and islands* project[6], 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). Due to the presence of headlands and rocky reefs these conditions do not necessarily result in the largest nearshore waves, which have not been calculated within this study, however could be estimated using a wave transformation model. 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). Predicted extreme still water levels (SWL) at Stonehaven for a range of return periods are presented in Table 3-4.

The latest UK Climate Projections (UKCP09)[7] include estimates of the future effects of climate change on mean sea levels. A medium emissions scenario with a 95th percentile confidence interval is considered to result in a **0.67m** rise in sea level by 2115. The resulting 2115 extreme sea levels are presented in Table 3-4.

Table 3-1: Tide levels at Stonehaven

Location	Level (mAOD)
Highest Astronomical Tide (HAT)	2.65
Mean High Water Springs (MHWS)	2.05
Mean High Water Neaps (MHWN)	1.15
Mean Sea Level (MSL)	0.17
Mean Low Water Neaps (MLWN)	-0.75
Mean Low Water Springs (MLWS)	-1.85
Lowest Astronomical Tide (LAT)	-2.45

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

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

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

Wave direction	1-year	2-year	5-year	10-year	50-year	100-year	200-year
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 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

Table 3-4: Extreme water levels at Stonehaven for different return periods

Return Period (year)	Water levels (mAOD) (2008)	2115 (mAOD) (2008 level +0.67m)	Return Period (year)	Water levels (mAOD) (2008)	2115 (mAOD) (2008 level +0.67m)
0.2	2.60	3.27	**10**	2.97	3.64
0.5	2.69	3.36	**20**	3.03	3.70
1	2.73	3.40	**50**	3.12	3.79
2	2.80	3.47	**100**	3.19	3.86
5	2.89	3.56	**200**	3.25	3.92

3.3 Joint probability analysis

Whilst many extreme conditions are created from the same underlying coastal processes, extreme waves do not always coincide with extreme sea-levels. In reality, the likelihood of these conditions coinciding is a function of the level of interdependence of the dominant processes, the degree of which varies around the UK.

A number of extreme wave height and water level combinations for different combined return periods were determined through a joint probability analysis. This was achieved using methods described in the Defra best practice guidance[8], with a full description of the analysis provided in Appendix A. For example, a 200-year storm event at Stonehaven could consist of a range of wave and water level combinations, such as a 6.8m wave and 2.49mAOD sea level, or a 2.5m wave and a 3.25mAOD sea level. Both conditions have the same probability of occurrence; however, each scenario will result in different impacts at the shoreline (e.g. due to different wave overtopping rates).

A key parameter for the joint probability assessment is the level of dependence (ρ) between waves and water levels. The Defra guidance notes suggest a modest correlation along the eastern Scottish coastline ranging between the relatively low correlation of ρ=0.12 and the

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

relatively higher value of ρ= 0.37. A specific correlation coefficient is 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 will occur at greater levels of isolation. Within the assessment of historic events undertaken for this study, three correlation coefficients were used within the estimated range to consider their influence on quantifying the magnitude of previous storms.

Based on the analysis, the higher coefficient is considered to most accurately reflect the magnitude of historic events, with the associated offshore joint probability wave and sea level combinations shown in Table 3-5.

Table 3-5: Combinations of extreme still water levels and wave heights required to achieve various joint probability return periods for a high correlation coefficient (ρ = 0.37)

Extreme SWL		Joint probability return period (years)									
Magnitude (mAOD)	Return period (years)	0.2	0.5	1	2	5	10	20	50	100	200
		Extreme wave heights (m)									
2.49	0.1	1.66	2.35	3.04	3.51	4.14	4.62	5.10	5.77	6.28	6.80
2.57	0.2	1.36	2.05	2.66	3.13	3.76	4.24	4.71	5.36	5.87	6.39
2.66	0.5	-	1.57	2.15	2.63	3.26	3.73	4.21	4.84	5.34	5.85
2.73	1	-	-	1.77	2.25	2.88	3.35	3.83	4.46	4.94	5.44
2.80	2	-	-	-	1.87	2.50	2.97	3.45	4.08	4.55	5.04
2.89	5	-	-	-	-	2.00	2.47	2.95	3.58	4.05	4.53
2.97	10	-	-	-	-	-	2.09	2.57	3.20	3.67	4.15
3.03	20	-	-	-	-	-	-	2.19	2.82	3.29	3.77
3.12	50	-	-	-	-	-	-	-	2.31	2.79	3.27
3.19	100	-	-	-	-	-	-	-	-	2.41	2.89
3.25	200	-	-	-	-	-	-	-	-	-	2.51

3.4 Historical event analysis

3.4.1 Event data

In order to carry out a historic event analysis, a number of significant coastal events were identified. These events were selected based on events with known overtopping and previous investigations undertaken by JBA during the Coastal Flood Alert Tool (CFAT) study. The CFAT was developed for SEPA as a simple flood alert system, which compares sea level and wave forecasts against predefined thresholds to issue flood alerts. During a review of its performance, JBA identified several large events which were subsequently discussed and investigated by SEPA and Aberdeenshire Council. Events with evidence of overtopping or significantly high wave and water level combinations are listed in Table 3-6, which have then been assessed in terms of joint probability return-period. Also indicated on the table is further confirmation by Aberdeenshire Council on the events that are known to have caused overtopping. On review of the conditions during these events, the range of wave directions can be seen to originate from northeast (32 deg/N) to the south (180 deg/N)). Due to the presence of headlands and rocky reefs these conditions do not necessarily reflect the nearshore wave direction, which will be more aligned to be more shore-normal. This information could be calculated using a wave transformation model.

Table 3-6: Water levels and wave heights during observed storm events

Date	Water level (mAOD)	Wave height (m) / Period (s) / Dir (Deg/N)	Confirmation by Aberdeenshire Council
03/11/2002	2.43*	4.20 / 8.90 / 74*	Y
21/11/2002	2.02*	6.23 / 9.17 / 100*	-
06/11/2006	2.5	5.36 / 8.62 / 33*	-
21/02/2007	2.37*	3.02 / 8.37 / 64*	Y
05/03/2007	2.51*	2.97 / 6.67 / 176*	-
10/03/2008	2.67*	5.10 / 8.40 / 139*	Y
12/01/2009	2.81*	2.35 / 6.48 / 183*	-
30/03/2010	2.49	4.24 / 7.34 / 52	-
08/11/2010	2.5	4.35 / 7.39 / 135	Y
25/10/2011	2.23*	4.49 / 8.46 / 110*	-
15/12/2012	2.59**	7.50 / 9.9*** / 88	Y

* Source: Initial CFAT analysis (JBA 2013)[9]
** Source: British Oceanographic Data Centre (BODC) & SEPA Coastal Event Summary: East Coast December 2012
*** Recent conditions based on CFAT achieve forecast 2012121418 at Portlethen. No wave period information is available, which has been estimated based on wave height/period trends in the above data.

3.4.2 Return period assessment

The historic events presented in Table 3-6 were assessed to quantify their expected return period. Based on the extreme water level and wave height of each event, the joint probability of their wave and water levels occurring simultaneously is presented in Figure 3-1. The figure shows the three correlation coefficients discussed in Section 3.3. Since the largest extreme return period considered is 200-years, any events exceeding this band are categorised as 200-year or worse.

As shown by the results, the use of different correlation coefficients can significantly alter the perceived severity of an event. In particular, this can alter the number of extreme events that have been considered to occur, which is based on data spanning 2002 to 2012 (10 years). For instance,

- Using the coefficient **ρ = 0.12**, three 200-year events are considered to have occurred,
- Using the coefficient **ρ = 0.21**, two 200-year events are considered to have occurred,
- Using the coefficient **ρ = 0.37**, one 200-year event is considered to have occurred.

Based on the number of significantly large events, which appear to occur beyond the likely exceedance probability, the higher correlation coefficient is considered to give the most probable quantification of the magnitude of historic events. This trend of over prediction has been observed in recent studies by JBA where a joint sea level and wave height analysis was undertaken for the December 2013 and January 2014 coastal storms. One analysis, undertaken for the Environment Agency (EA) and Natural Resources Wales (NRW)[10], applied the Defra methodology method to determine the rarity of the observed events, which gave rise to very high estimates of joint return period. These estimates were considered implausible and appear to result from the simplifying assumptions of the desk-based method of joint probability analysis. The analysis suggests that the published correlation coefficients under predict the dependence between extreme wave and water levels, and the assumption of 707 separate 'events' per year has resulted in a greater uncertainty (weighted towards an over prediction) within the methodology.

[9] JBA (2012) Coastal Flood Alert Tool: Performance Review and Improvement. Prepared for the Scottish Environment Protection Agency.

[10] 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

Each of the historic events from Table 3-3 (that is wave condition between 32 to 180 deg/N) has been ranked in order of magnitude based on either the estimated return period of the offshore wave conditions, sea level or joint probability (using a ρ value of 0.37) and is presented in Table 3-7.

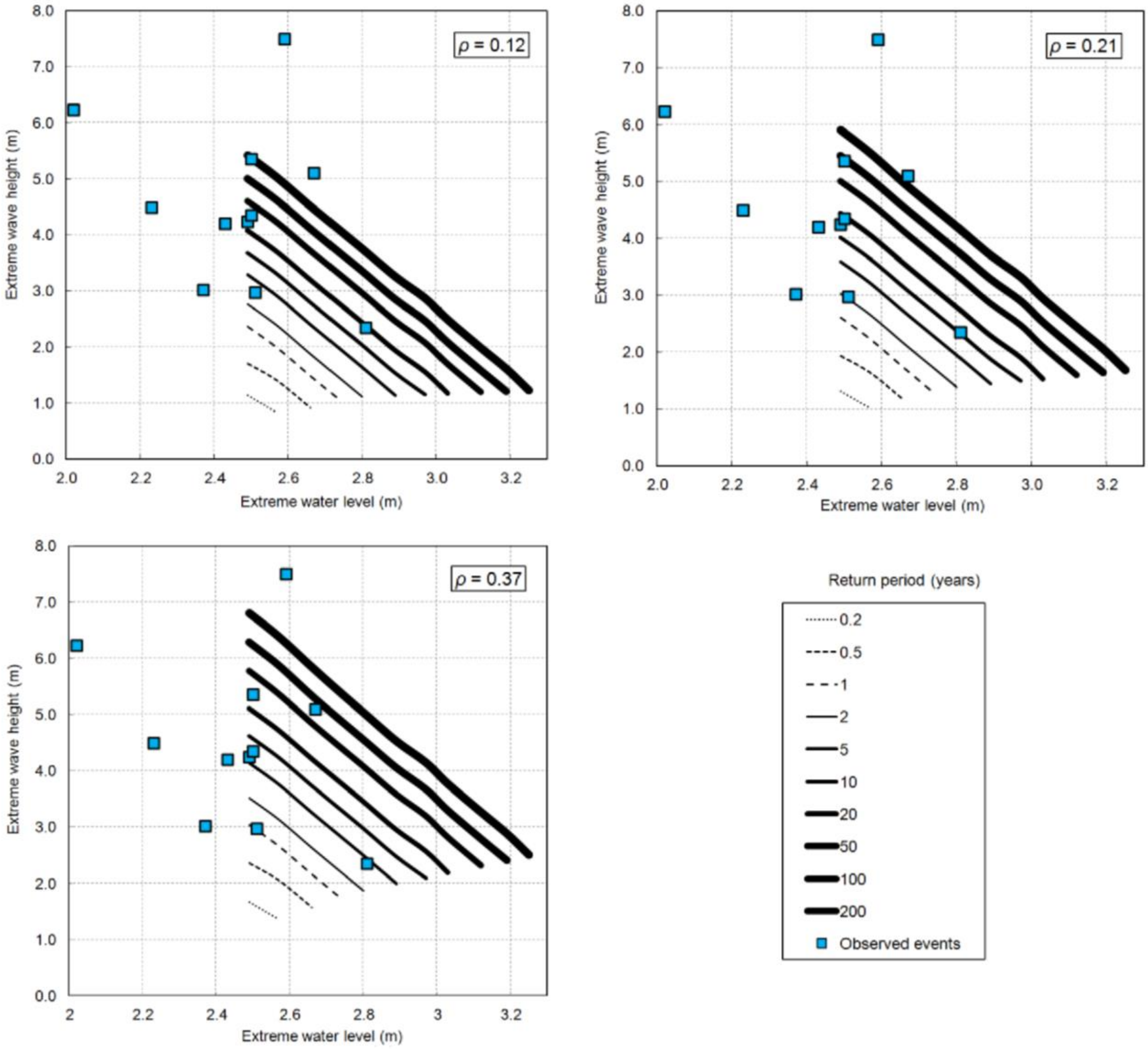

Figure 3-1: Observed and predicted coastal flooding events and how they relate to joint probability return periods accounting for extreme still water level and offshore wave height

Table 3-7: Water level, offshore wave height and joint probability (ρ = 0.37) return periods for historic events, ranked by highest return period

Event date	Offshore wave direction (Deg/N)	Water level return period (years)	Offshore wave height return period (years)	Joint probability return period (years) ρ = 0.37	Largest return period (years)
15/12/2012 **	88	< 1	> 200	> 200	> 200
10/03/2008 **	139	<1	5 - 10	50 - 100	50 - 100
06/11/2006	33	<1	5 - 10	20 - 50	20 - 50
21/11/2002	100	<1	20 - 50	<1	20 - 50
30/03/2010	52	<1	1 - 2	5 - 10	5 - 10
12/01/2009	183	2 - 5	<1	2 - 5	2 - 5
05/03/2007	176	<1	<1	1 - 2	1 - 2
03/11/2002 **	74	<1	1 - 2	<1	1 - 2
21/02/2007 **	64	<1	<1	<1	<1
25/10/2011	110	<1	1 - 2	<1	1 - 2
08/11/2012	135	< 1	<1	<1	<1
** Confirmation of overtopping by Aberdeenshire Council					

3.5 Wave overtopping analysis for historical events

The expected wave overtopping resulting from the historic events has been estimated to assess the current level of coastal flood protection at Stonehaven. Three representative beach profiles were used based on a beach survey completed during May 2013, as shown in Figure 3-2. The three profiles were selected to represent the variation in structure type, profile length and angle of exposure to wave action. The profiles were schematised (see Figure 3-3) and assessed using the Neural Network[11] overtopping tool which is described in Section 2.3.

[11] EurOtop (2010) "Wave Overtopping of Sea Defence and Related Structures: Assessment Manual", Overtopping Course Edition, November 2010. HR Wallingford.

Figure 3-2: Locations of the beach profiles used to assess the vulnerability of Stonehaven to coastal flooding

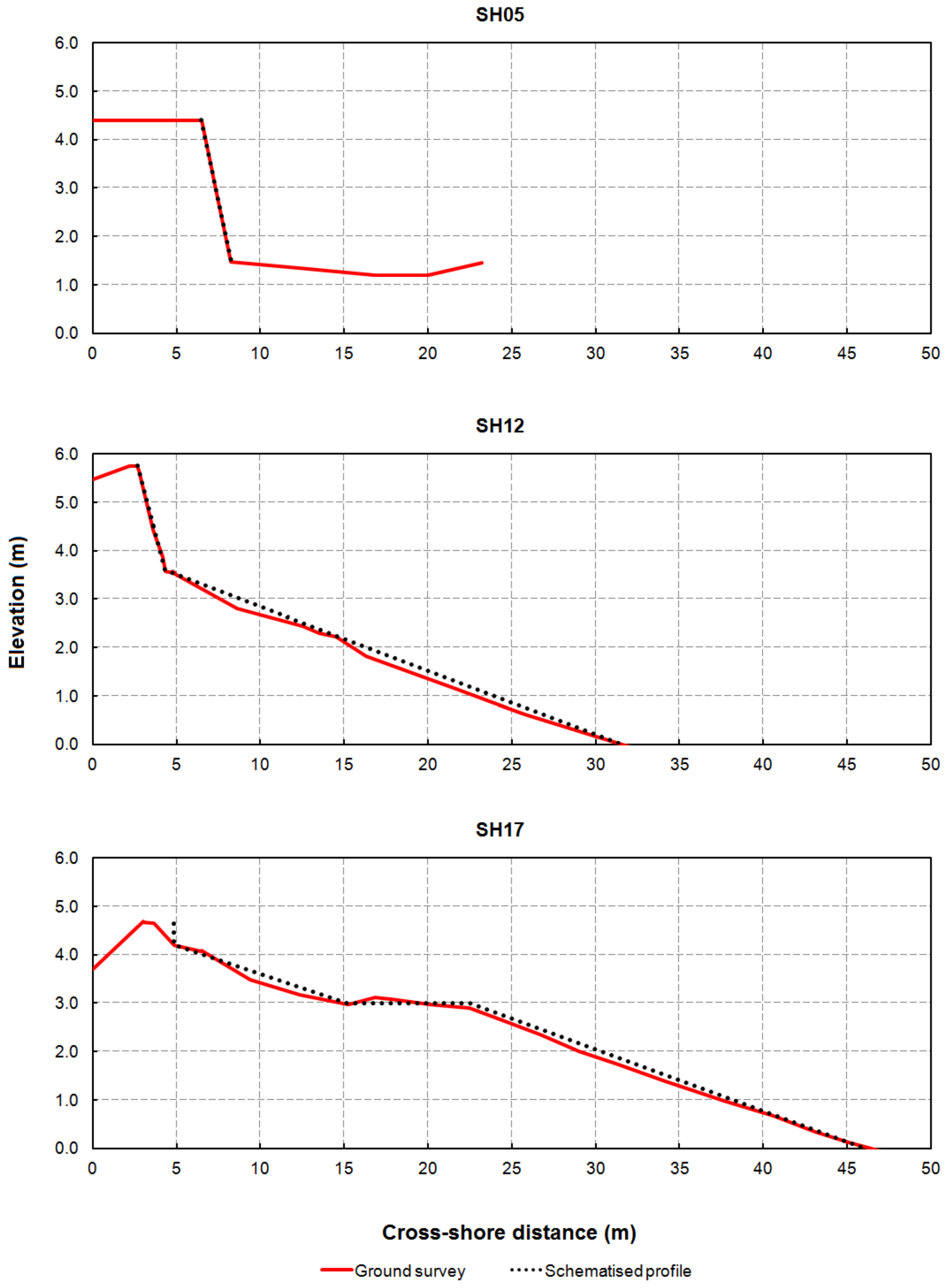

Figure 3-3: Surveyed ground elevations and schematised profiles used to represent beach sections in the overtopping analysis

3.5.1 Estimated nearshore wave heights

Wave overtopping is a complicated process controlled by the state of the sea (water depth, wave properties) and the geometry of the beach and foreshore. In particular, the nearshore wave conditions used within an analysis will play a significant role in determining the rate of wave overtopping. This project has not included a detailed assessment of wave transformation and breaking processes, and instead the nearshore wave heights were estimated empirically[12]. This approach is considered a high-level approach only, with the nearshore rock reefs expected to have a significant effect on nearshore wave conditions, and the nearby headland offering some protection during storm events. These processes are not accounted for in the empirical methods used.

The breaking wave height, H_b, is assumed to be directly proportional to the breaking depth, h_b. The relationship can be presented as:

$$H_b = \varkappa h_b$$

where $\varkappa$ is known as the breaker parameter. A breaker parameter of 0.52 has been used for the overtopping assessment, which is based on laboratory observations and is considered to offer the most realistic estimate when applied to the nearshore zone.

3.5.2 Estimated wave overtopping

The rate of wave overtopping was estimated for historic events using the Neural Network overtopping tool as described in Section 2.3. The estimated overtopping rate has been calculated for each profile based on the design still water levels and associated wave periods (refer to Table 3-6), and estimated nearshore wave height using a breaker parameter, $\varkappa$, of 0.52.

As shown in Table 3-8 the rate of overtopping during the December 2012 event is estimated to have been between 0.2 to 9.3 L/s/m. This is considered to be the highest rate of overtopping of the historic events, due to the relatively large water levels and estimated wave period. The lowest overtopping rate was calculated at Profile SH12 and the highest at Profile SH17. This is contrary to observed conditions, which is that Profile SH12 get overtopped the most regularly. This discrepancy is considered to be due to two primary factors:

1. The slope of the profile used for the overtopping assessment, which was based on the May 2013 survey (although steepened to reflect more scoured conditions). The Neural Network is very sensitive to small changes to the defence schematisation, and it may be possible to further 'calibrate' the overtopping profile by varying parameters such as the berm level or foreshore slope etc, or to based overtopping on profiles surveyed after coastal events.
2. The simplified method of estimating nearshore wave conditions, which did not consider the adjacent bathymetry and rock reefs. On review of aerial photographs of the site, it is possible that the adjacent reefs may act to funnel waves towards the profile SH12, which would increase the nearshore wave height and lead to increased wave overtopping.
3. It may be possible to use a larger breaker parameter to reflect the existing nearshore conditions (e.g. above 0.52), however there is no evidence to base this on at present.

Table 3-8: Predicted overtopping rates for the three beach sections for historic events, using the depth-limited estimated nearshore wave heights.

Historic event	Water level (mAOD)	Estimated overtopping rate (L/s/m)		
		Profile SH05	Profile SH12	Profile SH17
03/11/2002	2.43	0.0	0.3	3.8
21/11/2002	2.02	0.0	0.0	0.9
06/11/2006	2.50	0.0	0.3	4.6
21/02/2007	2.37	0.0	0.2	2.4
05/03/2007	2.51	0.0	0.1	2.1

[12] Battjes, J.A. Surf Similarity. Proc. 14th Intl. Conf. Coastal Eng., ASCE, Copenhagen, 466-480, 1974.

10/03/2008	2.67	0.0	0.6	7.9
12/01/2009	2.81	0.0	0.5	5.6
30/03/2010	2.49	0.0	0.2	2.7
08/11/2010	2.50	0.0	0.2	2.8
25/10/2011	2.23	0.0	0.1	1.3
15/12/2012	2.59	1.3	0.8	9.3

3.6 Summary of historic assessment

Based on the assessment, the December 2012 coastal storm is considered to have a return period of under 1-year for the observed water levels, over 200-years for offshore waves, and over 200-years for the joint-probability wave and water level conditions experienced. In the absence of wave transformation and breaking calculations, an estimate of wave overtopping has been made based on depth-limited nearshore waves, based on the observed water levels (i.e. nearshore waves are dependent on water levels with < 1-year return period). The estimated overtopping was between 0.2 to 9.3 L/s/m for three key profiles. Based on available EurOtop guidance, this range of overtopping would have been characterised by low level overtopping flows which would have been dangerous for unaware pedestrians, and may have caused minor damage to equipment behind the defence. This rate of overtopping is not expected to cause direct damage to the seawall or revetment.

The highest estimated overtopping rate was calculated at Profile 17. This is supported by anecdotal information of the event, which was that Profile 17 experienced the worst case overtopping. Profile 12 also were that also experienced overtopping, however at a reduced rate, and is also considered to be the most often overtopped profile.

Several overtopping trends were consistent with known observations. The analysis suggests the December 2012 event would have resulted in the highest rate of overtopping since 2002 (the extent of available data). This supports the anecdotal information that the event is the largest in recent years, with the most significant rate of overtopping observed historically. While the estimated overtopping trends are generally supported by anecdotal information, a more detailed assessment is required in order to characterise the rate of overtopping between individual defence profiles. This assessment would require a numerical wave model to calculate the wave transformation and breaking processes occurring at the toe of each profile location, with rocky reefs and adjacent headlands expected to have a significant effect on the nearshore wave conditions.

4 Wave overtopping analysis

The Stonehaven coastal frontage currently experiences wave overtopping, such as that observed in the December 2012 event. This section aims to quantify this wave overtopping during extreme events, consider the SoP of the frontage, and investigate potential beach recharge or alterations to the existing defences to reduce the rate of overtopping. This is described in the following subsections:

- **Extreme overtopping assessment:** This section assesses the potential wave overtopping based on extreme coastal conditions and compares these to recommended tolerable discharges to calculate the SoP.
- **Potential changes to defences:** This section investigates changes to the existing beach width (i.e. widening), beach elevation (e.g. raising) or changes in combination with changes to the existing defence crest elevation.

4.1 Extreme overtopping assessment

The wave overtopping resulting from a number of 'design' extreme events has been estimated using available extreme coastal information offshore of Stonehaven (refer to Table 3-3 and Table 3-4). As extreme nearshore wave conditions are not available for this assessment they have been estimated using the depth-limitation approach described in Section 3.5.1. Table 4-1 shows the overtopping under extreme conditions.

Table 4-1: Predicted overtopping rates for extreme sea-level and depth-limited nearshore wave conditions under present-day and climate change scenarios.

Return period	Estimated overtopping rate, present day (L/s/m)			Estimated overtopping rate, including climate change to 2100 (L/s/m)		
	SH05	SH12	SH17	SH05+CC	SH12+CC	SH17+CC
0.2	1.1	0.1	2.0	26.6	4.2	19.9
0.5	2.2	0.2	3.0	37.1	6.2	24.6
1.0	3.0	0.3	3.6	42.9	7.4	26.9
2.0	4.6	0.4	4.7	55.1	9.9	31.2
5.0	7.0	0.6	6.5	74.9	14.2	37.2
10.0	9.4	0.9	8.6	96.6	19.2	43.2
20.0	11.6	1.3	10.4	115.6	23.9	48.1
50.0	15.6	2.0	13.5	157.1	32.7	57.9
100.0	20.0	2.9	16.3	196.3	41.4	66.2
200.0	24.7	3.8	19.0	245.3	50.3	75.6
* For these conditions the still-water level is approaching the crest level of the defence and an estimate is beyond the capabilities of the Neural Network tool. A preliminary estimate has been provided by extrapolating the data. These conditions are expected to result in an extremely high rate of overtopping.						

The estimated overtopping rate was used in conjunction with information on tolerable discharges provided by the EurOtop manual to determine the SoP for each profile. For this assessment a design standard and serviceability target must first be considered. For example, coastal defences can be constructed to offer total protection during specific storm events (e.g. no overtopping during a 1 in X-year event) or to allow a controlled rate of overtopping to occur (e.g. minor overtopping during a 1 in X-year event). For this assessment a nominal overtopping rate of 5 L/s/m has been used, approximately half that experienced in December 2012, and below the onset of structural damage estimated in the EurOtop manual. This level has been used as an arbitrary 'preliminary' target only, and more stringent targets (e.g. 1 L/s/m) or relaxed (e.g. 10-20 L/s/m) may be adopted following further consideration by Aberdeenshire Council.

Profile SH05:

Profile SH05 is considered to have a 1 in 2-year, present day SoP. Under climate change this reduces to below 1 in 0.2-year protection.

During a present day 1 in 200-year coastal event the estimated overtopping rate is 24.7 L/s/m. Using available information on tolerable discharges provided by the EurOtop manual this rate of overtopping is expected to prevent pedestrians from accessing the defence, although trained staff with adequate protection may do so. There is the potential for damage to any equipment positioned behind the defence.

Under a climate change scenario the 1 in 200-year overtopping rate increases to 245.3 L/s/m. For such flows, the defence would be unaccessible to any pedestrians or vehicles, there would be damage to equipment behind the defence, and there may be damage to the paved or armoured seawall promenade behind the defence.

Profile SH12:

Profile SH12 is considered to have a 1 in 200-year, present day SoP. During a present day 1 in 200-year coastal event the estimated overtopping rate is 3.8 L/s/m (approximating the preliminary threshold). Using available information on tolerable discharges provided by the EurOtop manual this rate of overtopping is expected to be dangerous to unaware pedestrians, and cause damage to equipment left behind the defence.

Under a climate change scenario the 1 in 200-year overtopping rate increases to 50.3 L/s/m. For such flows the defence would be dangerous for any unaware pedestrians, vehicular access would only be possible at low speed, and there is the potential for damage to lightly grassed areas behind the seawall.

Profile SH12 is considered to experience the lowest rate of overtopping. However as noted in Section 3.5.2, this is not considered to be reflective of recent conditions. This is considered to be primarily due to the simplified method of estimating nearshore wave conditions, which may be larger due to the limited protection by rocky reefs.

Profile SH17:

Profile SH17 is considered to have a 1 in 2-year, present day SoP. During a present day 1 in 200-year coastal event the estimated overtopping is 19.0 L/s/m. Using available information on tolerable discharges provided by the EurOtop manual this rate of overtopping the defence would be dangerous for any unaware pedestrians, although trained staff would be able to access the defence. No damage would be expected to the defence.

Under a climate change scenario the 1 in 200-year overtopping rate increases to 75.6 L/s/m. For such flows, the defence would be dangerous for any unaware pedestrians, vehicular access would be extremely dangerous, and there is the potential for damage to lightly grassed areas behind the seawall.

4.2 Modifications to existing beach

Potential changes to the existing beach profiles have been investigated to increase the SoP. The effect of altering the beach width (i.e. widening) and the beach elevation (e.g. raising) has been assessed, in addition to changes in combination with an increased defence crest elevation.

4.2.1 Increased beach levels

The potential to increase the existing beach level has been investigated under a 1 in 200-year present day extreme sea level and corresponding depth-limited waves. Under these scenarios each profile was modified to have a 5.0m wide beach at 3.0mAOD as a base-case condition, with a seaward slope of 1 in 5. The beach level was then raised incrementally to 5.0mAOD to consider changes to the overtopping rate. The results are shown in Table 4-2 and presented graphically in Figure 4-1.

Based on the 5.0m wide beach the following beach levels have been estimated in order to reduce the rate of overtopping to the nominal 5 L/s/m target[13].

- Profile SH05: Between 4.0mAOD to 4.25mAOD.
- Profile SH12: Currently meets the nominal overtopping target.
- Profile SH17: No solution identified. The required beach elevation is estimated to be above 5.0mAOD, and is not considered practical.

Table 4-2: Predicted overtopping rates for various beach elevations under a present day 1 in 200-year extreme sea-level and depth-limited nearshore wave scenario.

Beach elevation (5m wide beach)	Profile SH05	Profile SH12	Profile SH17
3.00	10.98	2.42	14.53
3.25	8.33	0.80	13.69
3.50	7.02	0.75	12.92
3.75	6.31	0.73	12.19
4.00	5.82	0.72	11.40
4.25	4.86	0.72	10.63
4.50	4.29	0.73	10.02
4.75	3.81	0.75	9.49
5.00	3.41	0.00	8.87

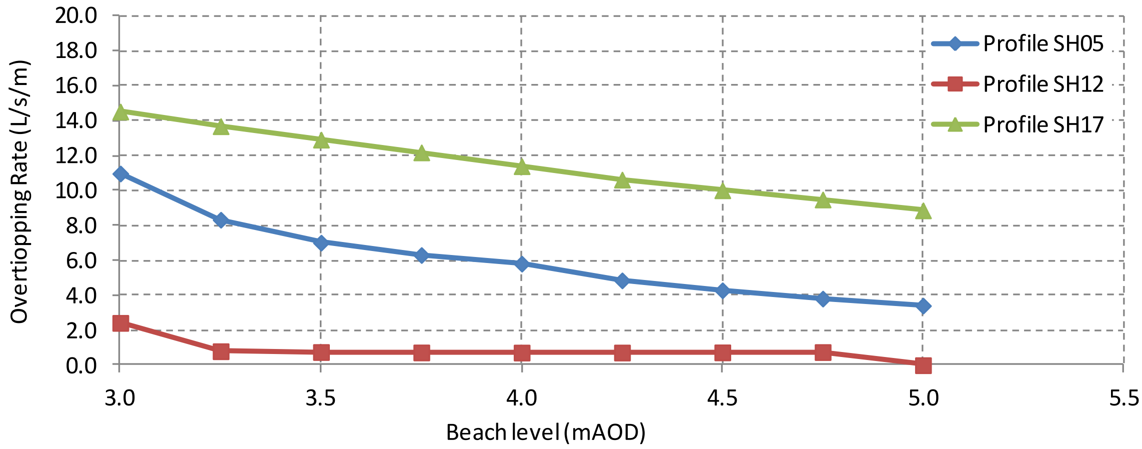

Figure 4-1: Predicted overtopping rates for various beach elevations under 1 in 200-year extreme sea-level and depth-limited nearshore wave conditions under a present-day scenario.

4.2.2 Increased beach width

The potential to increase the existing beach width has been investigated under a 1 in 200-year present day extreme sea level and corresponding depth-limited waves. Under this scenario Profiles SH05 and SH12 was assumed to initially have no beach, whilst Profile SH17 had a 7.0m wide beach as shown in the May 2013 profiles. The beach width was increased in 5m increments to 20m. The results are shown in Table 4-3 and presented graphically in Figure 4-2.

Based on a 3.0mAOD beach elevation the following beach widths have been estimated in order to reduce the rate of overtopping to the nominal 5 L/s/m target[14].

[13] For this assessment a nominal overtopping rate of 5 L/s/m has been used, approximately half that experienced in December 2012. This level has been used as an arbitrary 'preliminary' target only, and more stringent targets (e.g. 1 L/s/m) or relaxed (e.g. 10-20 L/s/m) may be adopted following further consideration by Aberdeenshire Council.

- Profile SH05: Between 10 to 15m wide.
- Profile SH12: Currently meets the nominal overtopping target.
- Profile SH17: No solution identified. The required beach width is estimated to be beyond 20m, and beyond the limitations of the Neural Network calculation tool.

Table 4-3: Predicted overtopping rates for various beach widths under a present day 1 in 200-year extreme sea-level and depth-limited nearshore wave scenario.

Beach width	Profile SH05	Profile SH12	Profile SH17
0.00	24.74	3.79	-
5.00	14.54	1.19	(7m) 18.95
10.00	8.95	0.51	18.51
15.00	3.35	0.27	13.97
20.00	0.00	0.17	8.81

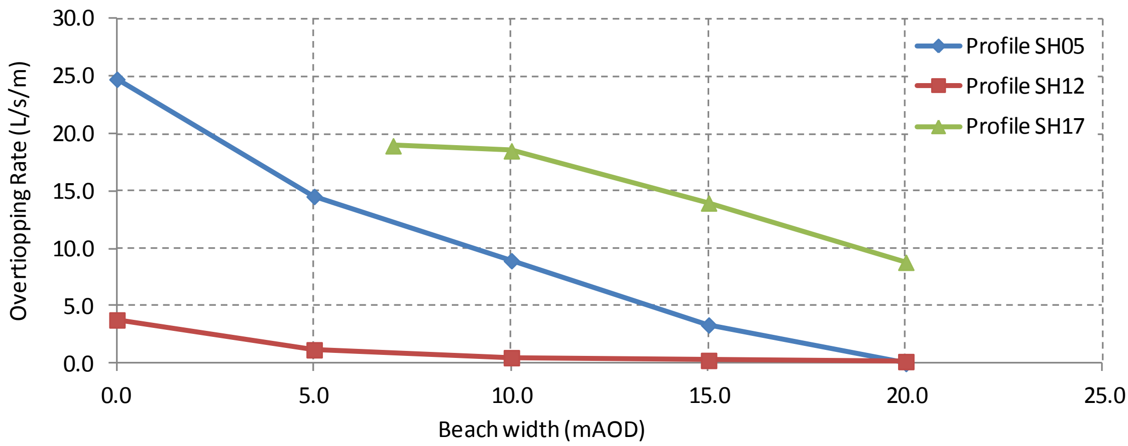

Figure 4-2: Predicted overtopping rates for various beach widths under 1 in 200-year extreme sea-level and depth-limited nearshore wave conditions under a present-day scenario.

4.2.3 Combined beach and defence alterations

The potential to widen the existing beach in addition to altering the existing crest level of the defences was investigated under a 1 in 200-year extreme sea level and corresponding depth-limited waves, under both present day and climate change scenarios. Under these scenarios Profiles SH05 and SH12 was assumed to initially have no beach, whilst Profile SH17 had a 7.0m wide beach as shown in the May 2013 profiles. The beach width was increased in 5m increments to 20m, which was repeated under incremental raises to the defence crest level. The results are shown in Table 4-4 to Table 4-6 for Profiles SH05, SH12 and SH17 respectively.

The scenarios show a number of potential combinations to meet the nominal 5 L/s/m target. For each profile the following changes are considered to be the most practical to meet present day and climate change scenarios.

- Profile SH05: For the present day 200-year scenario an existing crest level and a beach width between 10 to 15m. Under a 200-year climate change a 2m increase to the crest level and a 15m wide beach is required.
- Profile SH12: The profile currently meets the 200-year present day overtopping target. Under a 200-year climate change scenario either a 20m wide beach, or a 0.5m increase to the crest level in addition to a 10m wide beach is required.

[14] For this assessment a nominal overtopping rate of 5 L/s/m has been used, approximately half that experienced in December 2012. This level has been used as an arbitrary 'preliminary' target only, and more stringent targets (e.g. 1 L/s/m) or relaxed (e.g. 10-20 L/s/m) may be adopted following further consideration by Aberdeenshire Council.

- Profile SH17: For the present day 200-year scenario an increase of 0.5m to the existing defence crest level and a 15m wide beach is required, or a 1m increase to the defence crest level. Under a 200-year climate change a 2m increase to the crest level is required.

Table 4-4: Predicted overtopping rates for Profile SH05 for various beach widths and defence crest levels under 1 in 200-year extreme sea-level and depth-limited nearshore wave conditions under present-day and climate change scenarios.

Crest height (mAOD)	Beach width (m)	200-year present day overtopping rate (L/s/m)	200-year + CC overtopping rate (L/s/m)
Crest wall 4.4mAOD	**0.0**	24.7	245.3
	5.0	14.5	105.9
	10.0	8.9	101.9
	15.0	3.4	97.4
	20.0	0.0	84.5
Crest wall 4.9mAOD (+ 0.50m)	**0.0**	10.5	98.3
	5.0	3.2	88.6
	10.0	2.0	84.9
	15.0	0.0	81.3
	20.0	0.0	70.5
Crest wall 5.4mAOD (+1.0m)	**0.0**	4.4	46.9
	5.0	0.9	40.9
	10.0	0.4	39.7
	15.0	0.0	37.1
	20.0	0.0	32.3
Crest wall 5.9mAOD (+ 1.5m)	**0.0**	1.9	20.6
	5.0	0.3	15.8
	10.0	0.1	15.8
	15.0	0.0	14.3
	20.0	0.0	12.4
Crest wall 6.4mAOD (+ 2.0m)	**0.0**	0.8	9.1
	5.0	0.2	6.2
	10.0	0.0	5.8
	15.0	0.0	5.0
	20.0	0.0	4.3

Table 4-5: Predicted overtopping rates for Profile SH12 for various beach widths and defence crest levels under 1 in 200-year extreme sea-level and depth-limited nearshore wave conditions under present-day and climate change scenarios.

Crest height (mAOD)	Beach width (m)	200-year present day overtopping rate (L/s/m)	200-year + CC overtopping rate (L/s/m)
Crest wall 5.75mAOD	**0.0**	3.8	50.3
	5.0	1.2	21.8
	10.0	0.5	11.3
	15.0	0.3	6.3
	20.0	0.2	3.8
Crest wall 6.25mAOD (+ 0.50m)	**0.0**	1.5	22.0
	5.0	0.5	9.5
	10.0	0.2	4.8
	15.0	0.1	2.6
	20.0	0.1	1.6
Crest wall 6.75mAOD (+1.0m)	**0.0**	0.6	9.8
	5.0	0.2	4.2
	10.0	0.1	2.1
	15.0	0.1	1.1
	20.0	0.0	0.7
Crest wall 7.25mAOD (+ 1.5m)	**0.0**	0.3	4.6
	5.0	0.1	2.0
	10.0	0.1	1.0
	15.0	0.0	0.5
	20.0	0.0	0.3
Crest wall 7.75mAOD (+ 2.0m)	**0.0**	0.2	2.3
	5.0	0.1	1.0
	10.0	0.0	0.5
	15.0	0.0	0.3
	20.0	0.0	0.2

Table 4-6: Predicted overtopping rates for Profile SH17 for various beach widths and defence crest levels under 1 in 200-year extreme sea-level and depth-limited nearshore wave conditions under present-day and climate change scenarios.

Crest height (mAOD)	Beach width (m)	200-year present day overtopping rate (L/s/m)	200-year + CC overtopping rate (L/s/m)
Crest wall 4.70mAOD	**7.0***	19.0	75.6
	10.0	18.5	43.4
	15.0	14.0	43.4
	20.0	8.8	28.0
Crest wall 5.20mAOD (+ 0.50m)	**7.0***	7.2	36.2
	10.0	7.2	36.1
	15.0	5.7	31.2
	20.0	3.6	23.1
Crest wall 5.70mAOD (+1.0m)	**7.0***	2.9	19.1
	10.0	2.9	19.4
	15.0	2.4	17.5
	20.0	1.6	13.1
Crest wall 6.20mAOD (+ 1.5m)	**7.0***	1.2	9.5
	10.0	1.2	9.7
	15.0	1.0	9.1
	20.0	0.7	6.9
Crest wall 6.70mAOD (+ 2.0m)	**7.0***	0.5	4.6
	10.0	0.5	4.7
	15.0	0.4	4.4
	20.0	0.3	3.5

* Based on the May 20013 topography an existing beach/berm width of 7m is present, which has been used as the base-case scenario.

4.3 Summary of overtopping analysis

Recent events such as that of December 2012 have shown that the Stonehaven coastal frontage currently experiences wave overtopping during extreme conditions. The rate of overtopping has been calculated under a number of 'design' extreme events to quantify the existing SoP, based on a nominal overtopping rate of 5 L/s/m, approximately half that experienced in December 2012. This level has been used as an arbitrary 'preliminary' target only, and more stringent targets (e.g. 1 L/s/m) or relaxed (e.g. 10-20 L/s/m) may be adopted following further consideration by Aberdeenshire Council. In the absence of nearshore wave information this assessment was made using estimated depth-limited waves conditions.

The assessment has indicated that Profile SH05 and SH17 are considered to have a 1 in 2-year SoP and Profile SH12 is considered to have a 1 in 200-year SoP under present day conditions. In contrary to this assessment, Profile SH12 has been observed to have the most frequent overtopping. This is considered to be due primarily to the simplified method of estimating nearshore wave conditions.

Potential changes to the existing beach width and elevation has been investigated. In order to meet the nominal 5 L/s/m SoP target, the following changes are required.

- Profile SH05: An increased beach elevation between 4.0mAOD to 4.25mAOD, or an widened beach between 10m to 15m.
- Profile SH12: Currently meets the nominal overtopping target under the simplified depth-limited assessment method.
- Profile SH17: A beach width of over 20m would be required (considered unpractical). An alternative is to increase the existing defence crest level by 0.5m and implement a 15m wide beach.

Further assessment was made to consider potential in-combination changes to both the beach width and the existing crest level of coastal defences. A number of potential combinations have been identified, with the analysis showing that small increases to the defence crest level can minimise the required beach width. Under a climate change scenario all profiles required an increase to the crest level, ranging between 0.5 to 2m in conjunction with a widened beach.

It is important to note that all assessments have been based on simplified wave calculation methods that do not take into consideration the important nearshore wave transformation and breaking processes that occur due to the rocky reefs adjacent to Stonehaven. As a result, the estimated overtopping differs from anecdotal reports, in particular at Profile SH12, which is considered to have the lowest SoP. Aerial photographs suggest this profile is situated in a break between the extensive rock reefs, which may allow larger waves to propagate to the defence, leading to a larger rate of overtopping than estimated. It is recommended that refinement of this overtopping assessment is undertaken using more formal wave transformation modelling and incorporating accurate nearshore bathymetry. Aberdeenshire Council have currently engaged consultants to undertake nearshore wave monitoring and to develop a new wave transformation model which could be used for this purpose, thereby reducing one of the most significant uncertainties associated with the study.

5 Coastal management advice

5.1 Introduction

In the previous chapters, the potential to implement a beach recharge scheme was investigated to improve the SoP of the Stonehaven frontage. This chapter considers the potential beach recharge in more detail, assessing the historic beach management activities, the requirements to maintain a minimum beach width and the cost-benefits of implementing a scheme. This Section includes the subsections:

- **Historic beach management activities:** Reviews the historic management and engineering activities undertaken along the frontage.
- **Assessment of minimum beach width:** Assesses the potential for maintaining a minimum beach width adjacent to the coastal defences.
- **High level cost analysis:** Provides a high level assessment of the required volume of sediment and the likely costs for the beach recharge scheme.
- **Coastal flood warning:** Considers the logistics of implementing a local coastal flood warning system to further reduce the existing coastal flood risk.
- **Resilience measures:** Considers the benefits and effectiveness of property level protection in reducing the flood risk.

5.2 Historic beach management activities

A number of coastal process assessments have been undertaken for Stonehaven, with the most relevant to this study listed below:

- JBA (2012) Stonehaven River Carron Flood Alleviation Study;
- Canterbury City Council (2013) Topographic Baseline Survey Report 2013;
- HR Wallingford (2009) Stonehaven, Inverbervie and Rosehearty Beach Management, Technical Note DDM6256-01;
- HR Wallingford (1999) Stonehaven Bay, Aberdeenshire, A Strategic Review of Beaches and Coastal Defences, Report EX 4017;
- HR Wallingford (1998) Stonehaven Seawall, Aberdeenshire, Feasibility Study of Improvements, EX 3731.

A review of these studies was undertaken to summarise the coastal processes and the historic beach management strategies. The main issues recognised across the studies were the frequent occurrence of wave overtopping and minor coastal flooding. The previous studies recognise that the River Cowie and Carron both contribute to this problem due to their potential to alter the profile of the beach and therefore the SoP can fluctuate.

The primary coastal management activity occurring at Stonehaven is the recycling of beach sediment. Beach recycling involves moving sediment from areas of accumulation to areas of erosion. Table 5-1 shows the beach recycling operations carried out at Stonehaven since 2001, and indicates a decreasing frequency in recent years. Other historic management activities include the construction of training wall at the mouth of River Cowie, and a rock toe adjacent to the promenade seawall near the amusements/Green Hut. The latter was constructed to prevent the loss of sediment through the seawall and not specifically to reduce overtopping.

Table 5-1: Beach recycling operations carried out at Stonehaven. (source: HR Wallingford 2009 and Aberdeenshire Council)

	Collected (tonnes)		Deposited (tonnes)		
Year	From mouth of Cowie	From mouth of Carron	South of mouth of Carron	North of stepped seawall	South of mouth of Cowie
2001	2000	0	2000	0	0
2002	2000	0	2000	0	0
2003	2000	0	2000	0	0
2004	2000	0	2000	0	0
2005	2000	0	2000	0	0
2006	2000	0	500*	2000	0
2007	2000	150	2150	0	0
2008	2000[t]	150	2150	0	0
2009	4350	0	4000	0	350[!]
2010	3000	0	3000	0	0
2011	1500	0	1500	0	0
2012	1000	0	1000	0	0
2013	0	0	0	0	0
2014	0[£]	0[£]	0[£]	0[£]	0[£]

Notes:

* Shingle placed over manhole cover just north of groyne at Carron.

[t] c150 tonnes of rock armour transferred from groyne at the mouth of the Cowie to improve groyne at mouth of the Carron.

[!] Shingle placed c50m south of the mouth of the Cowie.

[£] There has not been any recharge as of 27/03/2014.

5.3 Sediment changes

Consecutive beach surveys undertaken between December 2008 and May 2013 have allowed the sediment changes along the Stonehaven coastline to be assessed. The following summarises the changes to four key sections of the coastline as shown in Figure 5-1.

Figure 5-1: Beach sections 1 to 4 assessed through 2008 and 2014 volumetric surveys.

5.3.1 Section One

Section One extends 300m south from the cliffs at Cowie to Amy Row and includes the overtopping profile SH05. The area is characterised by low beach levels, with the beach crest height often not exceeding 2.2mAOD, with an extensive rock platform. This area is considered to have experienced minor changes only, gaining 162m^3 over the five year period.

5.3.2 Section Two

Section Two extends south to the River Cowie, and includes overtopping profile SH12. The area is characterised by a beach backed by a rear seawall to the north and a stepped revetment to the south. Fluctuating beach levels have historically been observed, varying in response to changing wave conditions. However, based on the available surveys the observed changes show approximately 4,000m^3 accumulation of sediment, greatest towards the northern end of the stepped revetment where the profile has increased vertically by around 1m. At the southern end of the stepped revetment there is negligible change in beach crest levels - surprisingly as the short concrete groyne to the north of the Cowie was built to prevent captured shingle travelling south. Further seaward there has been an overall sediment loss observed in the lower beach attributed to changes in mobile beach sand.

5.3.3 Section Three

Section Three extends between the River Cowie to the River Carron. The major transport of sediment to the area is considered to be from the north, however there is a trend of shingle being transported back towards the mouth of the Cowie, indicating the potential for northerly directed sediment transport. The addition of a short groyne to the south of the outlet has been proposed, which could prevent sediment from blocking the channel, however no detailed assessments have been made. Accumulated sediment at this location has traditionally been a key source for beach recycling, with approximately 26,000 tonnes removed since 2001.

The beaches between the River Cowie and Carron have generally accumulated, in particular to the north, where beach levels have increased. Losses are observed in the nearshore area, again attributed to mobile beach sediment or onshore sediment transport. There is an overall loss of sediment towards the southern end of the beach. The trend of net accumulation to the north and losses to the south may be the result of beach rotation, as sediment is moved to the northern end of the beach due to a southerly shift in the dominant wave direction.

5.3.4 Section Four

Section Four extends south from the River Carron, with the sediment cell bounded by the training walls at the river outlet and the southern headland. The comparison of survey data shows a net loss of 333m^3 and a retreat of the beach face by 2.5m. This area has received 25,000 tonnes of sediment through beach recycling since 2001, and greater erosion is considered likely if this supply was to stop. The addition of several short groynes has been proposed to help stabilise the deposited sediment, which would limit the northerly transport back towards the Carron outlet, and may allow greater time between mechanical beach recycling works by Aberdeenshire Council.

5.4 Maintaining minimum beach widths

Based on the information collated in Table 5-1 and the comparison of beach surveys between 2008 and 2013 there has generally been a trend of sediment accumulation at the Stonehaven beaches, with some minor loss of sediment at the southern end. Beach recycling has historically peaked in 2009 when over 4000 tonnes of sediment was recycled from the mouth of the River Cowie and deposited south of the River Cowie, which has reduced to no recycling being undertaken since 2013. While these trends suggest the beach levels have been generally stable, they show the beach to the south of the River Carron has lost sediment, despite the addition of around 5000m^3 being recycled to the area since 2008. It is expected that this loss would be significantly larger if recycling was to stop. These trends support previous recommendations for the addition of two short groynes of dimensions similar to the northern River Carron training wall. The groynes would help stabilise the sediment deposited at this area, and may allow greater time between mechanical beach recycling works by Aberdeenshire Council.

5.4.1 Short-term losses

The sediment trends between 2008 and 2014 consider the net beach changes rather than capturing a particular extreme event. Even following recycling or recharge events there is the potential that a large storm will cause erosion of the beach over a short-term timeframe (i.e. hours). During a storm increased water levels and large waves have the potential to erode beach sediment which is transported offshore. The resulting eroded profile experiences deeper water, which will allow larger waves to reach the coastal defence and result in higher overtopping rates. It is important to include these sediment losses into any beach recharge schemes to ensure the beach is adequately sized during a storm event.

Using XBeach-G the potential storm-based losses of a recharged shingle beach has been assessed. X-Beach-G is a cross-shore sediment transport model developed specifically for shingle beaches through a joint collaboration between Plymouth University and Deltares. Compared to sandy beaches, relatively little is known about the processes occurring on shingle beaches, particularly during storms. The XBeach-G model incorporates the latest research, including processes for wave-transformation and breaking, wave run-up, depth-averaged currents, sediment transport and groundwater interaction, and is validated through a number of recent and historic events.

The XBeach-G model was used to model a 1 in 200-year present day extreme sea level and corresponding depth-limited nearshore waves. Several recharged beach levels were investigated, following the general profile of SH12 and SH17 to a depth of -2.0m. Beach levels included:

- a 3.0mAOD beach based on the existing berm level from available profile data,
- a 3.75mAOD beach based on the 1 in 200-year present day extreme sea level, plus a nominal 0.5m freeboard allowance,
- a 4.4mAOD beach based on the 1 in 200-year climate change extreme sea level, plus a nominal 0.5m freeboard allowance.

A 100 metre wide beach crest was modelled for each scenario, representing an 'infinitly' wide system that would erode unhindered, allowing the maximum erosion to be recorded. The resulting changes to the beach profile are shown in Figure 5-2. The modelling indicates a horizontal beach loss of 16m, 9m and 5m is expected for a 3.0mAOD, 3.75mAOD and 4.4mAOD beach elevation. These results show that less erosion is expected for higher beaches during a storm. This trend, combined with a lower rate of overtopping for higher beaches, shows the potential for a narrower beach to reduce overtopping if a higher beach was adopted.

A key assumption of the wave overtopping modelling and specification of beach widths is that the recharged beach is present during the extreme event, and has not reduced in size due to long-term recession or storm-based erosion events. It is therefore important that any recharged beach includes allowances for storm-based erosion, which could be between 5 to 16m loss of width for a relatively low to high beach respectively.

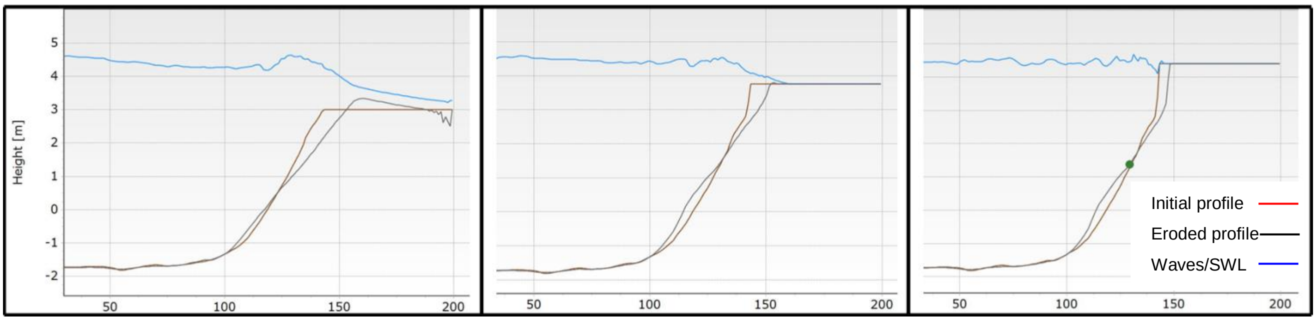

Figure 5-2: Predicted erosion for various beach elevations under 1 in 200-year extreme sea-level and depth-limited nearshore wave conditions under a present-day scenario.

5.5 High level assessment of beach recharge suitability

A high-level assessment has been undertaken to assess the potential for beach recharge. This analysis required a preliminary assessment on the required volume and cost of the design beach. This assessment is considered preliminary, as detailed information is unavailable for many of the parameters required for complete design. Therefore several assumptions have been made, described in the following subsections.

5.5.1 Crest elevation and width assessment

The high level cost assessment has been completed for two difference scenarios using an existing (3.0mAOD) and higher (4.5mAOD) beach level. These include:

Scenario 1:

- Profile SH05: A 31m wide beach at 3.0mAOD. This is made from a 15m wide required post-storm beach to limit overtopping, and allows for 16m of storm-based erosion.
- Profile SH12: Currently meets the nominal overtopping target under the simplified depth-limited assessment method.
- Profile SH17: A 0.5m increase to the existing defence crest level and a 31m wide beach at 3.0mAOD. This is made from a 15m wide required post-storm beach to limit overtopping, and allows for 16m of storm-based erosion.

Scenario 2:

- Profile SH05: A 9m wide beach at 4.5mAOD. This is made from a 4m wide required post-storm beach to limit overtopping, and allows for 5m of storm-based erosion.
- Profile SH12: Currently meets the nominal overtopping target under the simplified depth-limited assessment method.
- Profile SH17: A 0.5m increase to the existing defence crest level and a 10m wide beach at 4.5mAOD. This is made from the existing 7m wide required post-storm beach to limit overtopping, and allows for 5m of storm-based erosion.

5.5.2 Design profile and required nourishment

The design profile has been developed based on a beach elevation of 3.0mAOD and 4.5mAOD, the required width to account for storm losses whilst maintaining the required post-storm beach width, and a seaward slope of 1(v):10(h). The beach profiles for profiles SH05 and SH17 are presented in Figure 5-3 and Figure 5-4. The required recharge was calculated over the two frontage lengths determined through aerial photography to be 280 and 400m respectively. Recharge was not assessed at profile SH12, as the overtopping calculations undertaken in this assessment indicated the profile currently meets the 200-year SoP.

The volume required within the bay was calculated to be approximately 41,000m^3 for Scenario 1, and 44,000m^3 for Scenario 2. During placement of beach the density of the material can often be lower than would occur on the native beach – a process known as bulking. On a well-established beach, fines migrate to the core and fill interstices in the structure. Over the course of three or four tides, large losses of volume can occur as the density increases under wave action. In addition, recharged material (especially dredged aggregate) contains a significant portion of fines. These fines are often lost from the beach as they are too small for the hydrodynamic environment. These losses have been accounted for using a bias of 20%[15]. Applying this to the required volume produces a total required volume of between 49,000m^3 and 53,000m^3.

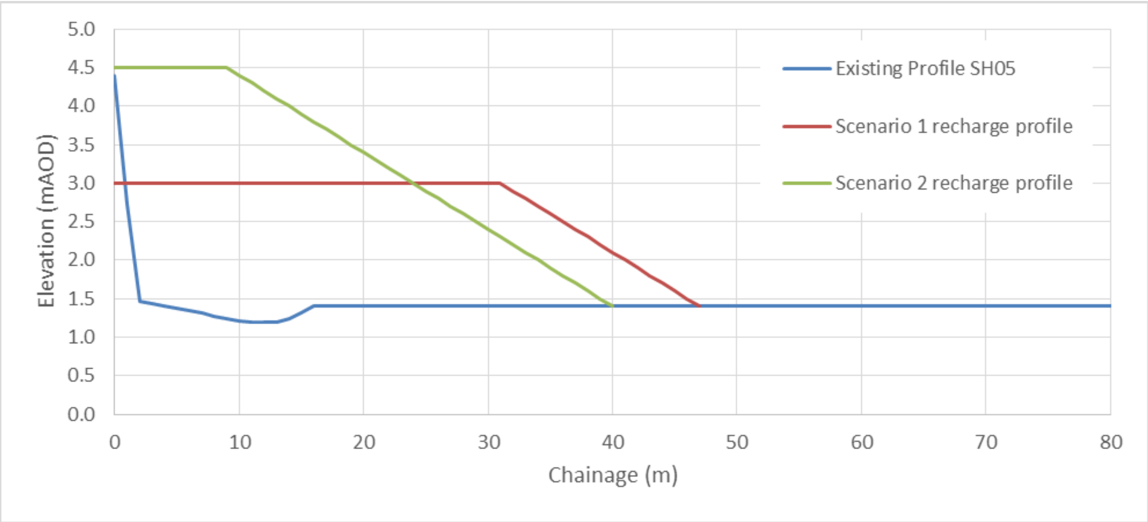

Figure 5-3: Existing beach profile SH05 and required recharge under Scenario 1 (existing beach) and 2 (raised beach).

[15] CIRIA (2010), The Beach Management Manual (second edition)

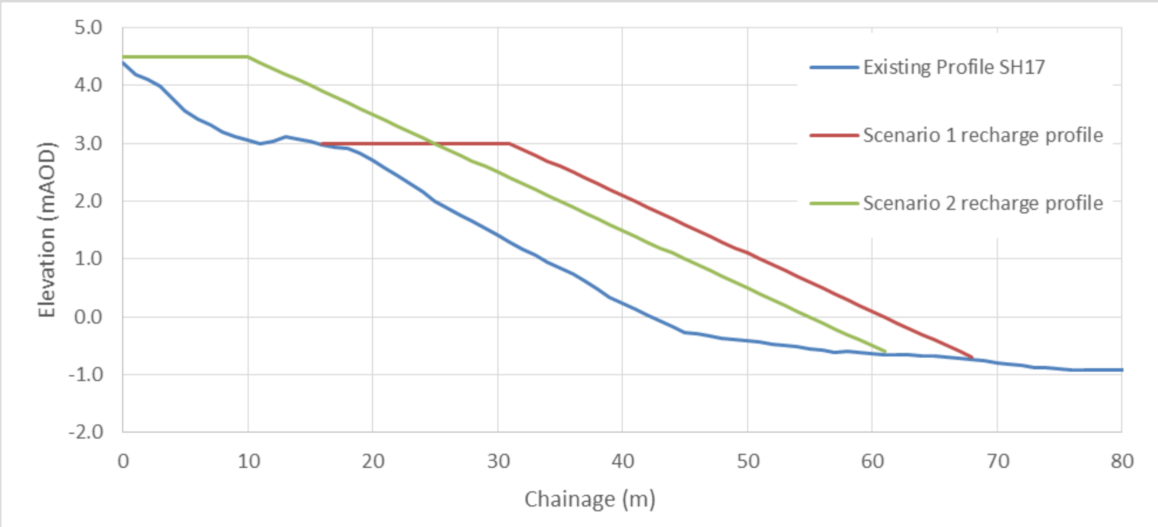

Figure 5-4: Existing beach profile SH17 and required recharge under Scenario 1 (existing beach) and 2 (raised beach).

5.5.3 Maintenance replenishment

The comparison of beach surveys between 2008 and 2013 indicate a general trend of sediment accumulation at the Stonehaven beaches, with some minor loss of sediment at the southern end. However, these trends cannot be confirmed without an assessment of the long-term processes in the area and potential recharge maintenance costs have been included within the cost-benefit analysis. For this high level assessment an annual replenishment rate of 5% has been used, which is assumed to be a combination of both cross shore and longshore losses.

5.5.4 Costs

In order to provide a construction cost estimate, a marine construction contractor has been engaged to provide advice on the pricing of the beach recharge scheme. Two costing options have been presented, depending if sediment was sourced from onshore or offshore locations. Locally sourced sediment is charged at a rate of £27/m^3 and offshore sediment at £49/m^3. The low and high beach scenarios have been assessed using each sediment cost. Consequently, the beach recharge rates provided are considered an initial estimate only, which can be refined when the dredge area is known. If this scheme is taken forward it is recommended that a further dredge fill assessment is conducted to identify potential sources of recharge sediment at nearby and offshore locations and a dredging contractor is engaged to develop a dredging strategy which will vary due to the location of source material.

In addition to the beach recharge, Profile SH17 requires an increase of 0.5m to the defence (sea wall) crest level. This was estimated using a rate of £1,440/m as given in the Flood Risk Management Estimating Guide[16] for walls under 1.2m high.

Table 5-2: Cost estimates for beach recharge and defence upgrades

Scenario	Sediment source	Required recharge (m3)	Rate (£/m3)	Total cost (£)	Annual replenishment cost (£)	Infrastructure upgrades (£)
1A	Local	40,651	27	£1,097,582	£54,879	£560,000
1B	Offshore	40,651	49	£1,991,909	£99,595	£560,000
2A	Local	44,650	27	£1,205,539	£60,277	£560,000
2B	Offshore	44,650	49	£2,187,830	£109,392	£560,000

[16] Environment Agency (2010), Flood risk management estimating guide - update 2010.

5.6 Flood damages avoided by proposed options

In the absence of detailed overtopping flood maps there are two approaches used to estimate coastal flood damages. These are as follows:

- Weighted Annual Average Damage (WAAD) calculations based on the predicted number of properties at risk of flooding.
- Write off damages assuming properties are lost or flooded too frequently.

Both approaches have been assessed for the purpose of this assessment.

5.6.1 Weighted annual average damages

Where the appraiser has little or no understanding of the potential overtopping flood depths the WAAD approach is typically used. Where the number of properties likely to be damaged can be estimated (e.g. along a frontage or within a nominal floodplain) the WAAD can be estimated based on information contained within the Multi Coloured Manual (MCM)[17].

The MCM provides estimates for the annual average damage (AAD) expected to properties within a number of flood risk bands. The AAD for a residential property with no flood warning and no flood protection is £4,728, which decreases for properties with a lower flood risk, higher SoPs and different levels of flood warning, which will give residents early warning of extreme events and will allow residents to take necessary precautions (such as moving portable property). A separate set of WAADs are available for non-residential properties.

The total number of properties affected by wave overtopping at Stonehaven has been visually assessed along the coastal frontage, considered to be as follows:

- 32 residential properties.
- 6 mainly retail properties (north of the Cowie).

The residential properties are assumed to have a five year SoP, and are considered to have a WAAD value of £2,828 per property. Several non-residential properties have been included in the analysis based on five retail and one public building. Allowances have been made for clean-up costs (ranging between £1,000 and £10,000 for a 10-year to 200-year return period event), vehicle damage (£3,100 per property for 28% of the properties at risk) and evacuation losses (£4,121 per property), and emergency services costs (56% of total damages) included as per standard MCM recommendations.

5.6.1.1 Damages

The total flood damages have been calculated to be £5.79M, based on the following breakdown:

- Direct property flood damages £5.33M
- Emergency services £0.3M
- Other flood damages £0.17M

5.6.2 Write off damages

In coastal environments where flooding occurs at a high frequency a second approach is to assume the 'walkaway' scenario where repeat damages exceed the 'write-off' value of properties impacted. Assets written off are assumed to flood too frequently to be useable for their particular purpose. Whilst this is not currently the case in Stonehaven, the frequency of overtopping is expected to increase under the impact of climate change. Properties predicted to flood more than once every three years (on average) are usually considered to be written-off unless they are flood resilient or water compatible. This is because it is unlikely that there is sufficient time available for the property to be repaired and returned to full use following the previous flood before the next flooding event occurs. As a result, repairing the property would be uneconomical.

Under this approach the write-off values are capped at the losses occurred and the market value of the assets at risk. The economic argument is that the most economically efficient response to

[17] Penning-Rowsell, Edmund C. and Priest, Sally J. and Parker, Dennis J. and Morris, Joe and Tunstall, Sylvia M. and Viviattene, Christophe and Chatterton, John and Owen, Damon (2013) *Flood and coastal erosion risk management: a manual for economic appraisal.* Routledge, Taylor & Francis, London, UK. ISBN 9780415815154

repeat flooding is abandonment and thus the risk free market values should be used. Present Value damages for properties impacted are capped at the total market value.

As with other flood risk appraisals, the economic losses due to erosion or frequent flooding should be calculated by considering the value without intervention (the baseline or *Do Nothing* option) and with intervention (*Do Something*). The more appropriate approach for coastal abandonment cases is to estimate the benefit of maintaining a defence line to delay encroachment, erosion or frequent overtopping damages for a defined number of years using market rates of the properties at risk. The formula used is as follows:

PVdn = MV (1/(1+r)p)

minus

PVds = MV (1/(1+r)p+s)

Where:

PVdn = Present Value without scheme (Do Nothing)

PVds = Present Value with scheme (Do Something)

MV = Market Value

r = treasury discount rate (3.5% with reductions in later years)

p = the year the property is lost

s = life of scheme (delay to erosion, encroachment or frequent overtopping damages)

The above approach is defined in the MCM[18] and is the recommended approach for coastal erosion cases. The calculation assumes that the greater the design life of the scheme the greater the benefits, although not proportionately because losses in the future are discounted more heavily.

The above approach does not take into account any damage up to the point of anticipated write-off for the *Do Nothing* case. Nor does it account for wave overtopping damages for the design case. This is because the approach taken is technically for coastal erosion and wave overtopping scenarios where a small degree of overtopping is anticipated and tolerated by the design.

The economic assessment and flood damage calculation relies on realistic risk free market values of properties. Property valuations have been recalculated for properties in Stonehaven using updated market values. Property valuations are estimated to be £130,000 for residential properties, and non-residential properties have been estimated from rateable values gathered from the Scottish Assessors Associated (SAA) website.

5.6.2.1 Damages

The maximum flood damage that could occur as a result of frequent and reoccurring flooding to all properties along the frontage of Stonehaven (i.e. the total write off value of any properties affected) is £4.0M. This is valid for a scenario where all property is lost in the first year with repeated flooded thereafter.

The calculation of *Do Nothing* damages requires certain assumptions with regard to the onset of flooding and the write-off of properties. A range of scenarios have been considered to test the sensitivity of the assumptions. These are:

- Write off of all properties in year 0
- Write off of all properties in year 5
- Write off of all properties in year 15

The following assumptions have been used to obtain present values:

- Present values have been based on the assumption of a 100-year financial period.
- Discounting of future costs to present values have been based on HM Treasury discount rates (Green Book). These are time varying discount rates starting at 3.5% for the first 30 years, reducing to 3% at year 31 and 2.5% at year 76.
- The present value factor based on these discount rates over a 100 year period is 29.813.

[18] Refer to Chapter 7 of the Multi-Coloured Manual

A discontinued FCDPAG[19] spreadsheet has been used for this type of flood damage assessment. This spreadsheet has been amended to calculate flood damages for the baseline assessment for the scenarios tested. Modifications are needed to cater for the variable discount rates. The *Do Nothing* damages for the range of scenarios listed are provided in the Table 5-3 below.

Table 5-3: Do Nothing damages for a range of flooding onset and write off assumptions (£k)

Assumption	Damages (£m)
Write off of all properties in year 0	£4.00
Write off of all properties in year 5	£3.37
Write off of all properties in year 15	£2.40
Delayed write off of all properties by 100 years	£0.00
Damages avoided	Between £2.40 and £4.00

5.6.2.2 Damages Avoided

The damages avoided for these options depend on the assumptions regarding the timing of flooding and property write-off, but are likely to be in the range of £4.0M to £2.4M.

5.7 Consideration of beach recharge economics

5.7.1 Cost benefit results

The cost benefit results for the WAAD and write off methodologies are presented in Table 5-4 and Table 5-5 respectively. The damages have been compared against the costs for each option assessed. Present value costs assume capital costs occur in year 0 and annual maintenance costs occur annually for the 100 year financial period. An optimism bias of 30% has also been added to the present value costs.

Both methodologies give similar results providing confidence in the methods applied. The results suggest that the economic viability of the scheme is only viable for the lower cost local sediment supply assumptions. However, if the 5% allowance for maintenance recharge is removed from the analysis each option is considered to be beneficial, with the cost-benefit ratio increasing to between approximately 1.5 and 2.5. Further consideration of the maintenance recharge needs to be considered before these figures are used.

5.7.2 Consideration of results

The approach undertaken for the cost-benefit assessment follows the *Overview Appraisal* methodology outlined in the MCM. However, even at this overview stage several assumptions have been made which have had an impact on the final results. Of the most importance to the assessment are the estimated damages (which could not be based on accurate overtopping rates), the required beach width (which was based on estimated overtopping rates only), the unknown source of sediment and the estimated annual maintenance recharge requirements.

While the cost-benefit results are considered to be above parity, they are considered finely balanced and very sensitive to the assumptions and limitations used throughout this study. In particular the MCM recommends that considerable effort is put into determining the extent and annual probabilities of flooding (i.e in this case the overtopping rate and resulting inundation) and the flooding at which damage begins. It is recommended that these elements are addressed in a more detailed study, which uses numerical modelling to support the decision making process.

[19] Flood and Coastal Defence Project Appraisal Guidance (http://archive.defra.gov.uk/environment/flooding/policy/guidance/project-appraisal.htm)

Table 5-4: Cost benefit results based on the WAAD methodology

		Costs and benefits £m			
	No Project	Option 1a	Option 1b	Option 2a	Option 2b
PV costs PVc	0.00	3.24	5.42	3.57	5.90
Optimism Bias (OB)	0.00	0.97	1.63	1.07	1.77
PVc including OB	0.00	4.21	7.05	4.65	7.67
Total PV damages £	5.79	0.00	0.00	0.00	0.00
Total PV benefits £		5.79	5.79	5.79	5.79
Net Present Value NPV		1.58	-1.25	1.15	-1.88
Average benefit/cost ratio BCR		1.38	0.82	1.25	0.76
Where Scenario 1 considers a beach recharged at 3.0mAOD using (1a) locally sourced sediment and (1b) offshore sediment, and Scenario 2 considers a beach recharged at 4.5mAOD using (2a) locally sourced sediment and (2b) offshore sediment.					

Table 5-5: Cost benefit results based on the write-off methodology

		Costs and benefits £m			
	No Project	Option 1a	Option 1b	Option 2a	Option 2b
PV costs PVc	0.00	3.24	5.42	3.57	5.90
Optimism Bias	0.00	0.97	1.63	1.07	1.77
PVc including OB	0.00	4.21	7.05	4.65	7.67
PV damage PVd	4.00	0.00	0.00	0.00	0.00
PV damage avoided		4.00	4.00	4.00	4.00
Net Present Value NPV		0.76	-1.42	0.43	-1.90
Average benefit/cost ratio		1.20	0.70	1.10	0.70
Where Scenario 1 considers a beach recharged at 3.0mAOD using (1a) locally sourced sediment and (1b) offshore sediment, and Scenario 2 considers a beach recharged at 4.5mAOD using (2a) locally sourced sediment and (2b) offshore sediment.					

5.8 Logistics of implementing a local coastal flood warning system

This report has reviewed the history of flooding at Stonehaven, calculated the required improvements to the coastal frontage to protect against extreme events now and under the influence of climate change, and has identified initial engineering works that would offer immediate protection. However, it is realised that the design, planning and construction of any engineering works would be undertaken under a staged programme, influenced by Council budget prioritisation and investment cycles. Furthermore, even after engineering improvements are undertaken, the entire risk of coastal flooding may not be eliminated, for economical or practicality reasons. Therefore the development of a coastal Flood Warning System (FWS) represents a cost effective manner in which the residual risk associated with coastal flooding can be better managed over the short term.

One of the key elements of work required to contribute to a FWS is the development of a suite of numerical models that represent the weather and sea-state conditions that drive coastal flood risk. The models can then be used to direct and inform incident management before and during an event. A FWS would enable coastal flood risks to be forecast up to 36 hours in advance, enabling greater staff mobilisation and preparation for an event. This will also allow a better planned, quicker and more targeted response in terms of post-event inspections and repairs, significantly reducing the duration of any road closures.

This section summarises the general methodology that could be adopted to develop a coastal FWS. In doing so, this section considers:

- How FWS will work when in operation;
- How the system will be implemented and monitored;
- The methods required to undertake the necessary model simulations used to develop the FWS;
- How incident management procedures can be developed.

5.8.1 Coastal forecasting

In the last decade, the Civil Contingencies Act (2004) and the Flood Risk Management (Scotland) Act 2009 (the FRM Act) have created a new framework for the management of flood risk in Scotland, which is supported by new responsibilities for SEPA. In 2012 SEPA published its Flood Warning Strategy, which aims to reduce the impact of coastal flooding through the provision of reliable and timely flood warnings. SEPA have developed tools to deal with coastal flooding in collaboration with the United Kingdom Coastal Monitoring and Forecasting service (UKCMF) to provide wind, wave and sea level forecast information. SEPA uses this information to provide Flood Alert information around the Scottish coast as part of a national Coastal CFAT (refer to Section 3.4.1). While the CFAT is currently in operation it is not detailed enough to predict overtopping at communities such as Stonehaven. A more refined model would be required to bring offshore forecasts to the toes of the defences to allow accurate overtopping modelling to be undertaken. Such a model would fit into the SEPA Flood Warning Strategy, and may be developed as part of SEPAs long-term investment planning. The following highlights the tasks required to develop such a model.

The development of a FWS normally involves the use and coupling of a suite of numerical models. Some of these modelling components are available nationally, whilst other components need to be developed on a regional level. The key components of modelling that are already available include the Wave Watch III (WW3) model developed by the Met Office, and the CSX3 surge model developed by the National Oceanographic Centre (NOC). Each of these models is operated by the Met Office. The WW3 model provides five day forecasts of deep water wave (wave height, period and direction) and wind properties (speed and direction) on a 12km spatial gird. These forecasts are issued four times a day and are already incorporated into the SEPA Flood Early Warning System (FEWS). The CSX3 model provides five day forecasts of sea-levels and surge magnitudes, also four times a day, and also on a 12km grid.

To forecast wave overtopping or flooding, it is necessary to transform the deep water wave forecasts from WW3 to the nearshore region, ideally to the base of coastal defences and beaches. This is usually done using a regional spectral wave transformation model. This wave transformation model then provides the input conditions to a separate set of erosion calculations and wave overtopping models, which forecast whether scour or overtopping is expected to occur.

Whilst it is technically feasible to run wave transformation models live within a FWS, this is complicated by the computation times of the models, the risk of model instability or failure, and the costly software development involved. To avoid these issues, the recommended approach is to pre-compute a high density of ensemble simulations which will be used to create look-up tables that relate the offshore wave, wind and sea-level forecasts to the resultant erosion, wave overtopping and inundation risk.

With all the possible permutations and combinations pre-computed, there are three possible methods that could be applied to run and manage the FWS. These are:

1. The forecasting systems could be incorporated into the FEWS. Under this approach, SEPA would issue operational messages to Aberdeenshire Council if the forecasted wave overtopping or flooding thresholds are exceeded.
2. A bespoke FWS could be developed and operated by JBA who would issue operational messages to Aberdeenshire Council in a similar fashion to that described above. Such a system is currently in place for Network Rail, and has the advantages of offering a more targeted approach, allowing specific warnings and procedures to be built into the service.
3. Using a direct feed of offshore forecasts supplied by the Met Office, Aberdeenshire Council could utilise an interactive GeoPDF based system to evaluate forecasted coastal flood risk and issue operation instructions. This approach could be used in isolation or in combination with the above.

Once a hosting system is selected the logistics of implementing a forecasting service can be progressed between Aberdeenshire Council and the relevant parties (e.g. SEPA). Several elements will then require development, described in further detail below.

5.8.2 Requirements for a forecasting service

Once in operation the FWS would operate based on a number of operational steps. These have been summarised below, along with further information that Aberdeenshire Council would need before a FWS could be developed.

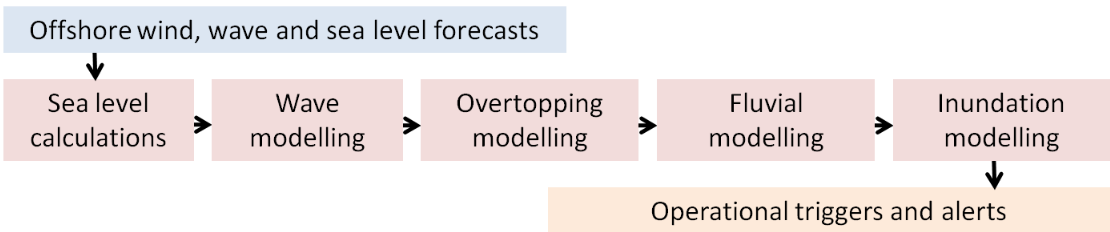

Figure 5-5: FWS operational procedure

Step 1: Offshore wave, wind and water level forecasts for the UK are currently supplied by the NOC and can be used for coastal forecasting. The forecasting system will obtain five day offshore wave and sea-level forecasts through direct feeds from the Met Office (or via FEWS). These forecasts will be received for the closest offshore forecast point.

Step 2: The forecasts of still water sea-level and wave conditions will be interrogated to identify corresponding high water time in the upcoming five day period. This results in approximately ten peak high water levels.

Step 3: It will be necessary to transform deep water wave forecasts from WW3 to the nearshore region, at the base of flood defences and beaches. It is proposed that this is done through the development and use of a spectral wave transformation model, such as SWAN (Simulating WAves Nearshore). The wave modelling will require nearshore recorded wave data to be used to calibrate the model.

Step 4: Overtopping modelling will be used to calculate the rate of water that will overtop the defences due to the nearshore wave conditions. This can be done by constructing local wave overtopping models that utilise tools such as the Neural Network, as described in Section 2.3. The wave overtopping calculation will require defence geometry information such as crest height, slope, beach levels etc.

Step 6: Flood inundation modelling will be undertaken to develop flood outlines that relate to the forecasted tide, fluvial and wave conditions. This can be undertaken based on horizontal projection modelling for still-water inundation, or through hydrodynamic modelling (such as JFlow+ or TUFLOW) for overtopping inundation. The inundation modelling will require topographic information such as LIDAR to represent the land behind the defence.

Step 7: The forecast still water sea-levels and wave overtopping are compared to thresholds established that indicate the onset of flooding. If the forecasted sea-levels exceed the threshold, a coastal flood alert or warning will be issued.

5.9 Resistance and resilience measures

Resistance and resilience measures provide individuals with practical and cost effective steps to help lower their flood risk through the use of affordable and effective bespoke property level protection (PLP) products. These offer an innovative new response which 'plugs the gap' that previously existed between engineered flood protection schemes and either sandbags or the 'do nothing' option. Raising awareness and encouraging the wider use of PLP, together with effective flood warning can help build resilience to flooding and establish better informed communities that can take action for themselves.

A key aspect of PLP measures are that they do not reduce the likelihood of flooding in the same way that capital schemes do, but they will assist in reducing and managing the consequences of flooding. These measures are typically broken down into 'property resistance' - measures that limit water entry into a property to reduce flood risk to an acceptable level or to buy time to move possessions to safety; and 'property resilience' - measures that limit the damage caused by floodwater once inside a property. PLP is most commonly associated with the deployment of a range of flood resistance measures such as door barriers, airbrick covers, non-return valves, sealants and sump-pumps. In reality, PLP is most likely to require the deployment of a range of flood resistance and flood resilience measures with associated flood warning for systems deployments. PLP measures can further be sub-divided into manual or automatic (passive) systems. Automatic approaches tend to be more expensive to install but offer a key benefit in that they are more reliable as homeowners do not need to be at home to fit products.

Property level protection for Stonehaven has already been implemented for a number of properties within the town, primarily as a response to fluvial flooding from the River Carron. The extension of this scheme to cover coastal areas would need additional consideration. Key aspects to consider are as follows:

- PLP for coastal areas would need a good level of flood warning for coastal wave and surge events.
- PLP may not be suitable to protect against high impact wave overtopping as observed at some locations in Stonehaven.
- PLP is suitable for flood depths up to 0.6m.
- PLP relies upon a thorough independent property survey to establish the needs of each property. The importance of considering and addressing flood risk from all sources, in particular the risk of water rising up through floors and floodwater seeping through walls cannot be underestimated. This may require the appropriate use of more expensive sump pumps, non-return valves or resilience measures.
- PLP works best when undertaken as part of a local authority led scheme to encourage community buy-in and a more comprehensive take-up (critical where flats, terraced and semi-detached properties are present).
- An important factor to emphasise through appropriate public engagement is the management of public expectation: PLP will not provide any guarantees that a property will no longer suffer from flood inundation, rather it aims to help manage the consequences by reducing the chances of floodwater damage.
- Post implementation emergency planning, dry-runs and scheme administration are all required. However, a coordinated flood group with a regularly tested emergency plan will ensure cohesion in community and effective homeowner response.
- PLP could compromise the economics of any future flood mitigation works.

The performance and ultimate success of any PLP scheme depends on many factors that, if overlooked, could lead to problems, but if adopted will all help to contribute to a successful scheme. JBA Consulting and the Scottish Government is in the process of providing a best practice guidance document for local authorities, which is recommended should PLP be a suitable option for consideration.

6 Study limitations

Wave overtopping and sediment transport modelling is a complicated process, heavily reliant on the quality of input data such as sediment and boundary conditions, the sea bed and bathymetry, defence parameters, sediment parameters and wave conditions. As explained throughout this report, this project does not represent a comprehensive wave overtopping study, but rather a high level overview to help inform the future management of the Stonehaven frontage. At this level, there is a degree of uncertainty involved in all calculations, which will limit the accuracy of the overtopping results, the subsequent conceptual designs and the cost-benefit assessment. It is suggested that a future more detailed study is considered which would include a detailed numerical model to more accurately define the process of wave transformation from offshore to onshore and hence provide a more accurate estimate of nearshore waves. This would then allow a more detailed estimate of wave overtopping flood damage to be made and hence a more robust economic analysis could be completed.

7 Summary and recommendations

This study was undertaken by JBA Consulting for Aberdeenshire Council as a high level overview to help inform the future management of coastal flooding in Stonehaven. It has investigated the magnitude of recent storm events in terms of offshore waves, sea levels and expected overtopping (such as in December 2012). Using this information the standard of protection of the coastal frontage was considered, and potential improvements to the beach and rear seawall assessed that could reduce the level of coastal flood risk.

An assessment of historic storms between 2002 to 2012 considers the December 2012 coastal event has a return period of under 1-year for the observed water levels, over 200-years for offshore waves, and over 200-years for the joint-probability wave and water level conditions experienced. Using a simplified depth-limited approach the nearshore wave height and overtopping was estimated for historic events. The December 2012 event was considered to have the highest rate of overtopping since 2002, however a more detailed assessment is required in order to characterise the rate of overtopping between individual defence profiles.

Potential changes to the existing beach width and elevation was investigated to decrease the existing rate of overtopping. In order to meet a nominal present day 5 L/s/m overtopping target, the following changes have been calculated.

- Profile SH05: A widened beach between 10m to 15m.
- Profile SH12: Currently meets the nominal overtopping target under the simplified depth-limited assessment method (which is not considered reflective of existing conditions).
- Profile SH17: A 0.5m increase to the existing defence crest level and a 15m wide beach.

An assessment of the increased overtopping rate under the influence of climate change indicates all defences will require an increased crest level in the future, ranging between 0.5 to 2m.

A review of the beach recycling activities and sediment changes between 2008 and 2013 suggest an accumulation of sediment, with some minor loss of sediment to the southern end of the beach.

A high-level cost-benefit analysis was undertaken for two beach scenarios reflecting a mid- and high-level beach recharge scheme. Key limitations of the assessment were the lack of detailed overtopping and inundation data, which effects both the estimates of damages and costs for the scheme, the unknown source of sediment and the requirements for annual maintenance recharge. The analysis suggests the economic viability of the scheme is only viable if a local (onshore) sediment source can be identified (i.e. the cost:benefit goes above 1). This analysis includes an allowance for annual maintenance recharge estimated at 5% of the capital costs. If this allowance is removed both an onshore or offshore sediment source would be considered viable, with the cost-benefit ratio increasing to between approximately 1.5 and 2.5.

Recommendations

This assessment was undertaken as a high-level overview to consider potential improvements to the Stonehaven beach that could reduce the level of coastal flood risk. As such, there is considerable uncertainty associated with calculations that could be addressed through further wave and sediment analysis. Never-the-less, the following recommendations have been made based on the observations made during the assessment.

1. The use of property level protection (PLP) should be considered to provide a short term resilience measure against coastal flooding.
2. The continued trend of erosion to the south of the beach support previous recommendations for the addition of two short groynes to the south of the River Carron. These would help stabilise the sediment deposited in this area, and may allow greater time between mechanical beach recycling works by Aberdeenshire Council. A further study is recommended to assess the effectiveness and optimal orientation of the potential groynes.
3. It is suggested that a more detailed study is undertaken in the future, which would include a detailed numerical model to more accurately define the process of wave transformation from offshore to onshore and hence provide a more accurate estimate of nearshore waves. This would then allow a more detailed estimate of wave overtopping flood damage to be made and hence a more robust economic analysis could be completed.

4. The detailed study should also consider a more holistic defence strategy incorporating different defence requirements in different parts of the bay (e.g. wall raising vs. recharge), and consider the desired standards of defence (e.g. allowing 10l/s/m overtopping during a 200-year event). By considering these elements it is considered that a more cost effective strategy can be developed for the bay than beach recharge alone.
5. The detailed study would update the cost-benefit assessment for the beach recharge scheme, and would incorporate the following:
    a. The use of numerical wave modelling, overtopping and inundation modelling to ascertain the extent and depth of wave overtopping under an existing-case scenario.
    b. Identification of the source of sediment able to be used for beach recharge.
    c. A long-term longshore sediment transport assessment to predict the likely sediment losses and required maintenance recharge costs after implementation.

Appendices

A Offshore extreme wave and water level data

Extreme wave conditions have been obtained from the Environment Agency *Coastal flood boundary conditions for UK mainland and islands* project[20] which includes design swell wave conditions around Scotland, England and Wales. The extreme offshore wave data point used for this project is located approximately 18km east of Stonehaven (783177N 405475E). The most significant wave heights expected at Stonehaven are associated with waves originating from the northeast direction (a wave direction of 45°) with wave heights from this direction considered in this report. Extreme wave conditions have been assigned wave periods based on the upper range as presented in the *Coastal flood boundary* study, as shown in presented in Table 7-2. As the extreme wave heights are consistently greater than 3.0m they have been categorised within the 10s – 12s wave period range. For wave heights above 5.0m, this represents the more conservative estimate as such waves are equally as likely to fall within the 8s – 10s wave period range.

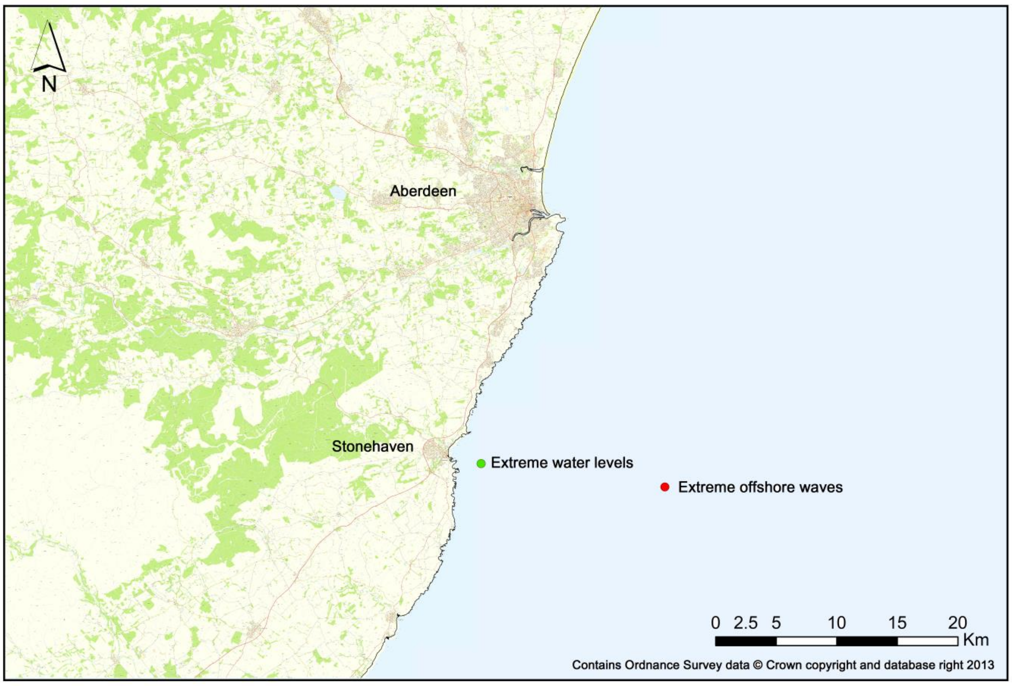

Figure 7-1: Location of the extraction points used extreme still water levels and wave heights offshore of Stonehaven

Table 7-1: Probability of occurrence of offshore wave period for different wave heights

Wave Height (m)	Period (Tz seconds)					
	<8s	8 - 10s	10 - 12s	12 - 14s	14 - 16s	>16s
<1	0.65	0.20	0.10	0.04	0.01	-
1-2	0.55	0.35	0.08	0.01	-	-
2-3	0.24	0.51	0.22	0.03	-	-
3-4	0.14	0.33	0.42	0.10	-	-
4-5	-	0.36	0.50	0.14	-	-
5-6	-	0.50	0.50	-	-	-
>6	*Not available*					

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

JBA consulting

Table 7-2: Extreme wave estimates for waves originating from the northeast.

Return Period (year)	Hs (m)	Tm (sec)	Return Period (year)	Hs (m)	Tm (sec)
0.2	3.16	12	**10**	5.47	12
0.5	3.71	12	**20**	5.88	12
1	4.18	12	**50**	6.42	12
2	4.56	12	**100**	6.84	12
5	5.07	12	**200**	7.27	12

Stonehaven is a secondary non-harmonic port and tidal predictions are based on the primary port of Aberdeen, located approximately 21km north. The astronomic tide levels for Stonehaven were extracted from the United Kingdom Admiralty Office Total Tide[21] software and are presented in Table 3-1. The highest astronomical tide level at Stonehaven is 2.65mAOD.

Table 7-3: Tide levels at Stonehaven

Location	Level (mAOD)
Highest Astronomical Tide (HAT)	2.65
Mean High Water Springs (MHWS)	2.05
Mean High Water Neaps (MHWN)	1.15
Mean Sea Level (MSL)	0.17
Mean Low Water Neaps (MLWN)	-0.75
Mean Low Water Springs (MLWS)	-1.85
Lowest Astronomical Tide (LAT)	-2.45

Extreme sea-levels conditions were obtained from the Environment Agency (EA) Coastal Flood Boundary Dataset (CFBD)[22] which consists of expected sea-level estimates during extreme storm events (but not including wave action). For the study, tide level data from Class A gauge sites spanning the UK coastline were analysed. Through statistical analysis, probabilities of predicted high tide and skew surge levels were generated. By combining these two elements, a set of design extreme SWLs for Scotland, England and Wales corresponding to 2008 conditions was produced.

The extreme SWL point used for this project is located approximately 3km east of Stonehaven shoreline (390256 East, 785116 North). Predicted extreme SWLs at Stonehaven for a range of return periods up to 200 years are presented in Table 3-4. For each return period, predicted sea levels were corrected for sea level rise to 2115.

The latest UK Climate Projections (UKCP09)[23] were used to estimate the future effects of climate change on mean sea levels. A medium emissions scenario with a 95th percentile confidence interval was assumed for the prediction of the likely magnitude of sea level rise at Stonehaven. A sea level rise of **0.67m** at Stonehaven is expected by 2115.

21 The United Kingdom Hydrographic Office Admiralty Total Tide software

22 Environment Agency (2011), 'Coastal flood boundary conditions for UK mainland and island's, Project: SC060064/TR2: Design sea-levels. Environment Agency, Feb 2011.

23 DEFRA, Crown Copyright, (2009), UK Climate Projections

Table 7-4: Extreme water levels at Stonehaven for different return levels

Return Period (year)	Water levels (mAOD) (2008)	2115 (mAOD) (2008 level +0.67m)	Return Period (year)	Water levels (mAOD) (2008)	2115 (mAOD) (2008 level +0.67m)
0.2	2.60	3.27	**10**	2.97	3.64
0.5	2.69	3.36	**20**	3.03	3.70
1	2.73	3.40	**50**	3.12	3.79
2	2.80	3.47	**100**	3.19	3.86
5	2.89	3.56	**200**	3.25	3.92

B Joint probability Analysis

The combinations of sea level and wave height values required to achieve the worst case scenario for any given return period can be determined through joint probability analysis. This is achieved through long-term time-series analysis using methods described in the Defra best practice guidance[24].

A joint probability analysis of two sets of variables requires an understanding of their interdependence. This is quantified in the Defra guidance notes for a selection of locations around the UK and shows considerable variation nationally. Along the east Scotland coastline, sea level and wave height are considered to be modestly correlated. This is quantified through a correlation coefficient. For the Stonehaven coastline, the coefficient ranges between values of 0.12 and 0.37, with a specific correlation coefficient available for Aberdeen with a value of 0.21. Three correlation coefficients were considered within the suggested range to demonstrate the influence on estimated joint probability return periods: a lower value of 0.12; a mid value of 0.21; and an upper value of 0.37.

The wave heights required to achieve a joint probability return period for any given extreme SWL are presented in Table 7-5 to Table 7-7 for the three correlation coefficients considered. Comparing results for the low and high correlation coefficients indicates that for larger return periods, the difference in wave height required can be up to 1.4 m. This demonstrates considerable sensitivity to the choice of correlation coefficient within the recommended range of values.

Results from the joint probability analysis are presented graphically in Figure 7-3. The low, mid and high correlation coefficients considered are represented by dotted, dashed and solid lines respectively. Each return period is represented by a different colour as highlighted by the figure legend. The lines effectively shift towards the bottom left corner with decreasing correlation coefficient and towards the upper right corner with increasing joint probability return period. This has been used to characterise historical events where flooding was evident. As the joint probability analysis will be used to assess recent events, it was conducted using the present-day extreme SWLs (i.e., excluding the impacts of climate change).

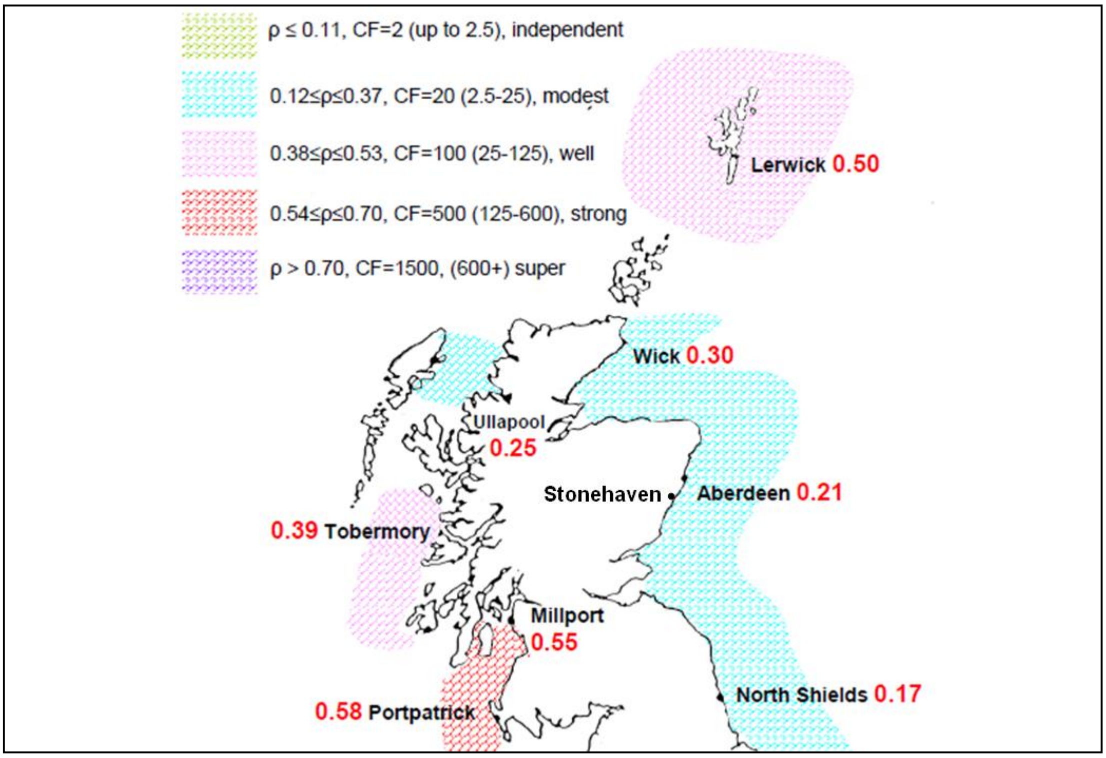

Figure 7-2: Dependence information for wave height and SWL (Source: Defra 2003)

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

Table 7-5: Combinations of extreme still water levels and wave heights required to achieve various joint probability return periods for a low correlation coefficient (ρ = 0.12)

Extreme SWL		Joint probability return period (years)									
Level (mAOD)	Return period (years)	0.2	0.5	1	2	5	10	20	50	100	200
		Extreme wave heights (m)									
2.49	0.1	1.13	1.70	2.36	2.76	3.28	3.68	4.07	4.60	5.00	5.41
2.57	0.2	0.83	1.40	1.98	2.38	2.90	3.30	3.69	4.22	4.61	5.02
2.66	0.5	-	0.92	1.48	1.88	2.40	2.80	3.19	3.72	4.11	4.51
2.73	1	-	-	1.10	1.50	2.02	2.42	2.81	3.34	3.73	4.13
2.80	2	-	-	-	1.12	1.64	2.04	2.43	2.96	3.35	3.75
2.89	5	-	-	-	-	1.14	1.53	1.93	2.45	2.85	3.25
2.97	10	-	-	-	-	-	1.15	1.55	2.07	2.47	2.87
3.03	20	-	-	-	-	-	-	1.17	1.69	2.09	2.49
3.12	50	-	-	-	-	-	-	-	1.19	1.59	1.98
3.19	100	-	-	-	-	-	-	-	-	1.21	1.60
3.25	200	-	-	-	-	-	-	-	-	-	1.22

Table 7-6: Combinations of extreme still water levels and wave heights required to achieve various joint probability return periods for a mid-range correlation coefficient (ρ = 0.21)

Extreme SWL		Joint probability return period (years)									
Level (mAOD)	Return period (years)	0.1	0.5	1	2	5	10	20	50	100	200
		Extreme wave heights (m)									
2.49	0.1	1.32	1.93	2.60	3.03	3.59	4.01	4.44	5.00	5.45	5.90
2.57	0.2	1.02	1.63	2.22	2.65	3.21	3.63	4.06	4.62	5.05	5.49
2.66	0.5	-	1.15	1.72	2.14	2.70	3.13	3.55	4.11	4.54	4.97
2.73	1	-	-	1.34	1.76	2.32	2.75	3.17	3.73	4.16	4.58
2.80	2	-	-	-	1.38	1.94	2.37	2.79	3.35	3.78	4.20
2.89	5	-	-	-	-	1.44	1.87	2.29	2.85	3.28	3.70
2.97	10	-	-	-	-	-	1.49	1.91	2.47	2.90	3.32
3.03	20	-	-	-	-	-	-	1.53	2.09	2.52	2.94
3.12	50	-	-	-	-	-	-	-	1.59	2.01	2.44
3.19	100	-	-	-	-	-	-	-	-	1.63	2.06
3.25	200	-	-	-	-	-	-	-	-	-	1.68

Table 7-7: Combinations of extreme still water levels and wave heights required to achieve various joint probability return periods for a high correlation coefficient (ρ = 0.37)

Extreme SWL		Joint probability return period (years)									
Magnitude (mAOD)	Return period (years)	0.2	0.5	1	2	5	10	20	50	100	200
		Extreme wave heights (m)									
2.49	0.1	1.66	2.35	3.04	3.51	4.14	4.62	5.10	5.77	6.28	6.80
2.57	0.2	1.36	2.05	2.66	3.13	3.76	4.24	4.71	5.36	5.87	6.39
2.66	0.5	-	1.57	2.15	2.63	3.26	3.73	4.21	4.84	5.34	5.85
2.73	1	-	-	1.77	2.25	2.88	3.35	3.83	4.46	4.94	5.44
2.80	2	-	-	-	1.87	2.50	2.97	3.45	4.08	4.55	5.04
2.89	5	-	-	-	-	2.00	2.47	2.95	3.58	4.05	4.53
2.97	10	-	-	-	-	-	2.09	2.57	3.20	3.67	4.15
3.03	20	-	-	-	-	-	-	2.19	2.82	3.29	3.77
3.12	50	-	-	-	-	-	-	-	2.31	2.79	3.27
3.19	100	-	-	-	-	-	-	-	-	2.41	2.89
3.25	200	-	-	-	-	-	-	-	-	-	2.51

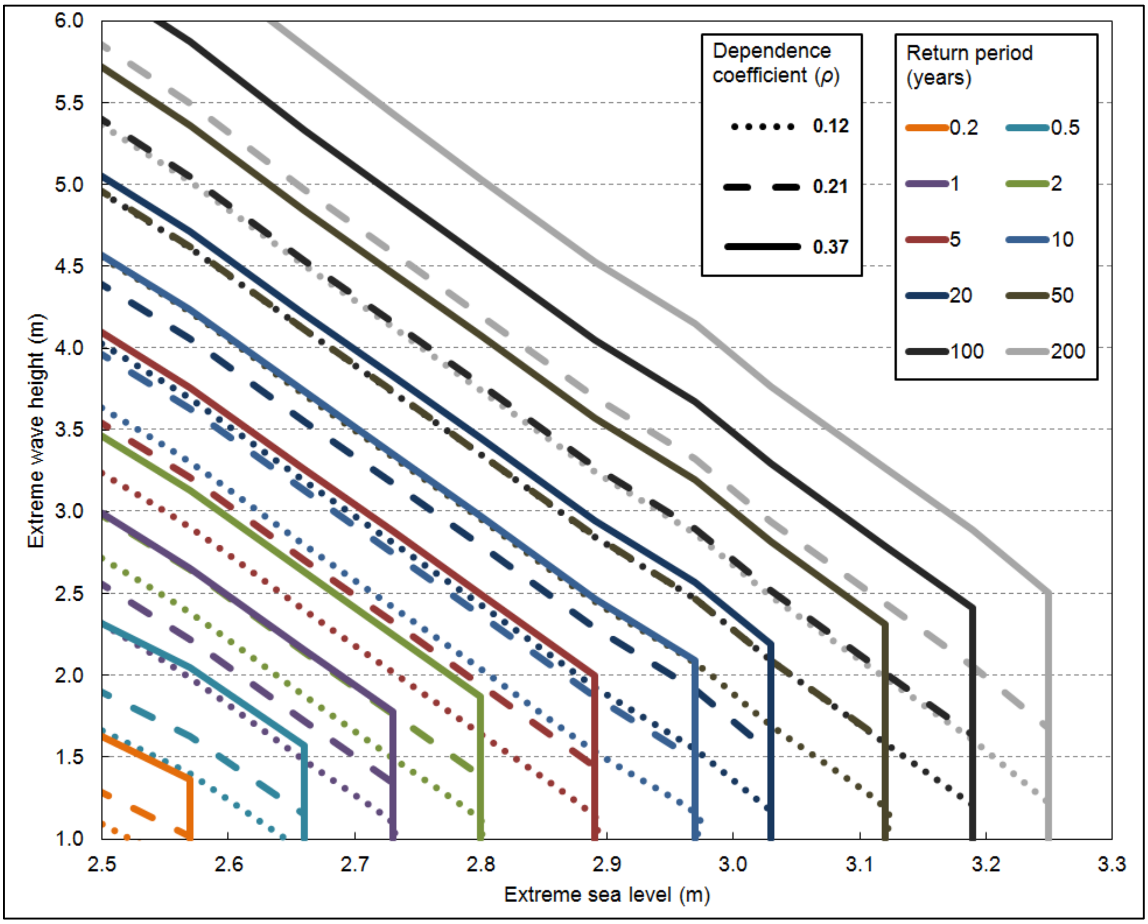

Figure 7-3: Lower limit bands for joint probability return periods accounting for extreme still water level and wave height.

Offices at

Coleshill

Doncaster

Edinburgh

Haywards Heath

Limerick

Newcastle upon Tyne

Newport

Saltaire

Skipton

Tadcaster

Thirsk

Wallingford

Warrington

Registered Office
South Barn
Broughton Hall
SKIPTON
North Yorkshire
BD23 3AE

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

Jeremy Benn Associates Ltd
Registered in England
3246693

ISO9001 • ISO14001
OHSAS 18001

UKAS MANAGEMENT SYSTEMS 001

ISO/IEC27001

UKAS MANAGEMENT SYSTEMS 001

CHAS Accredited Contractor
www.chas.gov.uk

Visit our website
www.jbaconsulting.com