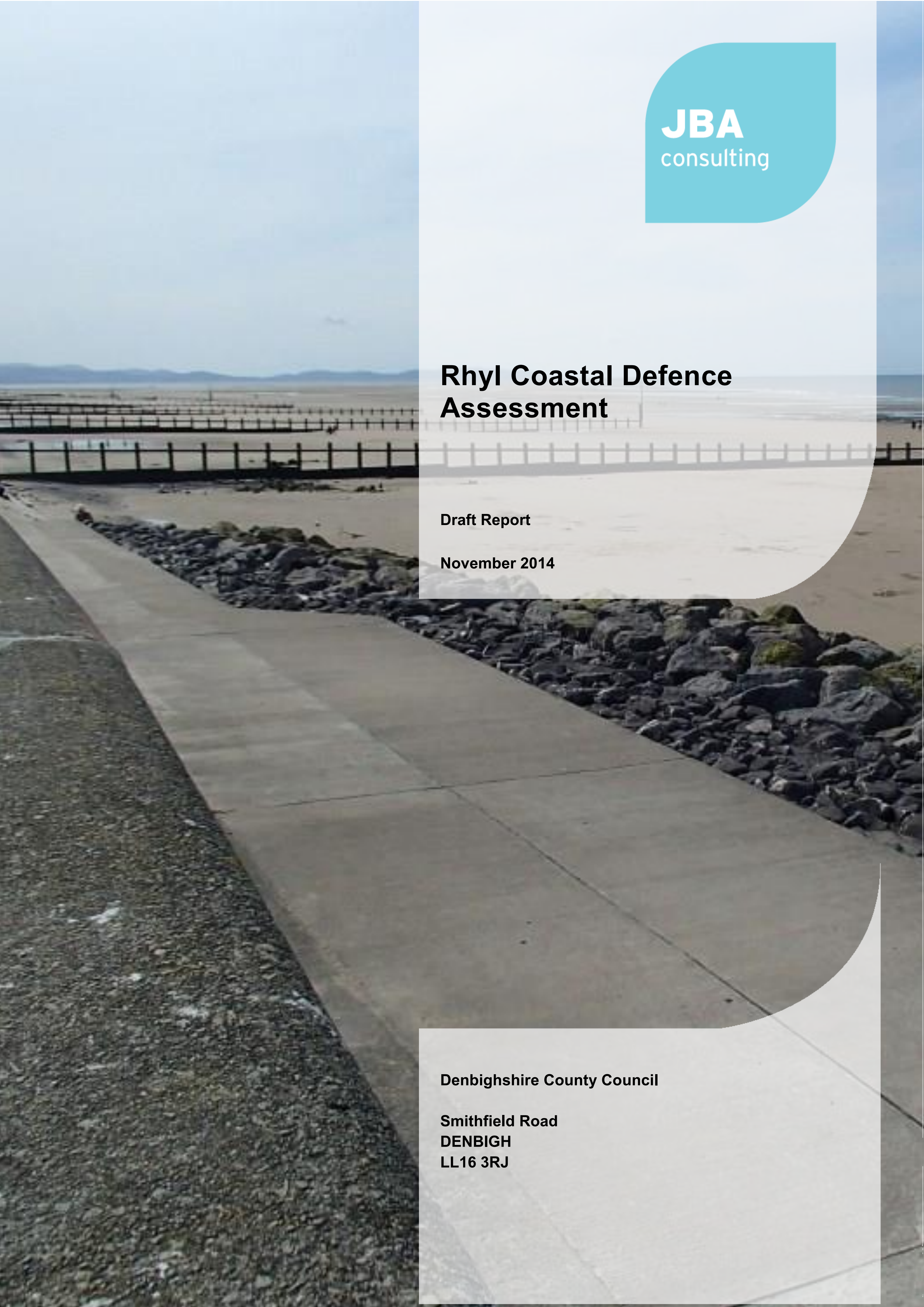

JBA consulting

Rhyl Coastal Defence Assessment

Draft Report

November 2014

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This report describes work commissioned by Denbighshire County Council, by an email dated 10 September 2014. Denbighshire County Council's representative for the contract was Wayne Hope. Alain Le Vieux and Daniel Rodger of JBA Consulting carried out this work.

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Purpose

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Executive summary

This investigation was undertaken by JBA Consulting on behalf of Denbighshire County Council to consider the Standard of Defence (SoP) of the Rhyl coastal defences, located on the North Wales coastline. The study had three aims; to establish the current SoP of the coastal defences, to consider the likely inundation of a design 200-year coastal event, and to consider the magnitude of the December 2013 storm.

There is a wide range of information relating to coastal processes and extreme conditions at the Rhyl coastline which includes previous assessments, strategies and reports. This information shows that there is the potential for extreme water level and wave conditions at the coastline, which may overtop the existing defence. Previous assessments of the frontage indicate a trend of long-term beach lowering, which may allow larger waves to reach the shoreline, leading to increased wave overtopping and undermining of the defences.

In order to calculate the current SoP a number of wave, overtopping and inundation models have been used. Wave overtopping rates were estimated for return periods ranging from 1-year to 200-years, in addition to the December 2013 event. The SoP was estimated for several defence cross-sections, where a rate of 10 l/s/m has been considered the limit of tolerable overtopping. Under this limit, the SoP varies between under 1 in 1-year to a 50-year return period across the defence. The estimated worst-case overtopping rate during the December 2013 event considered to be 1 in 40-years. The worst-case overtopping was located at Splash Point, with the least overtopping considered to the eastern end of the defence.

Inundation modelling was undertaken to compare the December 2013 event to the worst-case 200-year coastal event under present day and climate change conditions. The estimated 2013 event resulted in a relatively smaller flood extent with a coverage of 0.44km^2 with flooding mainly confined to the seaward side of Rhyl Coast Road. The modelled flood outline approximated the observations made by Denbighshire County Council, allowing an informal validation of the model. In contrast, the estimated 200-year present day event covered a far greater area calculated to be approximately over 2km^2, with the inundation spreading south of the railway. The 200-year plus climate change event increased the inundation further, with an extent of approximately 2.80km^2.

A key recommendation of this study is that further assessment should be undertaken to address limitations encountered in the numerical modelling. For coastal engineering and flood risk assessments it is essential that there is a source of high quality coastal extreme data and an accurate methodology for undertaking joint probability assessments. Previous assessments of the December 2013 event show a growing concern as to whether available offshore wave estimates accurately represent extreme conditions, and if the Defra joint probability methodology correctly predicts the coincidence of extreme waves and sea levels. While this study used extreme wind speeds to drive wave models, there remains the uncertainty due to joint probability of wind and sea levels used to develop design scenarios. It is important that a revised joint probability assessment is undertaken to increase the reliability of nearshore wave and overtopping estimates, adopting a methodology such as that proposed by Heffernan and Tawn. It is recommended that this is conducted prior to any future upgrade to the Rhyl defences, which will ensure it is designed to an appropriate SoP. Any upgrades should be designed to include the impact of climate change, which can produce far greater rates of overtopping and inundation consequences.

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Abbreviations

2D .... Two dimensional

CC.................................. Climate Change to year 2115

CEFAS............................ Centre for Environment, Fisheries and Aquaculture Science

CFB................................ Coastal Flood Boundary

CSV................................ Comma Separate Value

DTM .... Digital Terrain Model

EA .... Environment Agency

ESL .... Extreme Sea-Level

EurOtop........................... European Overtopping Manual

GIS................................. Geographical information systems

HAT................................ Highest Astronomical Tide

JBA .... Jeremy Benn Associates

LAT .... Lowest Astronomical Tide

LIDAR .... Light Detection and Ranging

POT................................ Peak Over Threshold

mAOD .... metres Above Ordnance Datum (UK)

MHWN .... Mean High Water Neaps

MHWS............................ Mean High Water Springs

MLWN............................ Mean Low Water Neaps

MLWS .... Mean High Water Springs

MSL................................ Mean Sea-Level

NRW .... Natural Resources Wales

RAM .... Random Access Memory

SoP .... Standard of Protection

ST .... Source Time

SWAN .... Simulating WAves Nearshore

TUFLOW .... 2D hydrodynamic flood inundation model

UK .... United Kingdom

UKCP09 .... UK Climate Change Impact Programme 09

1 Introduction

1.1 Study site

This investigation was undertaken by JBA Consulting on behalf of Denbighshire County Council to consider the standard of protection (SoP) of the Rhyl coastal defences, located on the North Wales coastline, as shown in Figure 1-1. The study has three aims:

1. To establish the current Standard of Protection (SoP).
2. To estimate the inundation due to a design 200-year present day coastal event, a 200-year plus climate change (to 2115) event and the December 2013 event.
3. To consider the magnitude of the December 2013 event at Rhyl.

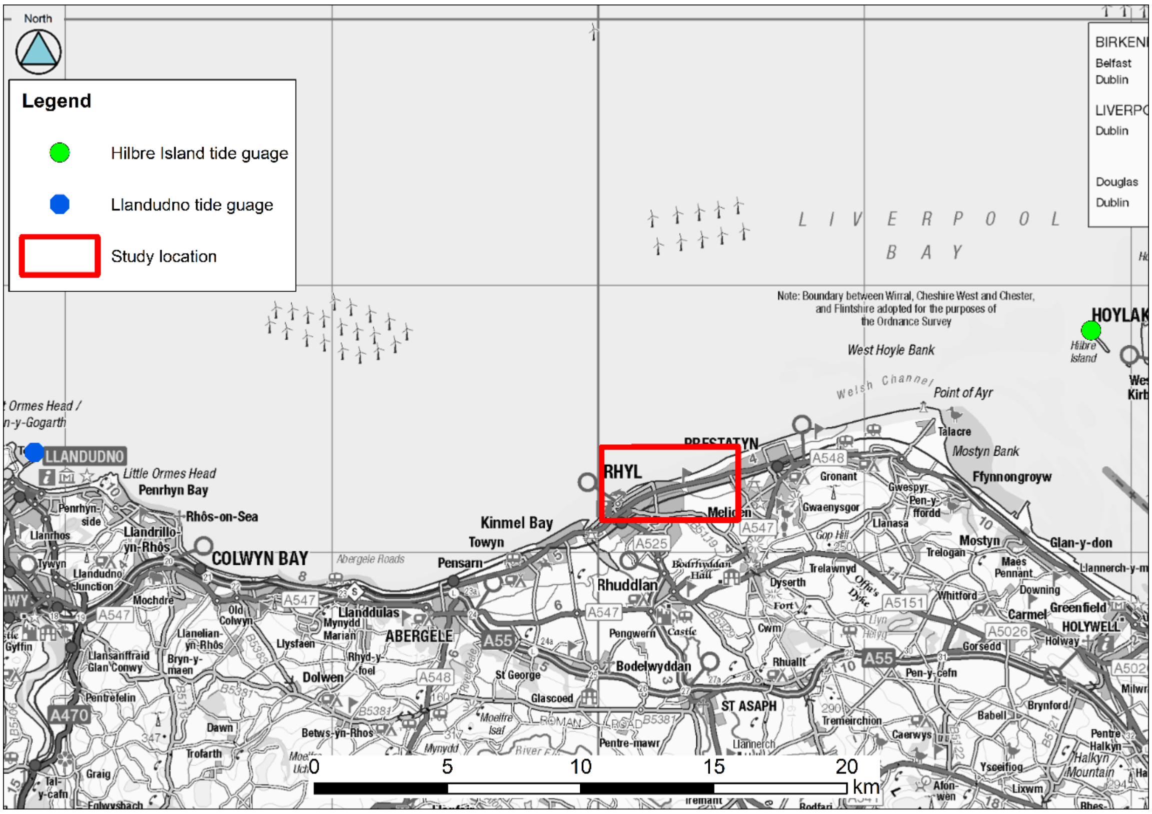

Figure 1-1: Location of the proposed Rhyl (Contains Ordnance Survey data © Crown copyright and database right 2014).

A number of coastal numerical models and investigations were undertaken to support the project aims. These have summarised in the following chapters:

- **Chapter 2 (Coastal processes)** describes the coastal processes at work in Rhyl, such as longshore drift, erosion and sediment supply.
- **Chapter 3 (Current standard of protection)** outlines the approach to calculate the overtopping resulting from the nearshore wave conditions and evaluate the current standard of coastal protection at Rhyl.
- **Chapter 4 (Flood inundation modelling)** outlines the approach and the results of the TUFLOW inundation modelling.
- **Chapter 5 (Consideration of the December 2013 event)** outlines the approach to calculate the inundation due to wave overtopping at the study site.
- **Chapter 6 (Summary and conclusions)** discusses the results, conclusions and presents recommendations.

2 Coastal processes

2.1 Background to coastal flooding

Before conducting wave overtopping investigations, it is important to first consider the drivers of coastal risk for the frontage. Coastal flooding is a complicated process, affected by a number of dependant and independent variables. Figure 2-1 illustrates the main components of sea-level variation that contribute to coastal flooding during a storm event. The base sea-level, often referred to as either the still water sea-level or total sea-level, is comprised of the underlying astronomical tide and the passage of a large scale storm surge. These two components determine the average sea-level for a specific location at a particular time. Whilst this variable is very important in terms of coastal flooding, still water-induced flooding is normally limited to sheltered locations such as tidal rivers and harbours. Not surprisingly, the sea is not still during a storm event and for more exposed locations such as Rhyl most flooding occurs through wave action, rather than still water flooding.

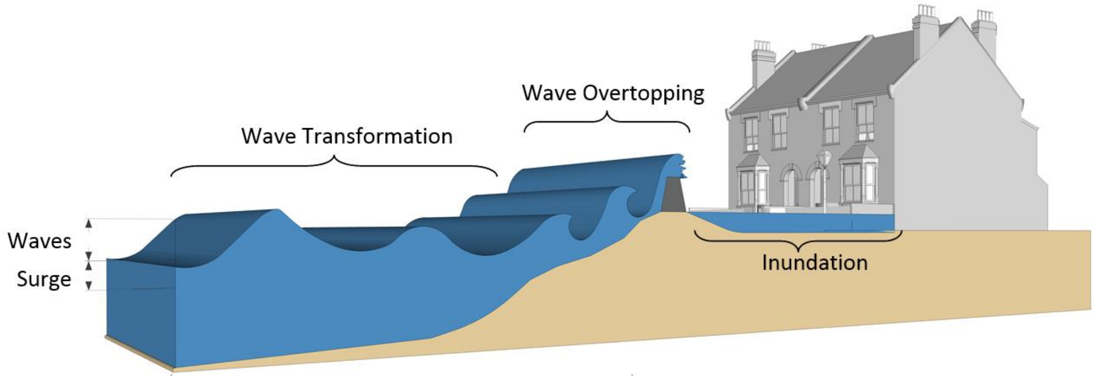

Figure 2-1: Components of sea-level variation that lead to typical coastal flooding.

Wave action is a complex process controlled by a number of factors. The manner in which these factors combine determines the magnitude of any wave induced flood impacts. Waves generate in deep water and then propagate towards land. As they do so, they enter shallower bathymetry where wave transformation processes occur, including shoaling, diffraction, refraction, depth limitation and breaking. These waves are also subject to additional influence from wind. The consequence of these processes is that the properties of the waves, when they reach the base of flood defences, are quite different to the waves in deep water. It is these nearshore waves that are of most importance because they interact with beaches and defences and lead to wave overtopping.

Wave overtopping itself is also a complicated process controlled by the state of the sea (depth, wave properties), the geometry of the beach and local flood defences. The impact of all of the above flood risk drivers during a particular storm is also heavily dependent upon the location and orientation of the defences with respect to the sea. This means that while one location may be flooded during a storm event another, just a short distance away, may be impacted to a lesser extent due to its orientation with respect to the dominant wind/wave direction.

At present there is no one numerical model or calculation approach able to replicate all of these processes. Instead, they are represented through a suite of numerical models as shown in Figure 2-2.

Figure 2-2: Modelling components of the wave overtopping assessment.

2.2 Previous coastal processes assessments

The coastal processes along the Denbighshire coastline were the focus of previous coastal assessments, which are detailed in the Rhyl to Prestatyn Coastal Defences Strategy Study Report (WMA 2012)[1]. This is summarised below in terms of the defence condition and the likely sediment transport trends that have an influence on wave overtopping. This information can be used to provide further information to the condition of the defences, and the coastal processes that need to be addressed in the future.

2.2.1 Coastal defences

The Rhyl coastal defence is a composite of a number of sections, varying in form, material and age. This ranges from historic vertical concrete sea walls (circa 110 years old), to the latest defences incorporating re-curved seawalls constructed in 2012 for the West Rhyl Coastal Defence Scheme. A number of timber groynes were constructed to control the longshore sediment transport, which have deteriorated and are now in various states of disrepair. Other beach control structures are located towards the east of the defence scheme and include a rock groyne field and vegetated sand dunes.

2.2.2 Coastal management

The regional coastline is managed under the Liverpool Bay Shoreline Management Plan (1999)[2], with the Rhyl defences falling under Subcell 11a Management Unit (MU) 4/1 adjacent to the golf course, and MU 4/2 to the west towards Splash Point. The current, short-term and long-term management approach consists of 'hold the line', which will ensure the defences will be maintained against future sea-level rise and deterioration.

2.2.3 Effect of coastal processes on wave overtopping

The Coastal Defence Strategy references previous wave overtopping assessments undertaken by HR Wallingford[3] (refer to Section 3.4), which considers the main coastal processes effecting the defences to be beach lowering. The lowering of beach levels can impact overtopping rates by allowing larger waves to reach the shoreline and have the added risk of undermining defences. The main process responsible for beach lowering is contributed to by a change in the longshore drift patterns, which is estimated at 330,000m^3/year eastwards towards Splash Point (see Figure 3-6) and 485,000m^3/year eastwards to East Prestatyn. The transition in coastline angle causes the change in magnitudes, and results in a deficiency in the sediment budget at the Rhyl defences. In simple terms there is more sediment leaving the site than is being replaced by natural processes.

In order to retain this sediment a series of timber groynes were constructed that act to reduce littoral drift. The Coastal Defence Strategy considers the groynes to have had a positive impact, with the beach levels rising along the frontage and a sandy beach being formed at the toe of the defence structure. However, based on field inspections there remains a degree of toe erosion that can be observed in front of the defences, which may be attributed to the structural deterioration of the timber groynes.

2.3 Coastal extremes

In order to assess the magnitude of the event a range of metocean data were collected. This includes astronomical tides, extreme sea-level and wave height estimates.

2.3.1 Tide levels

Admiralty Total Tide software was used to extract the underlying astronomical tide for Denbighshire coastline. The astronomic tide levels at Rhyl were based on the two closest secondary harmonic ports, being Llandudno 20km to the west and Hilbre Island 17km the east (Figure 1-1). Using a distance weighted approach the tide levels were calculated for Rhyl. These are shown in Table 2-1. The region experiences a macro-tidal climate, with an astronomic (mean spring) tidal range of 7.47m, and the highest astronomical tide is 5.03mAOD.

[1] MWA (2012) Rhyl - Prestatyn Coastal Defences Strategy Study Report, Denbighshire County Council, 2012 August, Martin Wright Associated.

[2] LBCG (1999) Liverpool Bay Shoreline Management Plan, Sub Cell 11a: Great Ormes Head to Formby Point, Liverpool Bay Coastal Group.

[3] Referenced in MWA (2012) as: HR Wallingford, July, 2008

Table 2-1: Astronomic tide levels at Rhyl calculated through distance weighting.

Location	Level (mAOD)
Highest Astronomical Tide (HAT)	5.03
Mean High Water Springs (MHWS)	3.97
Mean High Water Neaps (MHWN)	2.17
Mean Sea-level (MSL)	0.22
Mean Low Water Neaps (MLWN)	-1.70
Mean Low Water Springs (MLWS)	-3.50
Lowest Astronomical Tide (LAT)	-4.56

2.3.2 Extreme sea-level estimates

Extreme coastal conditions were obtained from the Environment Agency (EA) *Coastal flood boundary conditions for UK mainland and islands* project, which produced the Coastal Flood Boundary Dataset (CFBD). The CFBD contains the estimated extreme sea-levels throughout the UK based on research involving more than 40 Class A water level gauges[4]. The predicted extreme still water levels (SWL) at Rhyl for a range of return periods are presented in Table 2-2.

The table also includes the likely changes to extreme sea-levels based on the latest UK Climate Projections (UKCP09)[5]. A medium emissions scenario with a 95th percentile confidence interval is considered to result in a 0.71m rise in sea-level by 2115, which was added to the present day extremes.

Table 2-2: Extreme water levels at Rhyl for different return periods.

Return Period (year)	Present day (2014) water levels (mAOD)	2115 water levels (mAOD) (2014 level +0.705m)
1	5.06	5.76
5	5.28	5.98
10	5.37	6.07
20	5.46	6.16
50	5.58	6.28
100	5.67	6.37
200	5.77	6.47

2.3.3 Extreme wave height estimates

Extreme wave conditions were obtained from the CFBD based on the EA *Coastal flood boundary conditions for UK mainland and islands* project for design swell waves[6]. Predicted extreme offshore swell waves for a range of return periods are presented in Table 2-3.

Table 2-3: Extreme offshore swell waves at Rhyl for different return periods.

Return Period (years)	1	5	10	20	50	100	200	1,000
Offshore swell wave height (m)	2.19	2.49	2.60	2.70	2.82	2.90	2.97	3.11

[4] Coastal flood boundary conditions for UK mainland and islands, Project: SC060064/TR2: Design sea-levels. Environment Agency, Feb 2011.

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

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

3 Current standard of protection

3.1 Introduction

In order to calculate the current SoP for the Rhyl coastal frontage a number of wave, overtopping and inundation models were used. First a wave transformation model was used to calculate extreme wave conditions at the toe of the Rhyl defences. These conditions were then used to calculate the rate of overtopping occurring along the frontage, with the resulting inundation mapped using a hydrodynamic model. Each of these elements are described in the following sections, and are used to develop an understanding of the current standard of protection of the Rhyl coastal defence.

3.2 Wave transformation modelling

3.2.1 Wave model development

A wave transformation model was developed to calculate the extreme wave conditions at the toe of the Rhyl defences. This model simulates how waves develop and change (or 'transform') as they propagate from a deep water location to the shoreline. The industry-standard SWAN (Simulating WAves Nearshore) model was used, which is a third generation wave model capable of simulating the following nearshore wave transformation processes:

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

SWAN calculates steady state wave conditions for specific inputs of wave height, period and direction at an offshore boundary, and wind speed and direction applied across the model domain surface. Water levels can also be configured to account for tidal/surge variations.

Development of the model involved several stages, including: construction of a wave model grid, interpolation of a bathymetric dataset, calibration, joint probability analysis and extreme event modelling. To ensure accurate wave growth the model domain encompasses the majority of the Irish Sea, with land boundaries along North Wales, Western England, Southern Scotland and Eastern Ireland.

3.2.2 Wave model setup

Various data were required for the construction and calibration of the wave transformation model. Bathymetry and topography data were used to generate a grid of depth information (Figure 3-1). Modelled meteorological and wave data were used as boundary conditions to force the model. The model was calibrated against the following waverider buoys located in the Irish Sea (Figure 3-3):

- Liverpool Bay CEFAS WaveNet wave buoy (53°32'.01N, 003°21'.36W), for the period 13/11/2002 to 2012. This buoy is located in water of approximately 23m depth;
- Barrow Fugro GEOS wave buoy (53°59'.53N 003°19'.21W), for the period 21/01/2006 to 17/06/2006. This buoy was located in water of approximately 21m depth;
- Blackpool Sefton Council wave buoy (53.8188N, 3.1225W), for the period 30/09/2010 to 27/06/2011. This buoy was located in water of approximately 10m depth;
- Blackpool EA wave buoy (53°52'.50N, 003°02'.100W), for the period 23/01/2008 to 26/01/2008. This buoy was located in water of approximately 6m depth (i.e. shallow water).

3.2.3 Computational mesh

The model grid, with which SWAN performs its calculations of wave parameters, was designed using an unstructured mesh employing triangular elements. This type of grid allows for very high resolution detail around the North Wales coastline, whilst allowing for low resolution across the wider Irish Sea where high resolution detail was not required. The mesh resolution varies from 4km in deep areas of the Irish Sea to 10m in the shallow areas along the North Wales coastline where outputs were required. This high resolution allows the wave transformation processes to be computed with a high degree of accuracy, as sudden changes in depth will induce shoaling, diffraction and breaking processes. The wave model comprised 200,115 computational nodes.

3.2.4 Bathymetry data

The bathymetry for the computational mesh was constructed based on two sources of data. The wider bathymetric information was sourced from X, Y, Z survey points derived from surveys undertaken by the Civil Hydrographic Programme, Royal Navy surveys, Centre for Environment, Fisheries and Aquaculture Science (CEFAS) surveys as well as surveys from local port and harbour authorities. The data were supplied by FindMAPS[7] as a gridded dataset, processed and output into a 0.5 arc second grid covering the wider Irish Sea region. The data were also inspected, once merged, to ensure that the locations where datasets intersected did not experience a discontinuity in bathymetry, which would distort the wave transformation processes. Figure 3-1 shows the wave model computational mesh and bathymetry.

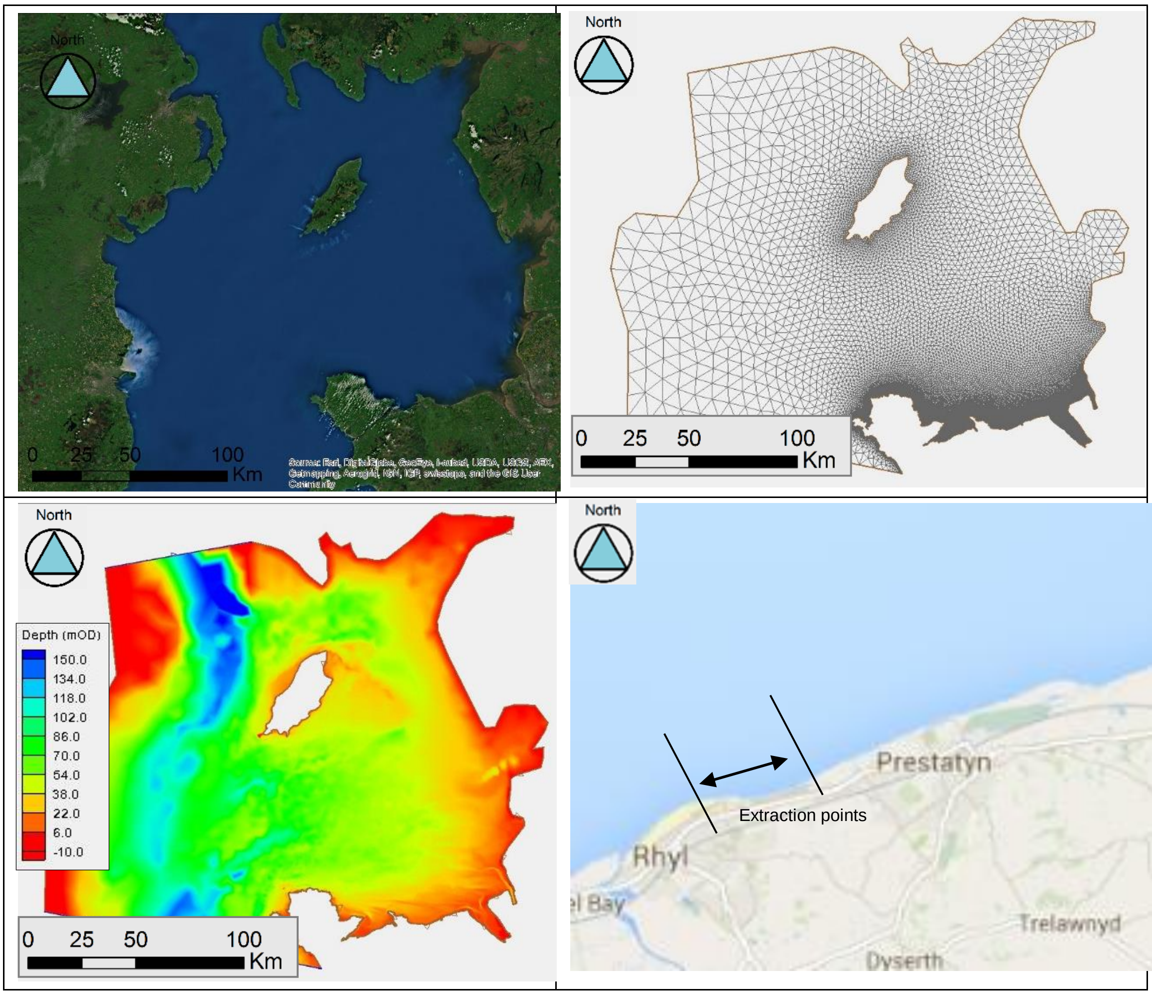

Figure 3-1: SWAN model computational grid, showing: Top left: Extent. Top right: Computational mesh. Bottom left: model bathymetry. Bottom right: example of wave reporting point (Contains Ordnance Survey data © Crown copyright and database right 2014).

[7] FIND Mapping Limited 2013

3.2.5 Calibration

The wave model was calibrated against three wave buoys located in the Irish Sea; Liverpool wave buoy, Blackpool wave buoy and Barrow wave buoy (Figure 3-3). Observed waves parameters were compared to simulated information to verify the model performance. For specific calibration events the wave model reported an average error of 0.35m, 0.17m and 0.20m at Liverpool, Blackpool and Barrow respectively. This represents an average standard error across the gauges of 0.24m. This is considered appropriate for further use in this study.

3.2.6 Model simulations

The model was used to estimate the nearshore wave conditions for a number of design coastal events between 1 in 1-year to 1 in 200-years, in addition to the December 2013 event.

3.2.6.1 Design events

Extreme design wind conditions were calculated using the British Standard BS6399[8] which provides estimates of hourly wind speeds with a standard 50-year return period. Several factors were applied to the 50-year hourly wind speed to account for altitude, direction and seasonality, and a number of return period factors applied to calculate the extreme design wind conditions for each location. The extreme design wind speed formula is:

$$U_D = U_b S_a S_d S_p S_f S_w$$

Where U_D is the design wind speed (m/s), U_b is the 50-year basic hourly wind speed (m/s), S_a is an altitude factor, S_d is a factor to account for the wind direction (e.g. south-westerly winds tend to be stronger than north-easterlies over the England and Wales), S_p is a factor to adjust for different return periods, S_f is a factor to convert hourly wind speed to a more appropriate duration for the water body under study and S_W is an over-water speed-up factor to account for the effect of reduced friction as wind travels over water.

3.2.6.2 Joint probability

A joint probability analysis was undertaken to consider the likelihood of significant winds and water levels coinciding during an extreme event. The level of dependence between wind and water levels was calculated using the industry standard Department of the Environment, Food and Rural Affairs (Defra) desk-based method[9]. A dependence value, χ (chi), of 0.3 was applied based on the surge vs wind speed dependence estimates presented in Defra technical report on dependence mapping[10], as shown in Figure 3-2.

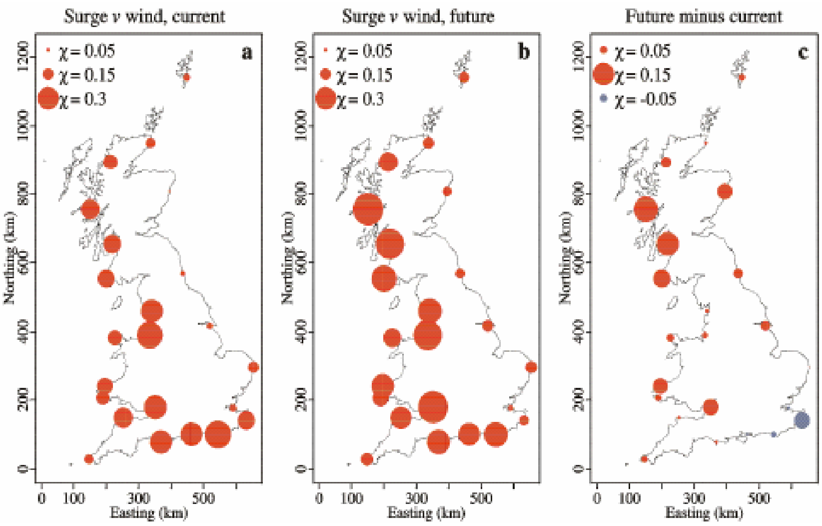

Figure 3-2: Dependence of surge vs wind conditions (Defra 2005)

[8] British Standard, 1997, BS 6399-2 Loading for buildings – Part 2: Code of practice for wind loads

[9] Defra (2005) Use of Joint Probability Methods in Flood Management: A Guide to Best Practice, Defra and the Environment Agency, March 2005, including associated spreadsheet.

[10] Defra (2005) Joint Probability: Dependence Mapping and Best Practice: Technical report on dependence mapping R&D Technical Report FD2308/TR1 March 2005

3.2.6.3 Wave model set up for the December 2013 event

The calibrated wave model was used to simulate the December 2013 event. To simulate the event as closely as possible the SWAN model was forced with the following (refer to Figure 3-3):

- Met Office hindcast event data for the north boundary. The data location used was Ref: 1,999. Waves used on the northern boundary were had a significant wave height of 4.44m, peak period of 8.33s and a direction of 309º.
- Met Office hindcast event data for the southern boundary. The data location used was Ref: 1,352. Waves used on the southern boundary had a significant wave height of 2.60m, peak period of 6.58s and a direction of 243º.
- Hindcast Met Office wind data for the event was 18.98m/s from 258º
- Recorded water level for the event of 5.65 mAOD in Rhyl harbour.

The wave model was driven with these variables to simulate the 2013 event, with the nearshore wave conditions used to estimate the resulting overtopping along the Rhyl coastal defence.

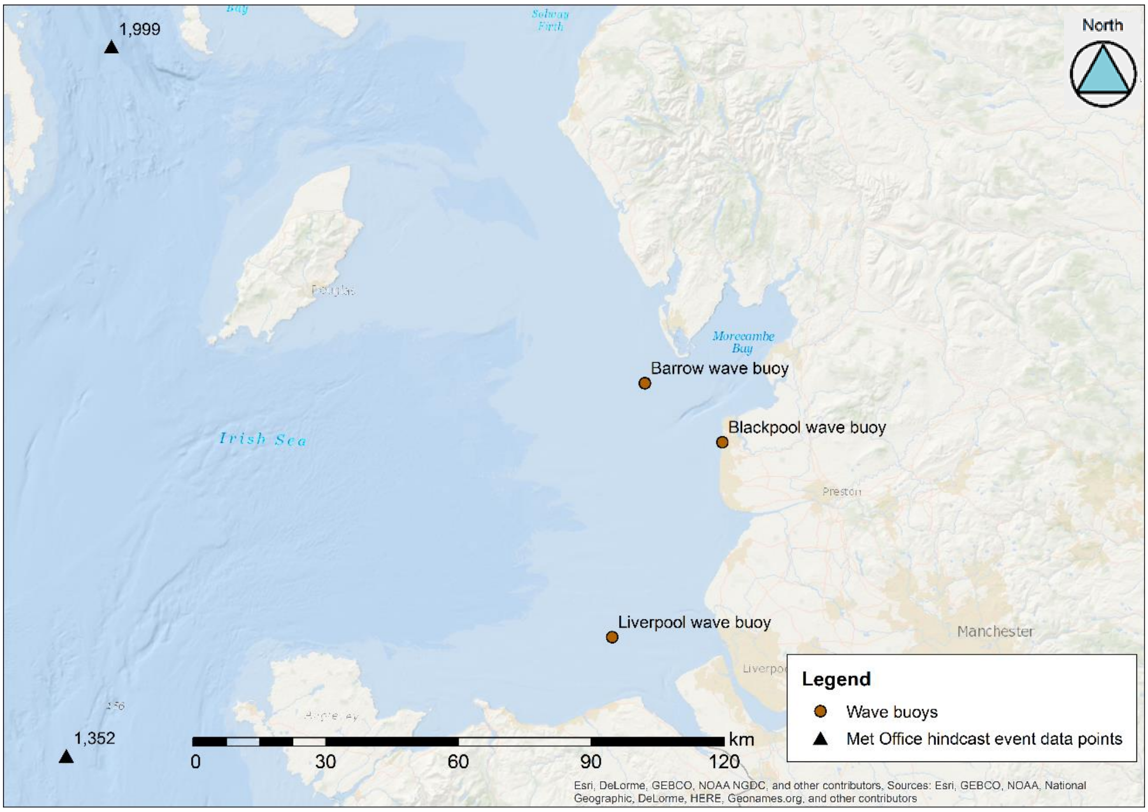

Figure 3-3: SWAN model Met Office hindcast event data points and wave buoys used for calibration.

3.3 Wave overtopping

3.3.1 Approach 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, which was used for this study. This empirical-based model is described in the industry standard EurOtop[11] 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 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,

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

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); armour width (Gc); berm elevation (hb); berm width (B); upper slope (α_u); lower slope (α_d); and roughness (γ_f) (see Figure 3-4).

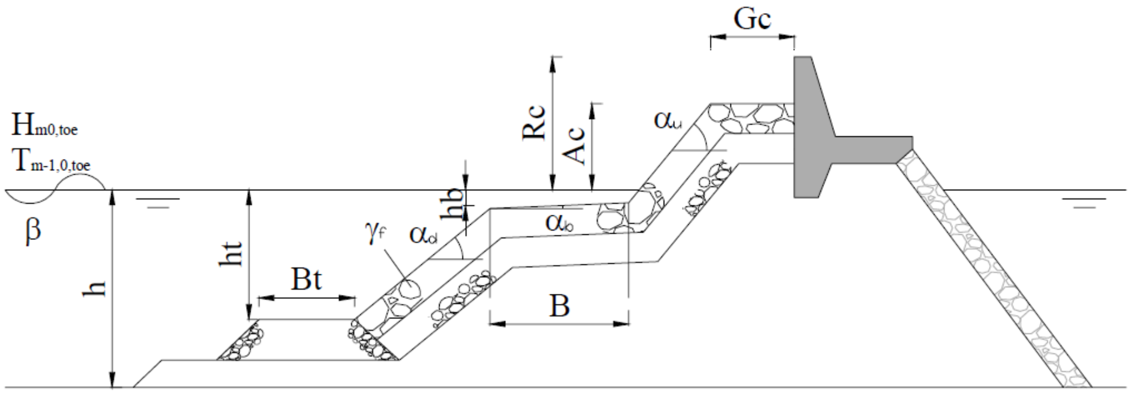

Figure 3-4: Schematisations of a typical beach profile for analysis using the Neural Network overtopping tool.

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 3-1 and Table 3-2 respectively. The limits for damage to the defences by overtopping discharge is presented in Table 3-3. As discussed within this report, these tolerable limits provide a basis for the design of mitigation strategies.

Table 3-1: 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 3-2: 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[12]	100 – 1,000
Driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets.	0.01 – 0.05[13]	5 – 50 at high level or velocity

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

Table 3-3: 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[14]
Damage to equipment set back 5-10m	0.4[15]
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

3.3.2 Overtopping model setup

The Rhyl coastal defence is a composite of a number of sections, varying in form and material. The defence was divided into seven sections and schematised using the 15 Neural Network parameters. The profiles schematisations were based on field survey supplied by the Denbighshire County Council, based on surveyed coastal profiles referenced as DCC02, DCC04, DCC05, DCC06, DCC08, DCC10 and DCC12. The locations of the surveys profiles are displayed in Figure 3-6, which were surveyed in 2010. An example of a schematised defence section is shown in Figure 3-5.

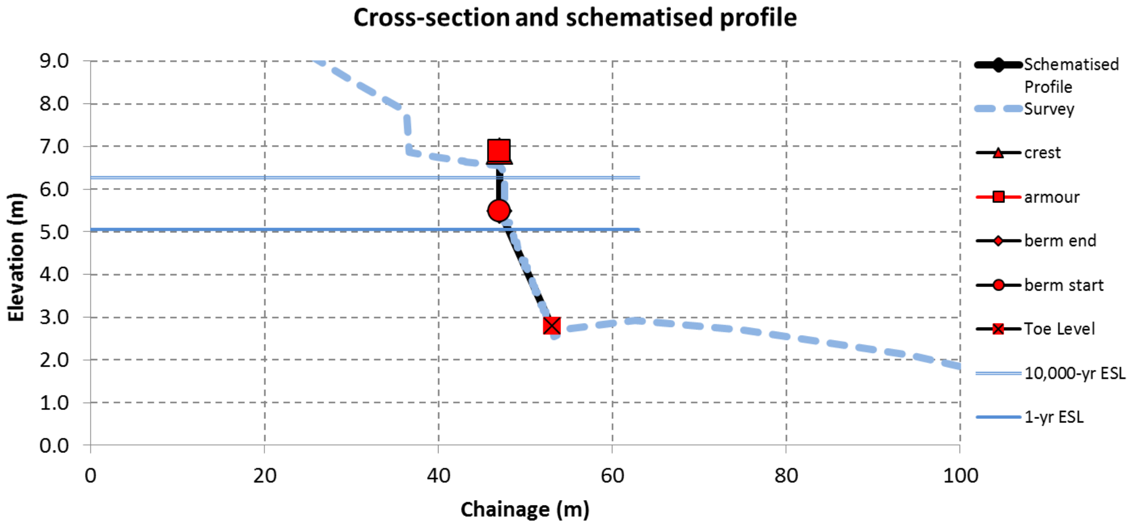

Figure 3-5: Defence profile schematised using Neural Network.

3.3.3 Overtopping model results

The overtopping modelling was performed for the seven defence sections for a number of joint probability scenarios, ranging from 1 to 200-year return periods. Under present day conditions the rate of overtopping for each worst-case return period simulation is shown in Table 3-4. These range from 0 to 92 l/s/m for a 1-year event, and 25 to 531 l/s/m for the 200-year event.

[14] Note: This limit relates to the effective overtopping defined at the building

[15] Note; This limit relate to overtopping defined at the defence

Table 3-4: Calculated overtopping rates at Rhyl.

	Overtopping rates at l/s/m						
Return period (year)	DCC02	DCC04	DCC05	DCC06	DCC08	DCC10	DCC12
1	16	3	92	2	0	0	22
5	41	15	210	8	1	2	103
10	59	25	278	15	4	3	166
20	76	38	328	24	7	6	229
50	100	63	409	41	16	12	259
100	118	88*	455	48	20	16	>260*
200	140	>90*	531	65	34	25	>260*
Dec 2013	73	56	340	20	7	7	177
* For these conditions the Neural Network is predicting high overtopping, which is capped as the parameters exceeding the calculation limits. While an estimate is given, it is assumed that the rate of overtopping will increase beyond 280 l/s/m as the magnitude of storm increases.							

3.4 Standard of defence

The modelling shows that the SoP varies along the frontage due to the changing wave conditions and defence profile. Generally profile DCC05 (located to the west of Splash Point) experiences the worst overtopping, ranging from 92 l/s/m in a 1-year event to 531 l/s/m in a 200-year event. In contrast profile DCC08 (fronting the golf course) is free of overtopping during a 1-year event and is estimated to have 34 l/s/m in the 200-year.

In comparison to the EurOtop guidelines on permissible discharge, and considering the uncertainties within the modelling, a 1 l/s/m discharge is considered to be the onset of wave overtopping, while a 10 l/s/m is considered to result in unsafe conditions and is the limit of protection from overtopping inundation. Using the latter, the lowest SoP is at DCC05 which is considered to be less than 1-year, and the highest is at DCC08 considered to be between 20 to 50-years return period. The modelling suggests four distinct zones of protection, is shown in Table 3-4:

- > 1-year to 5-year at DCC02, DCC04, DCC05 and DCC12
- Between 5 to 20-year at DCC06
- Between 20 to 50-year at DCC08 and DCC10.

These estimates were compared to previous investigations to ensure the calculations reflect observations made at the defences. The Rhyl to Prestatyn Coastal Defences Strategy Study Report (WMA 2012)[16] includes anecdotal information that the area around profile DCC02 (Butterton Road and John Street) experiences overtopping every two to three years. The flooding is accompanied by debris being washed over the sea defences, which indicates a rate greater than just spray. The overtopping rate calculated at this location would be above 20 l/s/m for such an event, and would be support of the anecdotal information.

[16] MWA (2012) Rhyl - Prestatyn Coastal Defences Strategy Study Report, Denbighshire County Council, 2012 August, Martin Wright Associated.

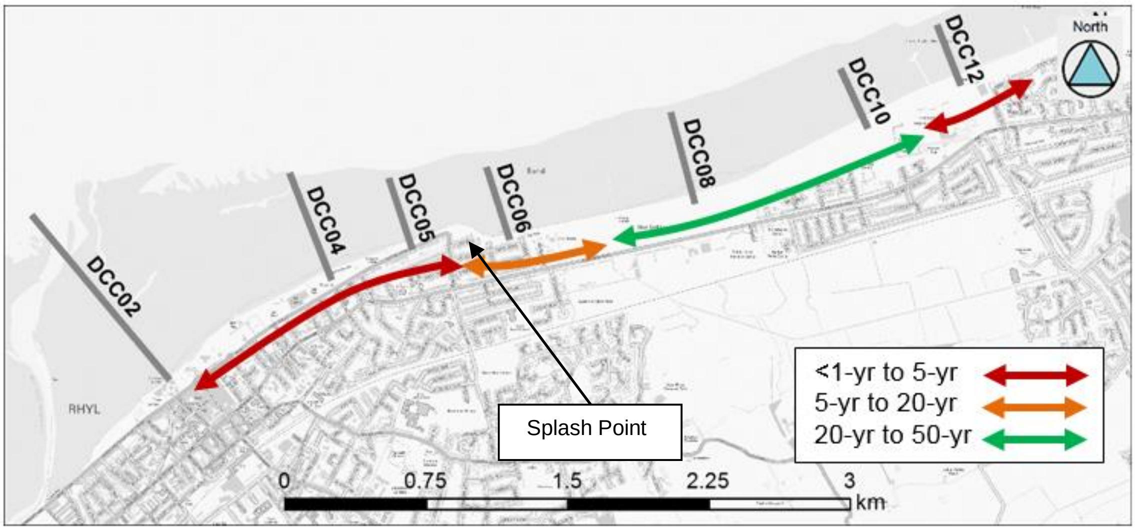

Figure 3-6: Survey cross-profiles used in the Neural Network overtopping and estimated overtopping SoP (Contains Ordnance Survey data © Crown copyright and database right 2014)

4 Flood inundation modelling

4.1 Introduction

This section describes the modelling used to estimate the inundation due to extreme coastal events. Waves overtopping the defences will inundate the surrounding coastal floodplain, and have the potential to cause widespread flooding. This has been estimated using a hydrodynamic model coupled to the overtopping estimates summarised in Section 3.3. This Section describes the model development and presents the estimated inundation outlines.

4.2 Model overview

4.2.1 Summary of model setup

Modelling for this study was undertaken using a 2D hydrodynamic model constructed using TUFLOW[17]. The model was used to estimate the coastal inundation extent for a design 200-year coastal event, including and excluding the impacts of climate change to 2115. It was then used to estimate the December 2013 flood extends based on new wave overtopping calculations.

The model extends from Splash Point in the west to Ffrith Beach near Prestatyn in the east, covering an area of 10.92 km^2 as shown in Figure 4-1. The model used a 2.00m resolution with a timestep of two seconds.

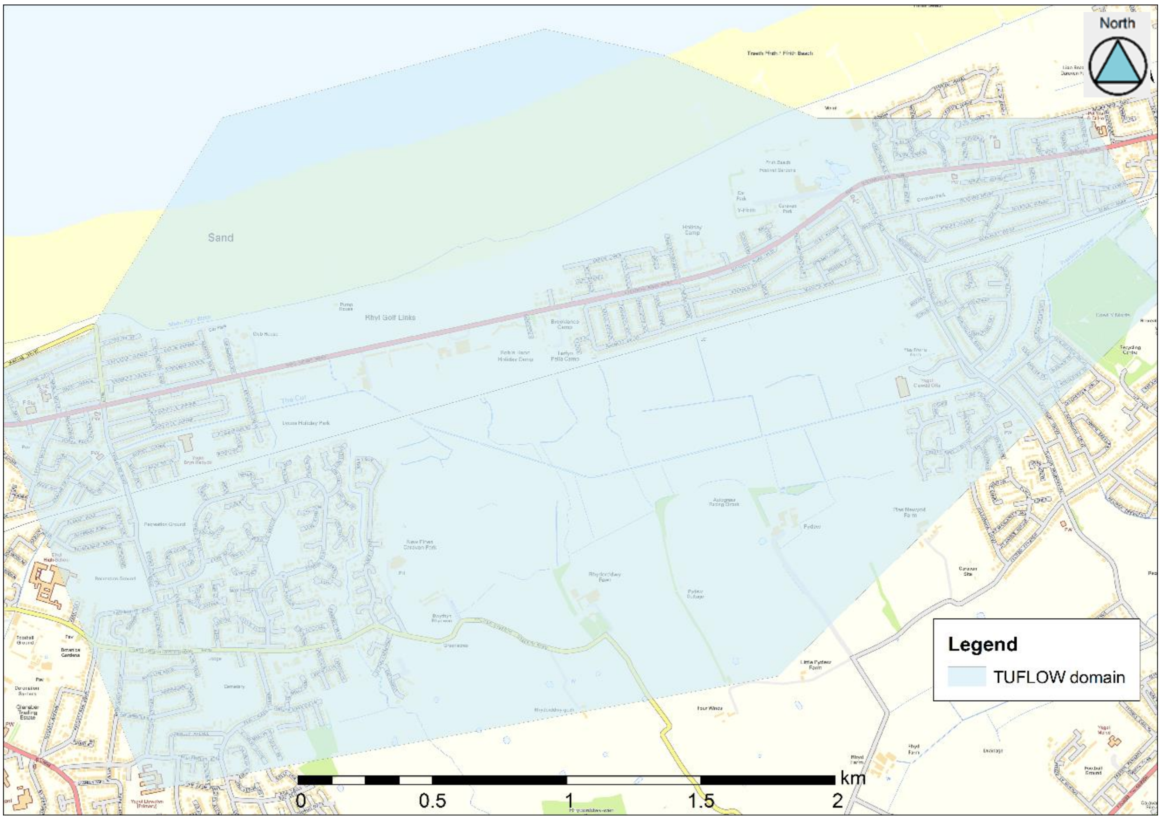

Figure 4-1: Rhyl TUFLOW model domain (Contains Ordnance Survey data © Crown copyright and database right 2014).

4.2.2 Topography and roughness

Hydraulic roughness across the 2D model domain was established using material classifications derived from Ordnance Survey (OS) MasterMap data. An appropriate Manning's *n* value was applied to each of these classifications derived from Hicks and Mason (1998)[18] and cross-checked with Chow (2009)[19]. The values used within the model are shown in Table 4-1.

[17] TUFLOW version 2013-12-AB-w64, 64bit. http://www.tuflow.com/.

[18] Hicks, D.M. & Mason, P.D., Roughness Characteristics of New Zealand Rivers, NIWA, Christchurch, (1998), 329pp.

[19] Chow, V.T. (1959). Open Channel Hydraulics. New York, NY: McGraw-Hill Book Co.

Table 4-1: Land use descriptions and applied Manning's *n* Values.

Land use description	Manning's n
Buildings	0.300
Inland and coastal water	0.030
Natural surface and gardens	0.070
Manmade surface roads and paths	0.025
Trees, rough land and scrub	0.100
Marsh, reeds or saltmarsh	0.046
Structures	0.100

4.2.3 Modifications to the Digital Terrain Model

TUFLOW requires a topographic grid, or Digital Terrain Model (DTM), to represent the surface of the earth. Ground level information was derived from Light Detection and Ranging (LIDAR) data and nearshore bathymetry. The two datasets were smoothed together in ArcGIS to minimised transitions which could cause model instabilities. Several changes were made to the DTM prior such as the following.

- Identification of blocked flow paths and channels. The DTM was reviewed to identify blocked flow paths. The LIDAR data were provided in "filtered" format and therefore excluded buildings and vegetation that could block flow paths. Additionally it did not include large drainage systems which would provide a conduit for water, such as through the railway embankment. The DTM was edited to reflect realistic flow paths supported by OS mapping, the LIDAR DTM, site visits and through an iterative process of inspecting draft model results.
- Drainage lines. The network of drains located to the south of the railway were included into the model DTM. These drains are typically only a few metres wide and therefore on the limit of the 2m grid resolution. These have been checked for continuity with any small obstructions removed to ensure water flow.
- Flood defences. Flood defences were added into the model as 3D breaklines to ensure accurate description of the defence crest.
- Representation of buildings. A relatively high Manning's n value was applied to represent individual buildings and a lower value to represent the surrounding roads and gardens. For this model setup water flow may pass across the building accounting for flood storage, however will be limited by the increased resistance.
- Initial water level. Initial water levels were set in the model domain to represent a low tide (e.g. the land was dry at the start of the model simulation).
- Hydraulic structures. In-channel structures such as bridges were not included in the modelling. It is beyond the scope of this study to accurately survey and model these structures. Therefore, the flow that occurs within the model is largely assumed to be open channel.

4.2.4 Model boundaries and simulation

Two model boundaries have been used for this study. A tidal boundary which runs parallel to the coastline and ties into high ground at Splash Point to the west and Ffrith Beaches near Prestatyn in the east. The second boundary is the wave overtopping boundary which is applied landward of the coastal defences.

4.2.4.1 Tidal boundary

The tidal boundary applies a time-varying sea level which includes the underlying astronomical tide and a component of surge to make the overall extreme sea level. The underlying tide is based on an interpolation of available tidal signatures from Llandudno and Hilbre Island. The surge and final extreme sea level was based on the latest coastal extreme guidance for the UK[20]. The peak extreme sea level used in each simulation is shown in Table 4-2.

[20] Defra. SEPA. The Scottish Government. Environment Agency (2011). Coastal flood boundary conditions for UK mainland

Table 4-2: Extreme sea-level data use in the derivation of design tidal-graphs.

Scenario	Extreme sea level (Ref point 1,134)
200-year present day	5.77mAOD
200-year including climate change to 2115	6.52 mAOD
December 2013 event	5.65mAOD*
*Water level based on recorded data from Rhyl harbour.	

4.2.4.2 Wave overtopping boundary

Three water inflow lines were used to represent overtopping into the model, as shown in Figure 4-2. The wave overtopping was calculated using the Neural Network and injected into the model landward of the coastal defence to simulate overtopping water. Wave overtopping was calculated at the following locations:

- Splash Point defence (DCC06)
- Defence fronting the golf course (DCC08)
- Defence fronting the dunes to the east of the model (DCC10).

The peak overtopping rates used in the TUFLOW model are shown in Table 4-3

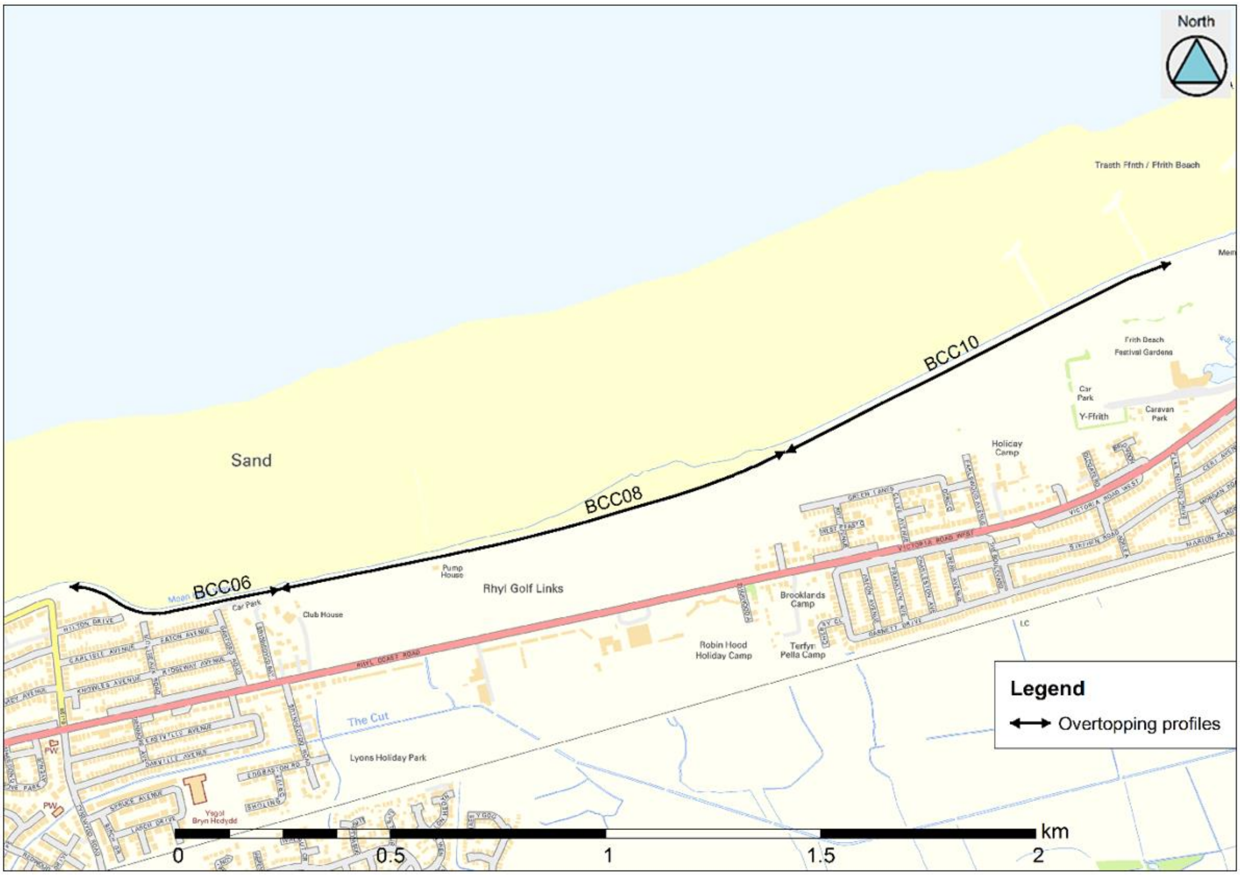

Figure 4-2: Overtopping profiles used in Rhyl TUFLOW model (Contains Ordnance Survey data © Crown copyright and database right 2014).

Table 4-3: TUFLOW model peak overtopping discharge at each of the defence sections.

	Overtopping rates at l/s/m		
Return period (year)	DCC06	DCC08	DCC10
200-year present day	65	34	25

and islands. Project: SC060064/TR2: Design sea-levels.

200-year including climate change (to 2115)	100	65	43
December 2013	20	7	7

4.2.4.3 Climate change

In addition to present day extreme still water level events, a 200-year climate change scenario for the year 2115 was modelled. The water level rise for climate change was based on the latest UKCP09 sea-level change guidance[21] using the medium emission 95th percentile scenario. The water level increase for climate change (from 2014 to 2115) was 0.78m for the study area.

4.2.4.4 Model simulation

The model was run for four consecutive tides, i.e. approximately two days, with wave overtopping simulated during the initial two high tides. The additional time then allowed the maximum water extent to be mapped, allowing the water to spread through the coastal floodplain.

4.3 Model results

The result of the flood inundation modelling are provided in Appendix B.1. The modelling indicates a 200-year present day coastal event would result in widespread inundation, covering an area of approximately 2.24km^2. The inundation includes the Rhyl links golf course and farm land on the landward side of the railway, including the surrounding properties. The deepest inundation on the coastal frontage occurs at the end of Hilton Drive near Splash Point, where the 200-year output obtains a maximum depth of 1.50m.

Modelling of the 200-year plus climate change event shows a larger area due to the increased rate of wave overtopping, covering an area of approximately 2.76km^2 . The estimated depth at the end of Hilton Drive near Splash Point is 1.56m.

In comparison, the modelling of the estimated December 2013 shows inundation was primarily contained to the north of Rhyl Coast Road, with an estimated extent of 0.44km^2. The estimated depth at the end of Hilton Drive near Splash Point is 0.36m. The modelled flood extent approximately matched observed data provided by Denbighshire County Council indicating the validity of the model (shown in Appendix B.2).

4.3.1 Model assumptions and limitations

There remains uncertainty in the estimated inundation extents due to several factors. These include the following:

- Unfortunately, there is no single model capable of simulating all the processes occurring as waves propagate towards and overtop a coastal defence. Therefore a suite of numerical models were used for this assessment. As a result of these limitations, and as appropriate in all complex modelling studies, the model results have been used in conjunction with a wider range of supporting information (e.g. anecdotal reports, photographs, surveys, etc.) to estimate inundation extents.
- The models have been used to simulate a sequence of events; first transforming offshore wave conditions to nearshore, before calculating overtopping and inundation. As such, any uncertainty in the offshore conditions and joint probability assessments will be present throughout the entire process.
- The inundation model only accounts for flooding from coastal and tidal sources. Surface water flooding and sewer surcharge is not accounted for.
- Channel openings were modelled using a gully line approach that lowers the DTM to that of the channel bed.
- Topography roughness values used in the model were derived approximately from Hicks and Mason (1998)[22], and cross-checked with Chow (2009)[23]. However, there is no

[21] http://ukclimateprojections.defra.gov.uk/

[22] Hicks, D.M. & Mason, P.D., Roughness Characteristics of New Zealand Rivers, NIWA, Christchurch, (1998), 329pp.

[23] Chow, V.T. (1959). Open Channel Hydraulics. New York, NY: McGraw-Hill Book Co.

definitive guidance on defining roughness values for 2D hydraulic models. It is assumed that the values used are representative.

- The wave overtopping input boundary is applied landward of the coastal defences. Difficulties arise where a defence is at an angle to the model grid, which introduces a degree of 'staircasing' into the model. This has been identified and minimised where possible.

JBA consulting

5 Consideration of December 2013

5.1 Introduction

The December 2013 coastal event resulted in significant overtopping and inundation behind the Rhyl defence, and was the catalyst for this study. This section investigates the magnitude of the event in order to support further coastal engineering advice. This was undertaken using the new wave overtopping estimates undertaken in this report, which have been compared against previous investigations undertaken by JBA Consulting for Natural Resources Wales (NRW).

5.2 Estimated overtopping rate

The estimated rate of wave overtopping during the December 2013 event was compared against standard 'design' events developed using the Defra method for assessing joint probability to gauge the magnitude of the storm. Due to storm specific conditions, such as wave angle and sea-level, the event resulted in varying rates of wave overtopping along the frontage. The worst rate of overtopping is considered to have a return period of up approximately 40 years, and was estimated near Splash Point. The modelled overtopping rates along the frontage and estimated return periods are shown in Table 5-1.

Table 5-1 Estimated overtopping rate and return period for the December 2013 event.

2013-event	DCC02	DCC04	DCC05	DCC06	DCC08	DCC10	DCC12
Calculated overtopping rates at (l/s/m)	73	56	340	20	7	7	177
Return period (years)	18	42	24	16	20	25	12

5.3 Summary of previous assessments of the 2013 event

Previous assessments of the 2013 event have been undertaken by JBA Consulting for Natural Resources Wales (NRW) based on water levels and wave heights observed during the event in isolation, and considered the joint probability of the two conditions using the 'desk study' approach outlined in Defra's "Use of joint probability Methods in Flood Management: A guide to best practice"[24].

Observed water levels were recorded at Rhyl Harbour which were checked against the extreme values calculated within the CFBD (refer to Table 2-2). The water level of 5.65mAOD is calculated to have a return period of approximately 1 in 100-years. This is in contrast to the observed water levels at the Liverpool gauge, where the recorded 6.22mAOD has a 40-year return period. The discrepancy between two locations in such close proximity suggests there is a level of uncertainty in the data; potentially due to errors in the gauge records.

Wave conditions were considered in two ways. First, the offshore wave records were obtained from the Liverpool wave buoy, and compared against the extreme swell wave conditions published within the CFBD (refer to Table 2-3), which suggests a return period of over 1,000 years (far above the published maximum). A second assessment was undertaken using a Peak Over Threshold (POT) analysis, where the return period was calculated based on the entire Liverpool wave buoy record, and the respective rank of the 2013 event. The wave height of 4.60m was considered to be the 8th largest during the 10.90 year record (further showing the limited confidence in the CFBD extreme wave dataset), with an estimated return period of 1 in 1.37 years. Importantly, larger events have been recorded such as that in February 2004 which a peak significant wave height of 5.37m.

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. Using the Defra best practice guidance, a joint probability assessment was undertaken using the extreme water levels from the CFBD and the extreme waves calculated using the POT analysis. When considered to occur coincidently, the storm event is estimated to have had a joint probability of 4,800-years at Liverpool and 9,300-years at Rhyl.

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

Such figures seem too extreme to be credible, with the assessment questioning the ability of the Defra joint probability approach to correctly predict the coincidence of extreme waves and sea levels.

6 Summary and conclusions

This investigation was undertaken by JBA Consulting on behalf of Denbighshire County Council to consider the standard of protection (SoP) of the Rhyl coastal defences. The study has three aims:

1. To establish the current Standard of Protection (SoP).
2. To estimate the inundation due to a design 200-year present day coastal event, a 200-year plus climate change (to 2115) event and the December 2013 event.
3. To consider the magnitude of the December 2013 event at Rhyl.

At present there is no one numerical model or calculation approach able to replicate all of the processes occurring in the coastal zone. Instead, this study utilised a suite of numerical models to calculate the nearshore wave conditions, the overtopping rate and the resulting inundation due to extreme coastal events.

The overtopping modelling shows a varying SoP along the Rhyl defences, ranging from under 1 in 1-year to 50-years. The wave overtopping along Rhyl frontage was estimated for the December 2013 event, and is considered to have a return period of 1 in 40-years.

Using a hydrodynamic TUFLOW model the inundation resulting from an extreme coastal event was estimated. The modelling indicates a 200-year present day coastal event would result in widespread inundation, spreading landward of the Railway and covering an area of approximately 2.24km^2. This is expected to increase due to the effect of climate change, with larger overtopping and sea levels increasing the inundation to approximately 2.76km^2. In contrast the modelling of the estimated December 2013 overtopping shows inundation was primarily contained to the north of Rhyl Coast Road, with an estimated extent of 0.44km^2.

A key recommendation of this study is that further assessment should be undertaken to address limitations encountered in the numerical modelling. For coastal engineering and flood risk assessments it is essential that there is a source of high quality coastal extreme data and an accurate methodology for undertaking joint probability assessments. Previous assessments of the December 2013 event show a growing concern as to whether available offshore wave estimates accurately represent extreme conditions, and if the Defra joint probability methodology correctly predicts the coincidence of extreme waves and sea levels. While this study used extreme wind speeds to drive wave models, there remains the uncertainty due to joint probability of wind and sea levels used to develop design scenarios. It is important that a revised joint probability assessment is undertaken to increase the reliability of nearshore wave and overtopping estimates, adopting a methodology such as that proposed by Heffernan and Tawn[25]. It is recommended that this is conducted prior to any future upgrade to the Rhyl defences, which will ensure it is designed to an appropriate SoP. Any upgrades should be designed to include the impact of climate change, which can produce far greater rates of overtopping and inundation consequences.

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

A Appendix A

A.1 Model control files and general model settings

Table A-6-1: Rhyl model control files.

Scenario	Events	Control Files	BC Database File	Geometry Control File
Defended	200, 200 including climate change to 2115, 2013	Rhyl_~e1~_~s1~.tcf	PR_bc_dbase_Rhyl.csv	Rhyl_Def_001.tgc

Boundary Control File	Materials Control File	Approx. Run Time (hrs)	Computer OS Req.
Rhyl_Def_001.tbc	Rhyl_001.tmf	127	64bit

Table A-6-2: Rhyl general model settings.

General Settings	
Start Time (hrs)	42
End Time (hrs)	91.25
Grid Cell Size (m)	2
Timestep (s)	1
Map Output Settings	
Map Output Format	XMDF
Map Output Data Types	d h v ZUK0
Start Map Output Time (hrs)	42
Map Output Interval (s)	1,800.0
Time Series Output Interval (s)	60.0

B TUFLOW inundation and depth maps

B.1 Rhyl TUFLOW inundation extents

B.2 2013-year inundation extent with event validation

B.3 200-year depth grid

B.4 200-year including climate change 2115 depth grid

B.5 2013-year depth grid

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