REPORT

Numerical Modelling Report

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HASKONINGDHV UK LTD.

Regus House
Exeter Business Park
1 Emperor Way
Exeter
EX1 3Qs
United Kingdom
Water & Maritime
VAT registration number: 792428892

+44 1392 447999 **T**
info.haywards.heath@uk.rhdhv.com **E**
royalhaskoningdhv.com **W**

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Author(s):	Thomas Green

Drafted by:	Thomas Green
Checked by:	Lucia Stothert
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Approved by:	Keming Hu
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Table of Contents

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Table of Tables

Table of Figures

1 Introduction

1.1 Sidmouth Beach Management Scheme extent

This report provides details of the approach to numerical modelling undertaken as part of the development of the Sidmouth Beach Management Scheme. The scheme is located in Sidmouth, East Devon as illustrated in Figure 1-1.

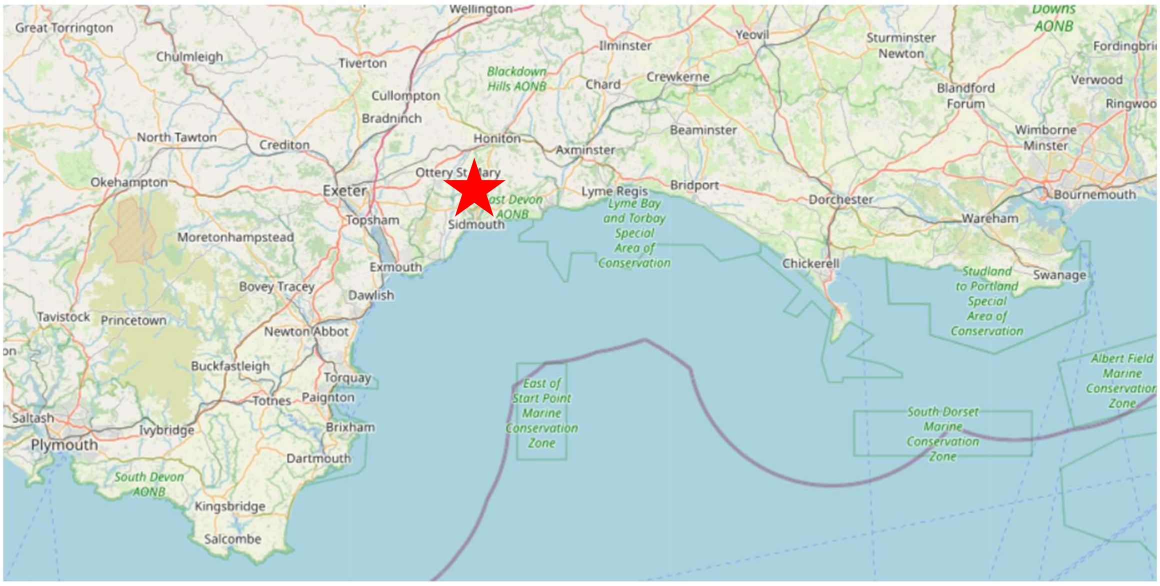

Figure 1-1: Sidmouth Beach Management Scheme Location Plan

The scheme frontage itself is approximately 1,250m in length and extends from Connaught Gardens in the west to East Beach / Pennington Point cliffs in the east (Grid Ref: SY 12633 87194), see Figure 1-2.

Figure 1-2: Sidmouth Beach Management Scheme Extent

1.2 Sidmouth’s current flood and coastal erosion risk management defences

The Sidmouth Beach Management Scheme frontage has a long history of construction and maintenance of coastal flooding and erosion risk management schemes (see Figure 1-3). Following sea storms of 1989 and 1990, the Sidmouth Town frontage experienced substantial damage to existing defences and large volumes of shingle were lost to the east of Sidmouth. This storm damage triggered the need for upgraded coastal flood and erosion risk management measures. The need for further works was triggered following storms in 1993 and 1994.

The current flood and coastal erosion risk management measures along the Sidmouth Town frontage were constructed over many phases between 1991 and 2000 and comprise: seawalls, rock revetments, splash wall, rock groynes, offshore breakwaters and a river training wall, coupled with beach recharge and recycling as required. Wave overtopping and the subsequent risk of coastal flooding along the Sidmouth town frontage is generally controlled by the retained beach in front of the seawall, the recurved seawall itself and the low splash wall on the landward side of the promenade. The beach, in conjunction with the buried rock revetment, also helps to protect the seawall from undermining and subsequent potential failure. Beach levels are now lower than 1990s design level and in places the toe of the seawall is exposed. This results in wave breaking and reflect on the exposed vertical seawall which exacerbate wave overtopping and thus the risk of flooding. Wave reflection also exacerbates the reduction in the already low beach levels.

The East Beach frontage consists of Pennington Point cliffs which has a small shingle beach at its base. The cliffs are otherwise undefended. The Pennington Point cliffs are eroding, and thereby retreating. One of the causes of erosion is wave impact on the lower cliff. Beach levels have lowered in recent years causing more exposure of the cliff toe to wave action. The continued erosion of the cliffs is exposing the river Sid wall to sea storm events from the south and east direction. This poses a risk of storm waters outflanking and overtopping the river defences with increased risk of defence failure due to the fact that the river Sid wall was not built to withstand sea storm forces.

Figure 1-3: Sidmouth’s current coastal defence arrangement

2 Approach to numerical modelling

Numerical modelling was undertaken in two stages. Stage 1 was to determine the present and future flood risk under a ‘Do Nothing’ scenario. Stage 2 was to develop numerical models to test and refine the preferred options identified with the Sidmouth and East Beach Management Plan (BMP)[1].

When undertaking coastal numerical modelling, a number of sequential steps and tools are required to understand the flood mechanisms and determine the resultant flood risk. A schematic overview of the approach taken for Stage 1 is presented below in Figure 2-1. Details of each step are discussed in the sections below.

Figure 2-1: Approach to numerical modelling

For Stage 2, a sediment transport model was developed which was used with the wave overtopping modelling to appraise and develop the preferred option. However, no additional breakwater option was included in the original sediment transport model as at the time the modelling was undertaken, this option had been discounted on economic grounds.

At the time of undertaking the numerical and sediment transport models, the chosen option was to reinstate the 1990s design beach and raise the splash wall at the landward side of the promenade (Frontage B). On East Beach, the preferred option included beach renourishment and constructing one 120m long groyne (Frontage C). The success of the originally proposed scheme heavily relied on beach recharge and recycle at regular intervals.

Following the updated Partnership Funding Calculator for Flood and Coastal Risk Management (FCRM) projects by the Environment Agency in 2020, additional Flood Defence Grant in Aid (FDGiA) was released for the originally proposed flood defence scheme in 2017. This provided the basis for exploring alternative options. Therefore, East Devon District Council, in collaboration with an elected Steering Group, requested a high-level assessment of additional flood defence options, including but not limited to, options that were previously discounted during the development of the BMP for the main town (Frontage B) and East Beach (Frontage C), see Appendix A. A preferred option was shortlisted comprising the construction of an offshore breakwater on Frontage B in addition to localised raising of the splash wall and beach renourishment. On Frontage C, the same original proposal was chosen and taken forward to the revised Outline Business Case. However, the results of the numerical model were broadly considered still valid and further modelling was considered not necessary at this stage of the project. Similarly, the sediment transport model was not revised as a high-level assessment was undertaken for the preferred option which broadly confirmed similar results, and the model validation simulations showed how effective the existing breakwaters are at trapping material.

3 Stage 1

3.1 Derive offshore conditions

This section outlined the first stage of the numerical modelling process which is to obtain boundary conditions to be used to drive the wave transformation model. For this, extreme water levels, wave heights, wind speeds and wave periods were derived from a variety of sources.

[1] *East Devon District Council, Sidmouth to East Beach Management Plan, CH2M 2017*

3.1.1 Water level analysis

Extreme water levels are key parameters when running numerical models for coastal conditions, as these establish the depths of water the waves will travel through. It is also a key parameter that controls the wave conditions of the nearshore region. Baseline extreme water levels were obtained from the Environment Agency's (EA) Coastal Flood Boundary Data study (CFBD) for the UK Mainland and Islands (Environment Agency, 2011) which has a base date of 2008. This study provides sea-level estimates for a number of return periods. Output point 2410 fronts Sidmouth and was used for the assessment. Figure 3.1 presents the location of the CFBD point in relation to Sidmouth.

Figure 3-1: Location of EA CFBD Water Level Point

The Environment Agency study (2011) recommends that sea level rise be represented using the UK Climate Change Impact Projections 2009[2] (UKCP09) 'Medium Emissions' scenario at the 95% level. Sea level rise rates were obtained from the UKCP09 User Interface which provides sea level rise predictions in the vicinity of Sidmouth. The estimated sea level rise for Sidmouth is presented in Table 3-1. Sensitivity tests were undertaken following the updated sea level rise projections UKCP18[3] in 2019 and Environment Agency Coastal Flood Boundary Dataset and Climate Change[4] guidance and whilst levels increase, it was concluded that the modelling work undertaken to date did not need amending as the flood extent would have not changed significantly. However, updated water levels will be reviewed at detailed design stage.

Table 3-1: Sea Level Rise based on UKCP09 predictions.

Year (Epoch)	95th % Predicted Sea Level Rise (m)
2017 (present day)	0.048
2037 (20yrs)	0.168
2067 (50yrs)	0.379
2100 (100yrs)	0.658

[2] *UKCP09 - http://ukclimateprojections.metoffice.gov.uk/*
[3] *UKCP18 - http://ukclimateprojections.metoffice.gov.uk/*
[4] *Environment Agency Climate Change guidance - https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances#sea-level-allowances*

In determining future extreme water levels, in addition to sea level rise it is best practice to allow for a growth in storm surge within climate change assessments. In terms of growth in surge the Environment Agency recommends that a rigorous assessment of the current coastal extreme water level is undertaken. The assessment of growth in surge was based on the latest research available from the UKCP09 User Interface which provides estimations of growth in surge within the vicinity of Sidmouth. The data is provided in Table 3-2 below. An uncertainty level of 95% has been used as this is considered appropriate for this study.

Table 3-2: 95% Growth in Storm Surge – UKCP09 Latest Research

Long term linear trend in skew surge (1951 2099) for return period 1 : x Years.	95 % Growth in Storm Surge (mm/year)
2 years	0.369
10 years	0.589
20 years	0.669
50 years and above	0.768

The extreme water levels, which include sea level rise and surge, are presented in Table 3-3.

Table 3-3: Summary of extreme water levels

Return Period	Extreme Water Level (2017)	Extreme Water Level (2037)	Extreme Water Level (2067)	Extreme Water Level (2117)
T1	2.75	2.88	3.10	3.39
T2	2.81	2.94	3.16	3.45
T5	2.91	3.04	3.26	3.55
T10	2.98	3.12	3.34	3.64
T20	3.05	3.19	3.42	3.72
T25	3.07	3.21	3.44	3.74
T50	3.15	3.29	3.52	3.83
T75	3.18	3.32	3.55	3.86
T100	3.21	3.35	3.58	3.89
T150	3.27	3.41	3.64	3.95
T200	3.29	3.43	3.66	3.97
T500	3.40	3.54	3.77	4.08
T1000	3.50	3.64	3.87	4.18

Following the publication of the UKCP18[5] revised sea level rise predictions, a high-level assessment was undertaken to assess the likely impact of the recommended RCP. It was ascertained that whilst sea level rise will increase the overtopping rates pre and post scheme in the medium to long term, these were not significant enough in the short term to substantially affect the estimated damages, the appraisal outcomes or flood extents. Therefore, in light of the time pressure and budgetary constraints, it was agreed that no re-running of the model was justified. More detailed modelling with up-to-date predictions will be undertaken at detailed design stage.

[5] *UKCP18 - https://www.metoffice.gov.uk/research/approach/collaboration/ukcp/data/index*

3.1.2 Extreme wave heights

The coastal frontage of Sidmouth is orientated in a northeast to southwest direction. Waves approach the coastline from the southeast, south and southwest as illustrated in Figure 3-2. The predominant wave regime along the coastline is south westerly waves, however south easterly storm conditions do occur throughout the year as presented below.

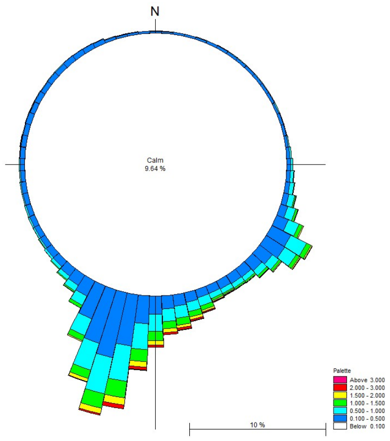

Figure 3-2: Met Office WaveWatch III Hindcast wave record

Wave Hindcast data was obtained from the CEFAS Wavenet Hindcast website which provided a time series of offshore conditions from 1st January 1980 through to the 31st December 2016. The dataset provided a time series of offshore wave height, wave period, direction and wind speed amongst other parameters. Figure 3-3 presents the location of the offshore data point. The data point matches the boundary of the wave transformation model which is discussed in the next section of this report.

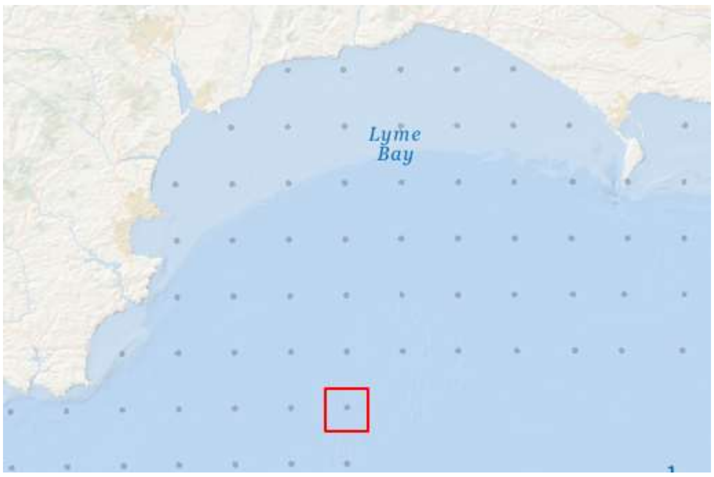

Figure 3-3: Location of Wavenet Hindcast output point

A statistical analysis was undertaken to derive extreme offshore wave heights. These were assigned to a 1 in 1, 10, 20, 50, 75, 100, 200 and 1000-year return periods and individual direction sectors of south east (90 – 150 degrees), south (150 – 210 degrees) and south west (210 – 270 degrees). Table 3-4 presents the extreme wave heights.

Table 3-4: Extreme offshore wave height by direction sector.

Direction Section / Return Period	South east 90 / 150deg	South 150 / 210 deg	South west 210 / 270 deg
T1	3.49	4.54	6.87

Direction Section / Return Period	South east 90 / 150deg	South 150 / 210 deg	South west 210 / 270 deg
T2	3.74	4.85	7.41
T5	4.02	5.21	8.13
T10	4.22	5.46	8.67
T20	4.39	5.68	9.21
T50	4.60	5.95	9.94
T75	4.67	6.04	10.21
T100	4.74	6.13	10.49
T200	4.86	6.30	11.04
T1000	5.10	6.63	12.32

Currently, there are no industry guidelines for addressing the impact of climate change and sea level rise on extreme wave heights. For this reason, the same extreme wave heights were applied across all four time epochs (2017, 2037, 2067 and 2117).

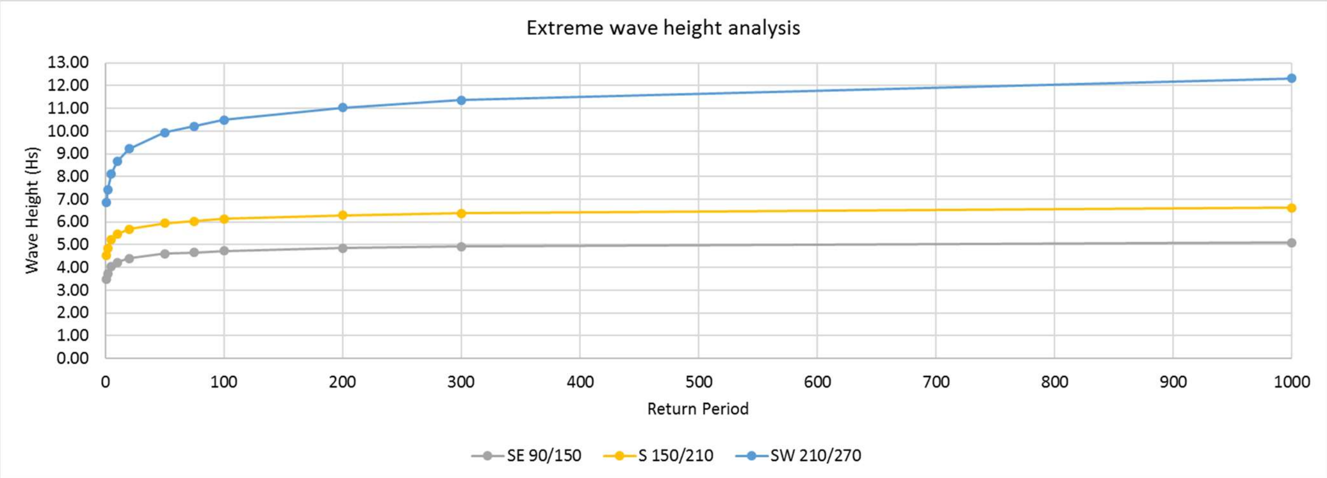

Figure 3-4: Extreme wave height analysis plot

3.1.3 Wind speed

Wind speed and direction is an important forcing factor required for wave transformation modelling as it ensures a realistic wave generation and propagation from offshore to nearshore within the model. Wind speed applied to the wave transformation model were based upon a 'typical wind speed' associated with the wave height. Wind speeds were derived by undertaking an analysis of the offshore wave height and wind speed data from the Hindcast dataset. The relationship between wind speed and wave height was then defined.

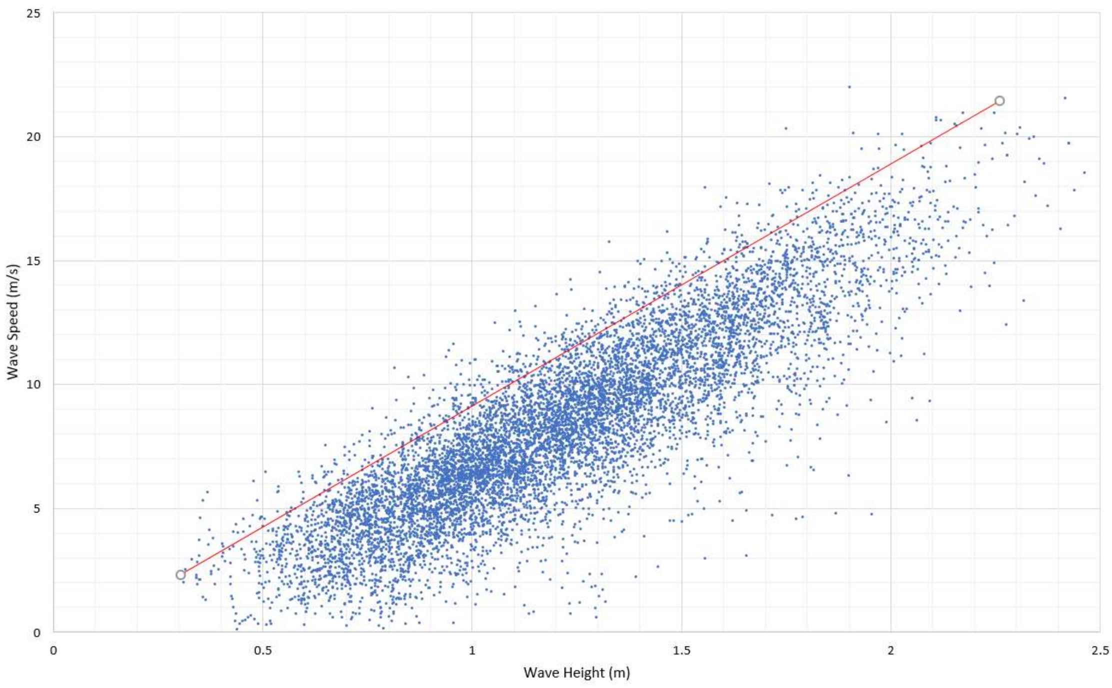

Figure 3-5: Hindcast dataset analysis between wind speed and wave height

3.1.4 Wave period

For each wave simulation, it is also important to derive an appropriate wave period. As with wind speed, the associated wave period for each simulation was derived by undertaking an analysis of the offshore wave height and wave period from the Hindcast dataset. The relationship between wave period and wave height was then defined.

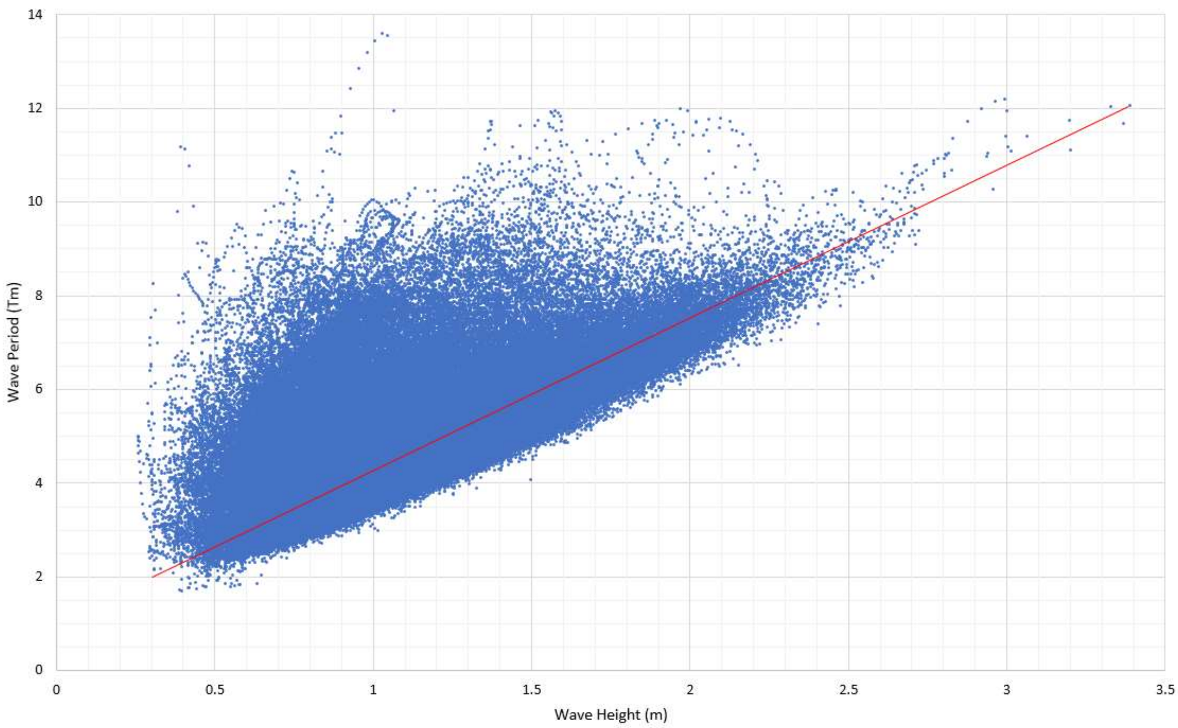

Figure 3-6: Hindcast dataset analysis between wave period and wave height

3.1.5 Joint probability analysis

A coastal flood risk event normally arises due to a combination of high water levels and large waves. The probability of occurrence of such events can be represented with return periods (e.g. a wave height that might be expected once in one hundred years). However, it is not appropriate to assume that a 1 in 100 year event will occur necessarily by the combination of a 100 year wave and a 100 year water level. Instead their joint probability of occurrence must be quantified.

The joint probability assessment investigations have been carried out in line with the Defra/EA Joint Probability: Dependence Mapping and Best Practice (2005). The joint probability investigates the relationship between water levels and wave heights. The technique used follows that of the 'desk based approach' which involves the application of published EA dependence values between water level and wave height. This section sets out the methodology used to estimate the joint probability events and presents the key findings.

Correlation coefficient estimates are available for the regions around Great Britain and these are expressed as coefficients from 0 (representing independence) to 1 (complete dependence). The correlation coefficient of 0.48 was applied for waves associated with a south and south east direction (70 – 210 degrees) and a correlation coefficient of 0.29 for waves associated with a south west direction (210 – 280 degrees) as illustrated in Figure 3-7 and Figure 3-8.

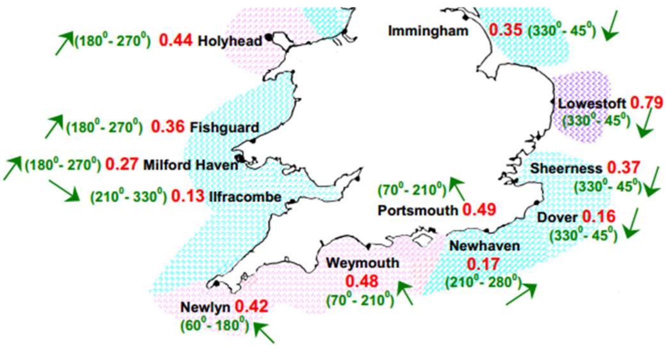

Figure 3-7: Correlation Coefficient (p, wave height and sea level), wave direction section in which dependence is higher.

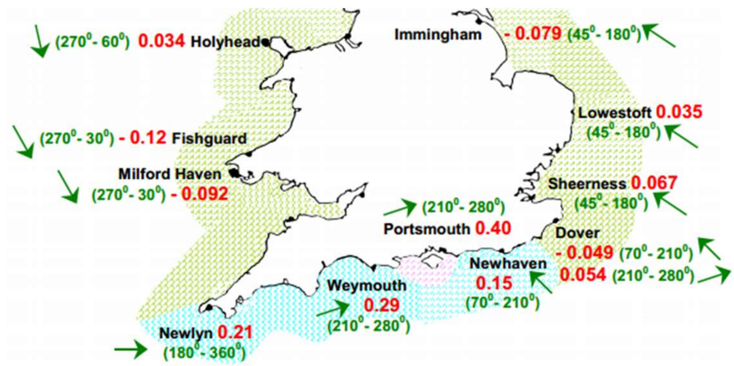

Figure 3-8: Correlation Coefficient (p, wave height and sea level), wave direction section in which dependence is lower

3.1.6 Offshore wave height and water level combinations

The present and future predicted offshore water levels and wave height combinations are presented below in Table 3-5 to Table 3-7.

Table 3-5: South east wave height and water level joint probability combination – 2017 (present day)

Water Level (ODN)	**1yr**	**10yr**	20yr	50yr	75yr	**100yr**	200yr	1000yr
2.35	3.49	4.22	4.39	4.60	4.67	4.74	4.90	5.28
2.41	3.49	4.22	4.39	4.60	4.67	4.74	4.90	5.28
2.47	3.19	4.21	4.39	4.60	4.67	4.74	4.90	5.28
2.55	2.94	4.01	4.27	4.56	4.66	4.74	4.90	5.28
2.61	2.69	3.79	4.08	4.40	4.52	4.61	4.81	5.28
2.69	2.36	3.47	3.80	4.17	4.31	4.41	4.61	5.10
2.75	2.12	3.22	3.56	3.97	4.13	4.23	4.46	4.94
2.81	-	2.98	3.31	3.75	3.92	4.04	4.29	4.77
2.91	-	2.65	2.98	3.42	3.62	3.75	4.05	4.58
2.98	-	2.40	2.73	3.17	3.37	3.51	3.83	4.43
3.05	-	-	2.49	2.93	3.12	3.26	3.59	4.26
3.15	-	-	-	2.60	2.79	2.93	3.27	4.00
3.18	-	-	-	-	2.65	2.79	3.12	3.88
3.21	-	-	-	-	-	2.69	3.02	3.79
3.29	-	-	-	-	-	-	2.77	3.54
3.50	-	-	-	-	-	-	-	2.97

Table 3-6: South wave height and water level joint probability combination – 2017 (present day)

Water Level (ODN)	**1yr**	**10yr**	20yr	50yr	75yr	**100yr**	200yr	1000yr
2.35	4.54	5.46	5.68	5.95	6.04	6.13	6.35	6.86
2.41	4.54	5.46	5.68	5.95	6.04	6.13	6.35	6.86
2.47	4.18	5.44	5.68	5.95	6.04	6.13	6.35	6.86
2.55	3.87	5.19	5.52	5.89	6.03	6.13	6.35	6.86
2.61	3.57	4.92	5.28	5.69	5.85	5.96	6.22	6.86
2.69	3.17	4.52	4.92	5.39	5.57	5.70	5.96	6.62
2.75	2.86	4.22	4.63	5.14	5.34	5.47	5.77	6.40
2.81	-	3.92	4.33	4.86	5.08	5.23	5.55	6.18
2.91	-	3.52	3.92	4.46	4.70	4.87	5.24	5.93
2.98	-	3.21	3.62	4.16	4.40	4.57	4.97	5.73
3.05	-	-	3.32	3.86	4.10	4.27	4.67	5.51
3.15	-	-	-	3.46	3.69	3.86	4.27	5.18
3.18	-	-	-	-	3.52	3.69	4.10	5.02
3.21	-	-	-	-	-	3.56	3.97	4.91
3.29	-	-	-	-	-	-	3.67	4.61
3.50	-	-	-	-	-	-	-	3.91

Table 3-7: South west wave height and water level joint probability combination – 2017 (present day)

Water Level (ODN)	**1yr**	**10yr**	20yr	50yr	75yr	**100yr**	200yr	1000yr
2.35	6.71	8.67	9.21	9.94	10.21	10.49	11.15	12.68
2.41	6.17	8.29	8.93	9.78	10.13	10.42	11.15	12.68
2.47	5.46	7.57	8.21	9.06	9.44	9.71	10.33	12.14
2.55	4.92	7.03	7.67	8.52	8.89	9.16	9.81	11.48
2.61	4.38	6.49	7.13	7.97	8.35	8.62	9.26	10.82
2.69	3.67	5.78	6.42	7.26	7.63	7.90	8.54	10.02
2.75	3.13	5.24	5.88	6.72	7.09	7.36	8.00	9.49
2.81	-	4.70	5.34	6.18	6.55	6.82	7.45	8.94
2.91	-	3.99	4.63	5.47	5.84	6.10	6.74	8.22
2.98	-	3.45	4.09	4.93	5.30	5.57	6.20	7.68
3.05	-	-	3.55	4.39	4.76	5.03	5.66	7.14
3.15	-	-	-	3.68	4.05	4.31	4.95	6.43
3.18	-	-	-	-	3.73	4.00	4.63	6.11
3.21	-	-	-	-	-	3.77	4.41	5.89
3.29	-	-	-	-	-	-	3.87	5.35
3.50	-	-	-	-	-	-	-	4.10

3.2 Wave transformation modelling

This section outlines the second stage of the numerical modelling process being wave transformation modelling. The wave transformation model SWAN (Simulating WAves Nearshore) was adopted to transform extreme offshore waves to a series of nearshore locations along the study frontage. The SWAN model software was developed by the Technical University of Delft and is a 3rd generation, state of the art, spectral wave transformation model, particularly suited for coastal wave modelling. The following sub-sections outline the approach taken to set up and SWAN model and derive the nearshore wave conditions.

3.2.1 Model domain

An existing SWAN model was made available by the Environment Agency for use in this project and Figure 3-9 presents the model domain. The SWAN model was developed and calibrated in 2015 as part of the State of the Nation Flood Risk Analysis project and details of this can be found in Appendix B – State of the Nation Flood Risk Analysis, Swan 2D Wave Calibration Report, JP9 Lyme Bay.

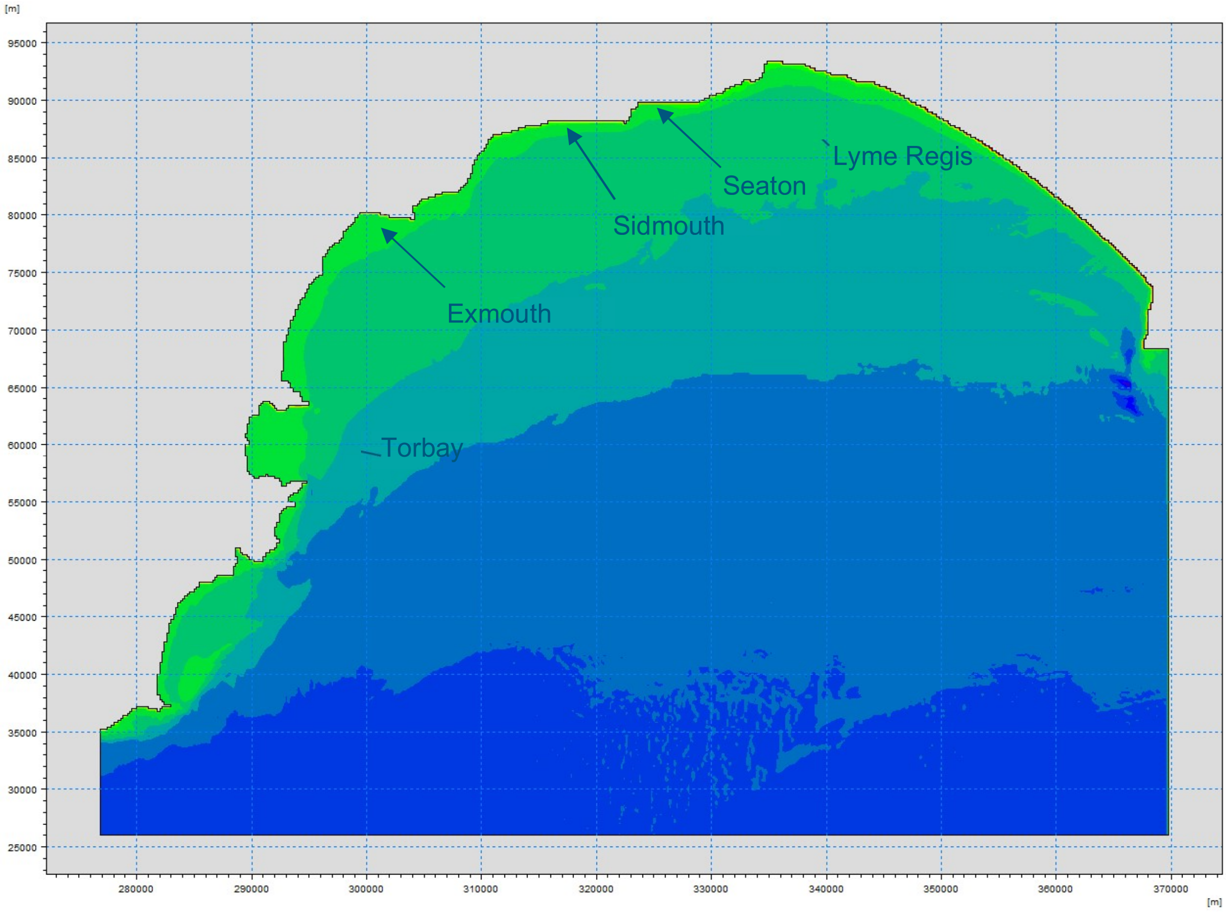

Figure 3-9: SWAN model domain

3.2.2 Model mesh development

The SWAN model provided has a computational grid resolution of 200 x 200 square metres and provided a simple representation of the existing coastline. To represent the study frontage in greater detail, two nested computation grids were created: one at 80 x 80 square metres and one at 10 x 10 square metres, as Figure 3-10 illustrates. Within these nested grids, the coastline was updated to include important influencing features such as the nearshore breakwaters. The grids were also updated to include the latest 2017 bathymetric survey to represent as accurately as possible the surface over which the waves propagate and interact. Figure 3-11 presents the extent of the 2017 bathymetric survey incorporated into the model.

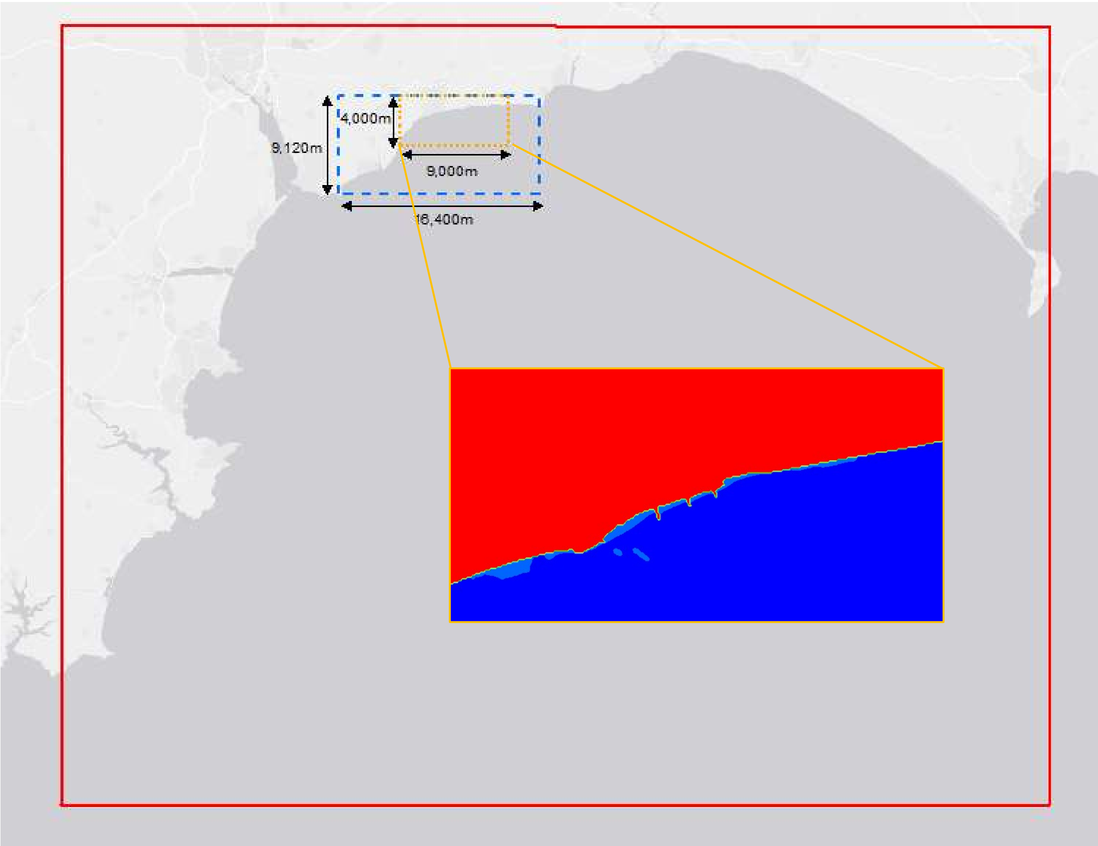

Figure 3-10: Swan model nested mesh

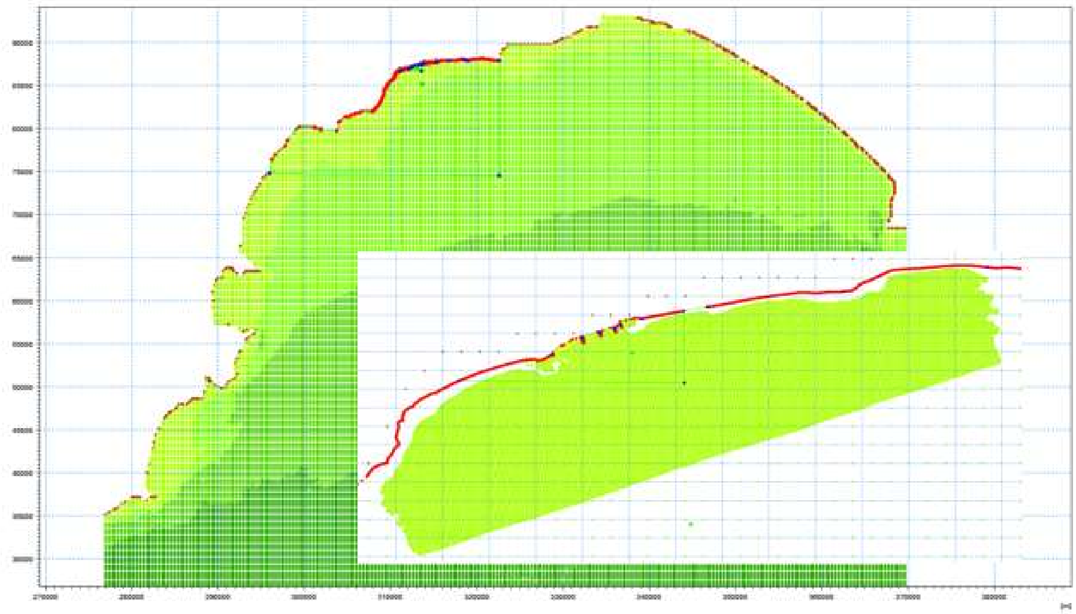

Figure 3-11: Bathymetric data coverages within the model.

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3.2.3 Wave Transformation model calibration and validation

The SWAN model was calibrated as part of the State of the Nation project. The general approach adopted for model calibration was to run the SWAN model for a number of model settings for a selection of peak events and selecting those settings that led to the overall lowest errors for subsequent application.

Although the model was calibrated and ready to use, it is always appropriate to undertake some validation runs to further ensure its suitability and demonstrate its performance. In order to validate the model, the Hindcast dataset was examined to identify major storm events for south westerly events. The selected storm events were simulated and the significant wave height, wave period and direction were extracted from the SWAN model at the location of the West Bay wave buoy. Results were then compared against the measured significant wave height from the West Bay wave buoy. The location of the wave buoy is presented in Figure 3-12. The measured data was downloaded from the Plymouth Coastal Observatory website. Table 3-8 presents the storm events and model results compared to the measured data and confirms that the model performs well.

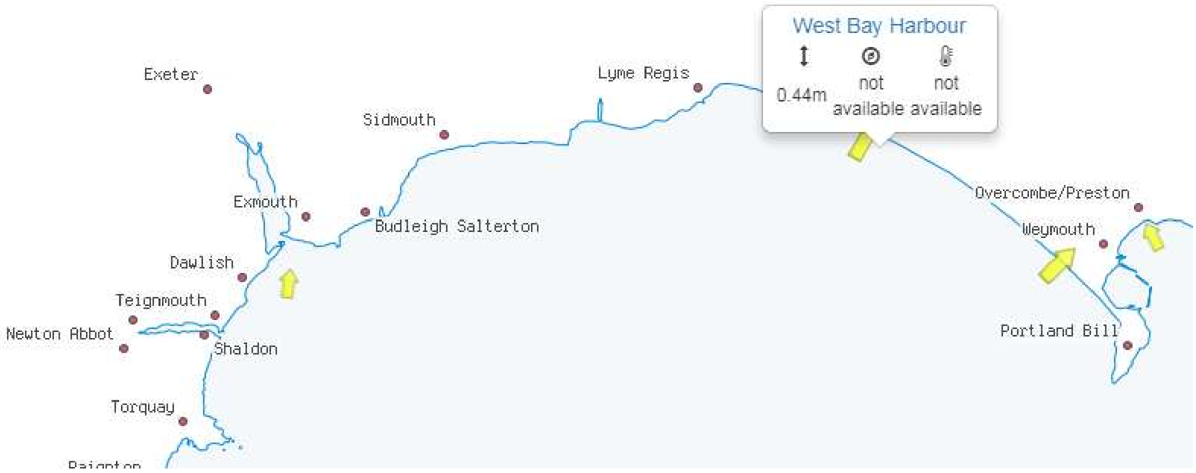

Figure 3-12: Location of West Bay wave buoy

Table 3-8: Model validation storm events

Date	Offshore			Inshore (modelled)			Inshore (measured)			Difference Hs
	Hs	**Tp**	**Dir**	**Hs**	**Tp**	**Dir**	**Hs**	**Tp**	**Dir**	**Modelled Measured**
30/03/2010	3.56	16.11	247	3.16	8.34	216	3.14	10.50	214	0.02
01/01/2014	5.42	16.08	217	4.17	9.67	201	4.51	10.00	212	-0.33
02/01/2016	6.07	13.75	213	4.96	13.74	198	4.98	14.3	208	-0.02

3.2.4 Modelled extreme events

The predominant wave direction for Sidmouth is south west, however storm events from the south and south east are also common. South and south easterly storms expose the River Sid training wall and East Beach cliffs and wave overtopping can occur along the training wall with wave propagating inland. To assess the impact of storms from all significant directions, wave transformation modelling was undertaken for both a southwest and a south-southeast event.

3.2.5 Nearshore extreme wave heights

The study area was divided into 8 frontages, as illustrated in Figure 3-13. Six frontages were located along the Sidmouth Town frontage and two located along the River Sid training wall. Nearshore wave heights were extracted along each frontage and these wave heights fed into the wave overtopping modelling discussion in the Section 6.3. The SWAN model was run for a 1 in 1, 10, 20, 50, 75, 100, 200 and 1000 year return period for all joint probability combinations. Table 3-9 presents an example of the modelled nearshore wave heights at 280m offshore. As discussed in Section 6.3.2, for each wave overtopping run nearshore wave heights were extracted at one wave length from the shoreline.

Figure 3-13: Sidmouth frontages

Table 3-9: Modelled nearshore wave heights for a 2017, 1 in 200 year return period south westerly event

Joint Probability Combination	Offshore Conditions				Nearshore Wave Height (Hs)						
	Wave Height (m)	**Wave Period (Tp)**	**Wind Speed (m/s)**	**Water Level (mODN)**	**F1**	**F2**	**F3**	**F4**	**F5**	**F6**	**F7**
1	11.15	14.14	25.89	2.41	4.11	4.09	4.29	4.42	4.59	4.16	4.11
2	10.33	13.66	24.84	2.47	4.09	4.07	4.27	4.40	4.56	4.14	4.09
3	9.81	13.34	24.15	2.55	4.10	4.08	4.27	4.40	4.56	4.14	4.10
4	9.26	13.00	23.41	2.61	4.09	4.07	4.25	4.38	4.54	4.14	4.09
5	8.54	12.53	22.40	2.69	4.06	4.03	4.21	4.34	4.49	4.11	4.06
6	8.00	12.17	21.60	2.75	4.05	4.00	4.17	4.30	4.44	4.09	4.05
7	7.45	11.79	20.79	2.81	4.01	3.96	4.11	4.24	4.37	4.06	4.01
8	6.74	11.27	19.66	2.91	3.92	3.87	4.01	4.13	4.24	3.97	3.92
9	6.20	10.86	18.77	2.98	3.83	3.79	3.89	3.99	4.09	3.87	3.83
10	5.66	10.44	17.85	3.05	3.70	3.63	3.67	3.75	3.83	3.69	3.70

11	4.95	9.84	16.55	3.15	3.43	3.30	3.28	3.33	3.40	3.33	3.43
12	4.63	9.56	15.94	3.18	3.25	3.10	3.07	3.11	3.17	3.12	3.25
13	4.41	9.35	15.50	3.21	3.11	2.95	2.92	2.97	3.02	2.97	3.11
14	3.87	8.84	14.39	3.29	2.70	2.56	2.53	2.56	2.60	2.57	2.70

In the absence of inshore wave data nor comprehensive footage of a well-defined storm event, it was not possible to calibrate inshore waves. However, as explained above, the transformation model used was develop by the Environment Agency as part of the State of the Nation Flood Risk Analysis project in 2015 and was calibrated and validated during this process. Therefore, there was confidence in the transformation processes used to calculate the correspondent inshore waves from the calibrated offshore conditions.

3.2.6 Wave overtopping modelling

The next stage in the process was to use the outputs from the wave modelling to undertake an overtopping analysis to provide inputs into a 2D TuFLOW flood propagation model. TuFLOW was then used to assess the present and future flood risk. The overtopping analysis included an assessment of the following:

- Overtopping risks at present and in the future along the Sidmouth Town frontage, assuming a low beach level.
- Overtopping risks in the future along the River Sid western wall, under a scenario of further erosion of East Cliff.

3.2.7 Wave overtopping mechanisms

Wave overtopping which runs up the face of a structure and over the crest in (relatively) coherent water mass is often terms ‘Green Water’ overtopping. ‘White Water’ overtopping tends to occur when waves break seaward of the defence structure or break onto its seaward face, producing non-continuous overtopping, and/or significant volumes of spray. Overtopping spray can be carried over a structure under its own momentum or assisted by an onshore wind. Both types of wave overtopping occur at Sidmouth as illustrated in Figure 3-14. At present, no modelling exists to confidently quantify White Water overtopping caused by onshore wind. Although this is a significant component of the overall overtopping occurring at Sidmouth, it cannot be accounted for when modelling flood risk.

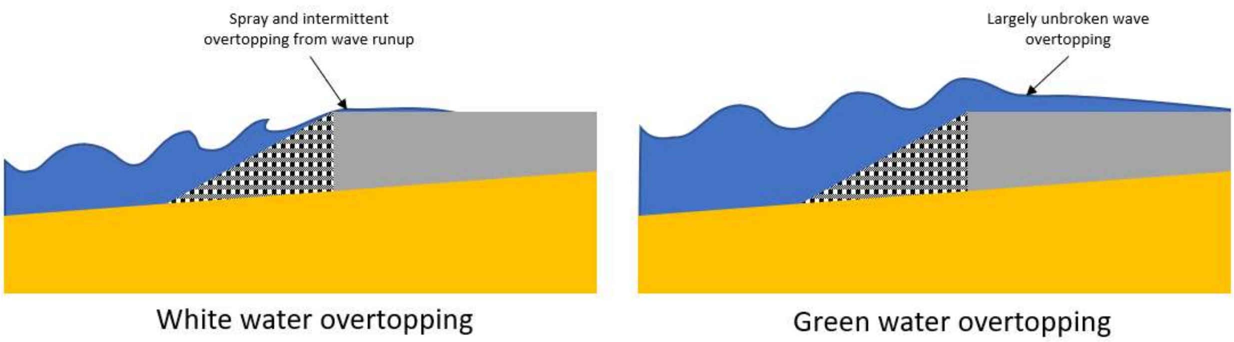

Figure 3-14: Wave overtopping

3.2.8 Wave overtopping mechanisms

Overtopping assessments are used to estimate the discharge of water that can pass over defences under wave action via Green Water overtopping.

The software used to undertake the wave overtopping assessment was AMAZON. AMAZON (Hu et al, 2000) is a high-resolution wave run-up and overtopping model, which was developed by Royal HaskoningDHV and the Manchester Metropolitan University. It is a finite volume model of 2nd order upwind scheme and can use irregular, structured and boundary-fitted mesh. The model solves the nonlinear shallow water equations of wave propagation and run-up providing time series changes in water levels and depth averaged velocities using random waves as a boundary condition. It accounts for wave diffraction, refraction, reflection, wave breaking and other shallow water effects and measures the overtopping known as Green Water overtopping. Figure 3-15 presents an example of the AMAZON model.

Although AMAZON is an ‘in house’ Royal HaskoningDHV tool, it has been published and peer reviewed, and is an established means of quantifying overtopping rates. It has, for example, been used for many Environment Agency projects. A key strength of this modelling tool is that it can be used to explicitly represent the observed profile of each shore (as opposed to the process of parameterisation required by empirical models such as EurOtop).

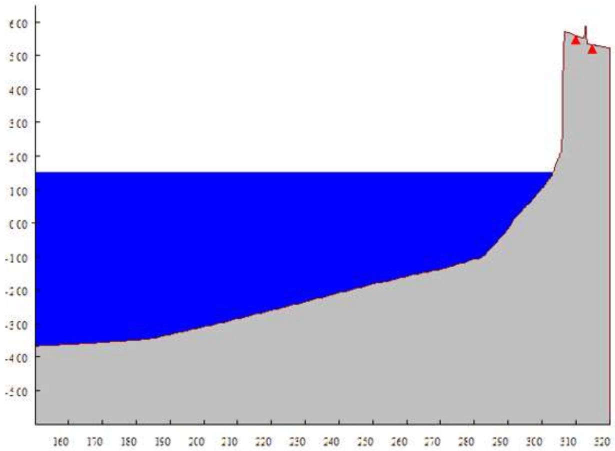

Figure 3-15: Example of AMAZON model run for frontage 6

3.2.9 Wave overtopping validation and calibration

In the absence of measured overtopping data, including overtopping volumes during a defined event, calibration of the overtopping model was not possible. However, the overtopping model was validated by comparing modelled overtopping rates to available videos and photos, especially of the 2014 storm event. It was determined that the overtopping rates appeared to align with the overtopping evidence available.

3.2.10 Model set up

As explained above, the Sidmouth frontage was divided into eight frontages. The sub-frontages allowed accurate representation of the different beach profiles and defence sections along the sea front. Available beach topographic survey profiles were obtained from the Plymouth Coastal Observatory (PCO) and analysed to select appropriate profiles for each sub-frontage. Surveys were not available representing the River Sid western wall and the training wall so these were recreated from LiDAR. The profiles were analysed and an appropriate average one was chosen to use for the present day overtopping modelling. For climate change, it was assumed that beach levels would slowly drop under a ‘Do Nothing’ scenario by 0.5m by 2037, a further 0.5m by 2067 and by 2117 beach levels will be down to bed level (i.e. no shingle remaining). This analysis was also informed by the beach profile analysis undertaken during the development of the BMP. Figure 3-16 illustrates the frontages and profiles used in the overtopping assessment and Figure 3-17 presents an example of the beach profiles modelled for Frontage 4 under a scenario of a gradually lowering beach.

Figure 3-16: Sidmouth sub-frontages and profiles

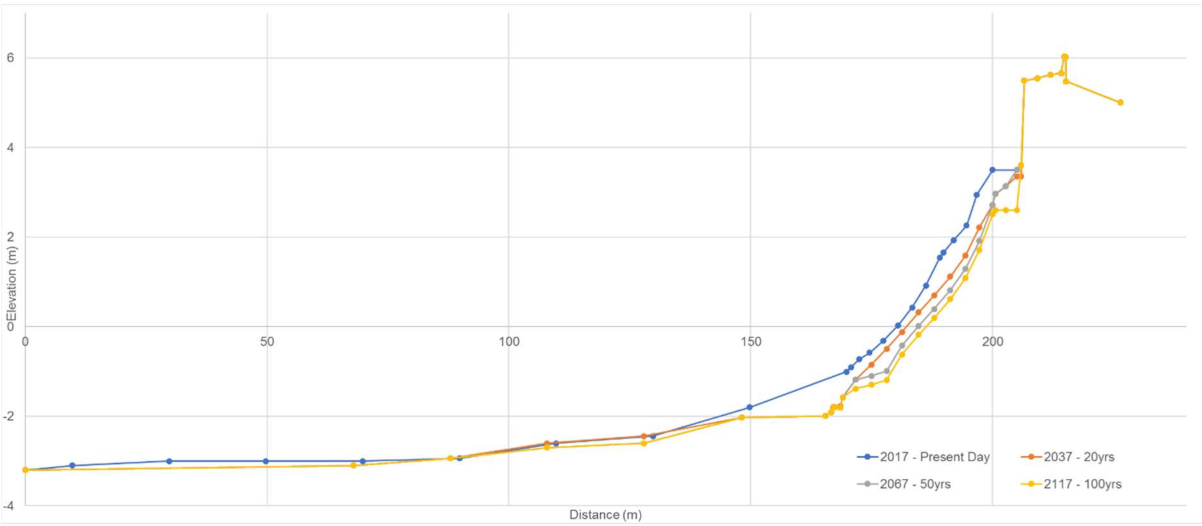

Figure 3-17: Frontage 4, Profile 6a1449 beach profiles used in the wave overtopping assessment.

For each model simulation, wave heights were extracted in front of each profile location. The wave heights were extracted at different offshore distances which match the wave lengths being simulated in AMAZON (i.e. if the wave length was 200m, the nearshore wave height would be extracted 200m from the shoreline). When using AMAZON, this method of varying locations for extracting wave heights is preferred to obtaining one individual wave height at a single point for all runs. This ensures that a more precise wave height is used for the representative wave length at each location. Therefore, a more accurate wave run up is calculated without under or over estimating wave overtopping.

For the present day runs, all joint probability combinations were modelled for a number of return periods and sub-frontages to identify the combination which results in the highest average wave overtopping amount. Table 3-10 presents the results for a present day 1 in 100 year return period along frontages 4 and 5 over the existing splash wall. For each return period, the analysis identified the worst case joint probability combination to use in the wave overtopping assessment. It also illustrated that wave heights rather than water levels have greater influence on wave overtopping.

Table 3-10: Wave overtopping assessment along Frontage 4 and 5 – South west 210 2017 1 in 100yr return period

Joint Probability Combination	**Wave Height (m)**	**Wave Period (Tp)**	Wind Speed (m/s)	Water Level (mODN)	**Wave Overtopping F4 (l/s/m)**	**Wave Overtopping F5 (l/s/m)**
1	10.49	13.75	25.05	2.35	2.3982	1.8797
2	10.42	13.71	24.96	2.41	4.2585	2.2039
3	9.71	13.28	24.02	2.47	2.8018	1.2586
4	9.16	12.94	23.27	2.55	1.7732	0.689
5	8.62	12.58	22.51	2.61	1.0358	0.4503
6	7.90	12.10	21.46	2.69	0.706	0.2304
7	7.36	11.72	20.64	2.75	0.4998	0.1193
8	6.82	11.33	19.79	2.81	0.3509	0.0501
9	6.10	10.79	18.61	2.91	0.3331	0.0742
10	5.57	10.36	17.67	2.98	0.3601	0.0395
11	5.03	9.90	16.69	3.05	0.3547	0.0078
12	4.31	9.26	15.31	3.15	-	-
13	4.00	8.97	14.66	3.18	-	-
14	3.77	8.75	14.18	3.21	-	-

For each profile, the model was simulated for 1000 random irregular waves and the average wave overtopping discharge rate (litres per second per metre) was calculated. Wave overtopping was calculated by considering overtopping rates at different sea levels of 0.25m intervals to represent the rise and fall of a tide in accordance with national guidance[6]. Like physical models, phase resolving overtopping modelling requires simulations of a sufficient number of waves to get a convergent mean overtopping rate. For each profile, wave overtopping was calculated along the esplanade and behind the splash wall. Locations where overtopping occurred where then used in TuFLOW model to propagate flooding inland.

3.3 Flood propagation modelling

The fourth stage in the modelling process was to undertake flood inundation modelling. This involves simulating how overtopped water propagates through Sidmouth and identifies the areas at risk of flooding with the current coastal defences in place now and over the next 100 years.

An existing TuFLOW flood inundation model was developed during the BMP and this was obtained from EDDC. TuFLOW (Two-dimensional Unsteady FLOW) is used for simulating depth-averaging two dimensional flows such as tides and floods. It solves the shallow water equations used for modelling 'long' waves (i.e. where the wavelength is significantly longer than the water depth) such as floods, tides and storm surges. The TuFLOW model was reviewed and updated where necessary and details of this are discussed below.

[6] *Computational modelling to assess flood and coastal risk. Environment Agency. Operational Instruction 379_05. 27.10.10.*

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3.3.1 Model set up

The model domain covered the low lying land across the town of Sidmouth and tied into high ground to ensure all potentially vulnerable areas were included within the model, as presented in Figure 3-18. The LiDAR data used for the model terrain was supplied by East Devon District Council at 1m resolution for the entire study area. The LiDAR tiles were supplied as ASCII files which were combined to produce a complete topographic gird of the study area. The surface of this grid was interrogated to assign elevations to the grid points within the TuFLOW model to define the surface elevation of the model.

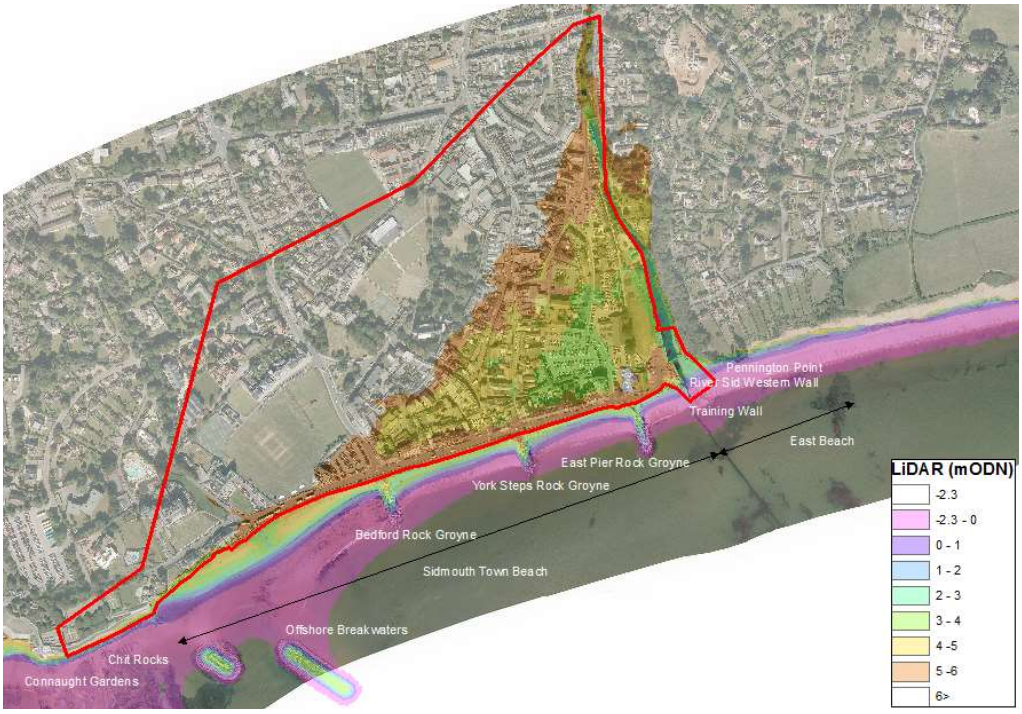

Figure 3-18: Sidmouth TuFLOW Model Domain

A 4m grid resolution was applied within the model to capture important flow routes such as roads and to represent important step changes in topography.

Surface roughness represents the effect within the floodplain which is places upon a wetter region to retain the effect of flow, bed and bank materials and structures. OS MasterMap data is a GIS compatible layer that delineates a suite of natural and manmade features on the surface which also includes building and roads. Each feature is given a unique numerical code. Through adopting the assigned OS MasterMap codes, a Manning's Roughness Coefficient values can be specified accordingly. The model roughness values have been derived on existing model data and best engineering practice. The model roughness values are provided in Table 3-11. In line with EA Operational Instruction 379_05[7], buildings were raised by 300mm with a high roughness to represent 'stubby buildings'.

Table 3-11: Model roughness values

Land type	Roughness Coefficient	Land use type	Roughness Coefficient
Grass	0.04	Water inland	0.035
Dense trees	0.06	Natural environment (coniferous/non coniferous trees)heavy woodland and forest	0.1
Fence shrubs	0.05	Roads tracks and paths manmade	0.02

[7] *Computational modelling to assess flood and coastal risk. Operational Instruction 379_05. 27/10/10.*

Gravel road	0.035	Roads tracks and paths tarmac or dirt tracks	0.25
Footpaths and paved areas	0.025	rail	0.05
Hard surface standing areas work yards	0.05	Roads tracks and paths tarmac	0.02
Culver Roughness	0.015	Roads tracks and paths (roadside) pavement	0.02
Channel	0.03	Structures road side	0.03
Flood Plain	0.05	Structures generally on top of buildings	0.5
default	0.04	Water foreshore	0.04
building	0.5	Water tidal water	0.035
General surface residential yards	0.04	Land (unclassified) industrial yards car parks	0.035
General surface step	0.025	slope	0.04
General surface grass parkland	0.03	pylon	0.04
Building glass house	0.5	Land assume grass	0.04
Land heritage and antiquities	0.5	Cliff assumed step	0.04

LiDAR was used to derive the surface elevation within the model. For the more intricate features such as the setback splash wall along the promenade and the walls around the River Sid training wall, topographic surveys and drawings were used to derive the levels. To represent these within the model 'THIN' z-lines have been used as illustrated in Figure 3-19.

Figure 3-19: Defence Z-Lines within the TuFLOW model

3.3.2 River Sid breach

It is outlined in the BMP that East Beach cliff will continue to retreat over the next 100 years. The BMP noted that the lowest estimate of retreat would be 20.9m and the upper estimate of retreat would be 30.9m. These rates were updated to use more recent data, as cliff erosion appears to have accelerated in the past decade. With the revised rate of -2.1m / year between year 0 to 20 years and 0.6m / year between year 20 and 100, the cliff is estimated to retreat by 92.5m over 100 years. The BMP also provides estimates on the residual life of the training arm and wall along the River Sid.

The retreat rates and residual life information was used to make a judgement as the likely timescales of cliff retreat and defence failure along the River Sid under a ‘Do Nothing’ scenario. This then fed into calculating overtopping in Amazon for the River Sid wall under a ‘Do Nothing’ scenario.

Figure 3-20: Do Nothing cliff retreat and defence failure over 100 years.

3.3.3 Model inputs

A tide curve was derived for each simulation to represent the rise and fall of the tide. This required a number of steps as follows:

1. Selecting a time series of astronomical tidal elevations for a Highest Astronomical Tidal event at the nearest tidal station.
2. Selecting a surge profile shape. The surge profile selected was obtained from the EA CFBD and the Devonport profile was selected.
3. Selecting of extreme water levels.
4. Deriving the design tidal graphs by scaling the surge shape so that when combined with the tide data, the peak of the tide level equalled the required extreme water level for each simulation. The water level was then adjusted for each return period being simulated.
5. In line with the EA CFBD, the surge profile was applied and the model ran for an appropriate storm duration.

Wave overtopping calculated by AMAZON for each event was used as input into the TuFLOW model. As discussed, wave overtopping was calculated per metre length of each frontage and with varying water levels. The overtopping rate was multiplied by the length of each frontage and then divided across a number of inflow points, as illustrated in Figure 3-21. This rate was then applied to the tide curve in order to simulate wave overtopping through the design tide curve using a Flow vs Time (ST) boundary, as illustrated in Figure 3-22.

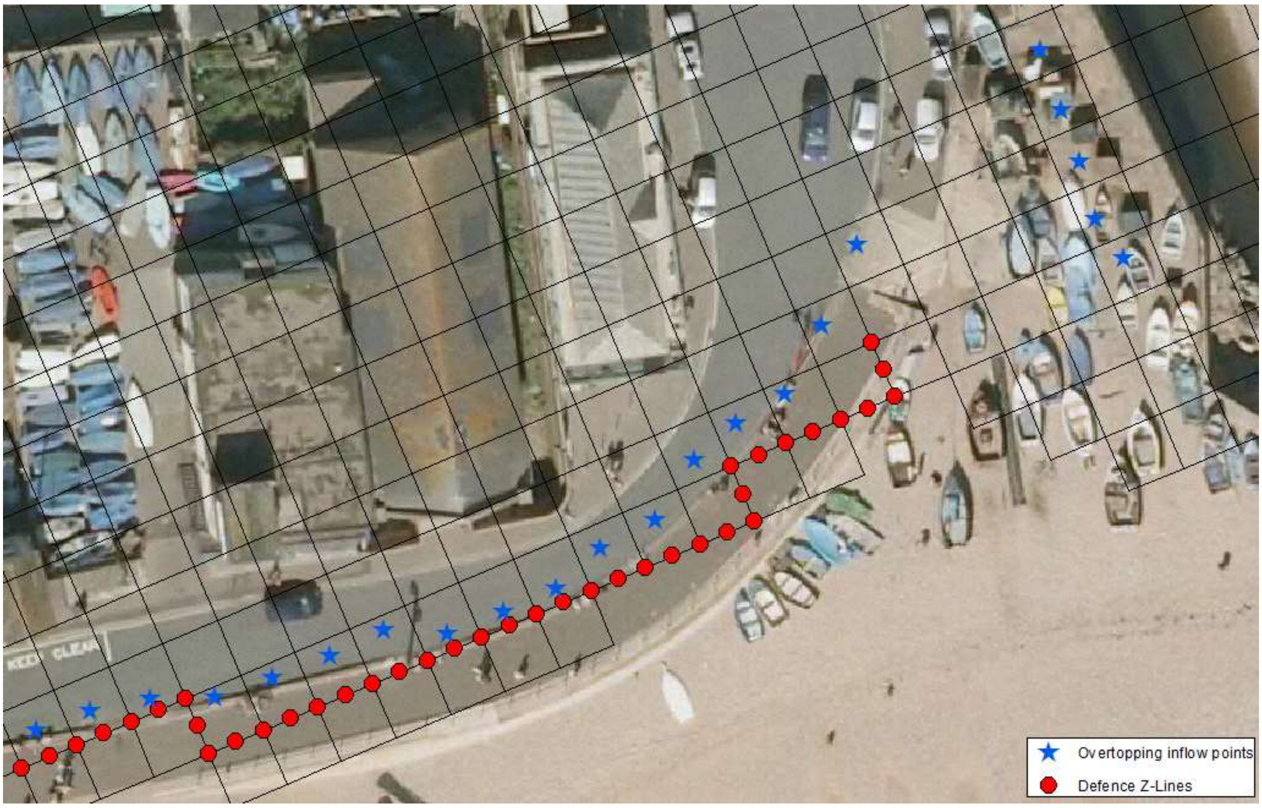

Figure 3-21: Wave overtopping inflow.

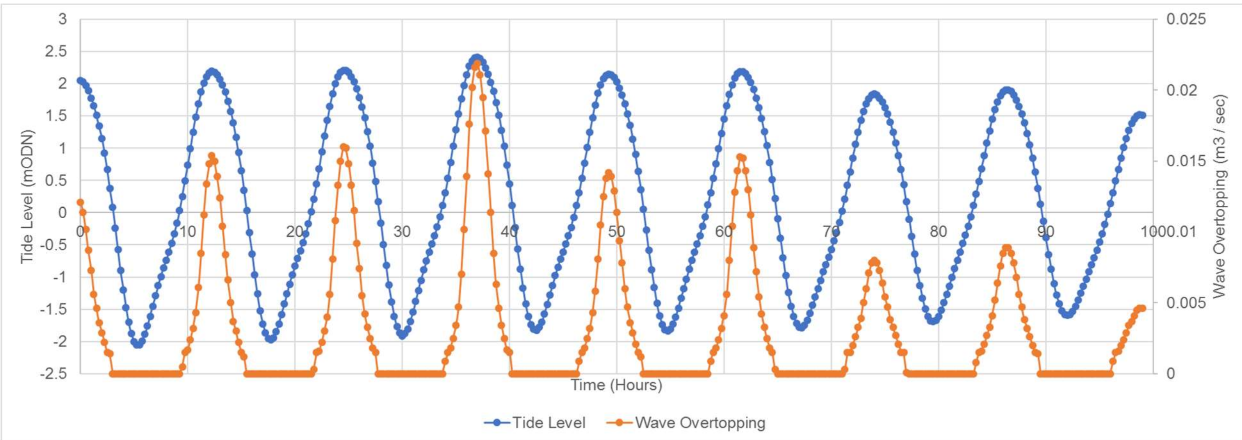

Figure 3-22: Tide vs overtopping

3.3.4 Model duration

Wave heights measured from the West Bay wave buoy were downloaded for recent storm from the Plymouth Coastal Observatory to identify a typical storm duration. As illustrated in Figure 3-23, a typical storm event ranges from 12 – 48 hours. It was agreed with EDDC and the Environment Agency that a 48 hours storm duration would be used to represent extreme storm events along Sidmouth.

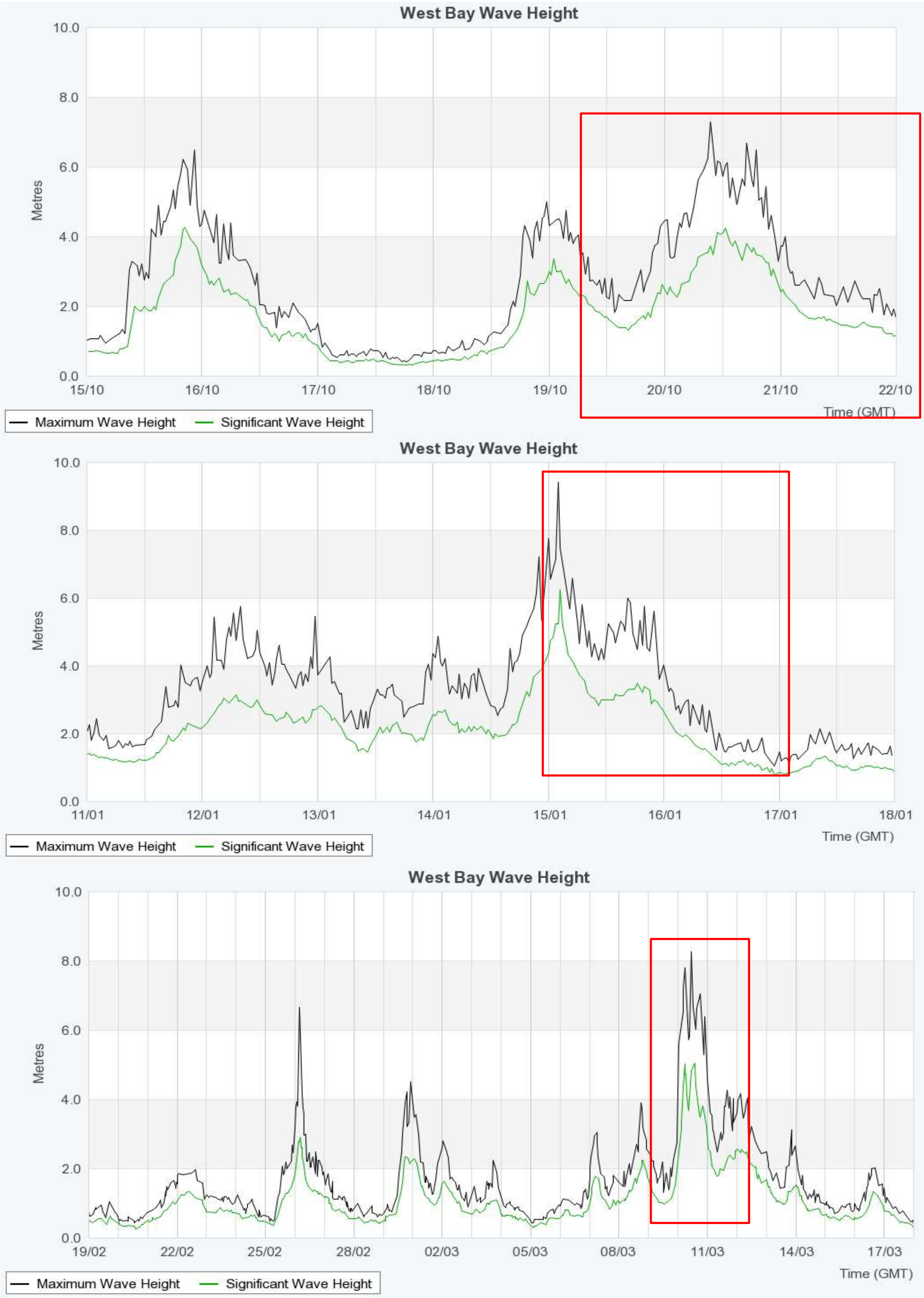

Figure 3-23: Measured wave heights at West Bay wave buoy during recent storm events. (A: Storm Brian, 2017, B: Winter Storms January / February 2015, C: February / March 2008).

3.3.5 Assumptions

Assumptions were necessary in order to implement the model sufficiently without excessive model developments or computational run times. The following assumptions were made:

- The profile used for each sub-frontage represented the whole length of the frontage and beach levels were assumed the same across the whole frontage.
- The overtopping rates were based on average rates over 1000 waves. The average overtopping rates from AMAZON are applied to the boundary of the TuFLOW model in a uniform manner.
- There is no allowance for surface water road drainage that may drain any overtopped water back out to sea. This represents a 'worst case scenario'.
- Wave heights remained the same for each over topping run.
- Walls and other manmade obstructions other than coastal defences have not been included in the model. However, buildings were raised by 300mm.

3.3.6 Model outputs

For each model simulation depth, level, velocity and hazard grids have been produced. For flood hazard the following equation was used:

HR = d x (v + 0.5) + DF (d = depth of flooding, v = velocity and DF = debris factor).

4 Stage 2

4.1 Sediment transport modelling

The sediment transport numerical modelling package, LITLINE, developed by DHI was utilised to simulate the shoreline evolution and to assess the beach management options. LITLINE is a 1D plan-shape model that simulates the development of a shingle beach contour under a series of coastal conditions (wave height, water level and wave period).

4.1.1 LITLINE model

LITPACK is an integrated modelling system developed by DHI which includes modules for the calculation of littoral drift / sediment transport across a profile, and the coastal evolution. This concerns the LITDRIFT and LITLINE modules.

LITDRIFT calculates the non-cohesive sediment transport across a user defined profile, using the vertical diffusion equation on a horizontal grid, allowing for breaking and non-breaking waves and currents. This can include different 1-d wave propagation theories to transform the waves from the nearshore to the element on the profile grid. These theories can account for a number of different effects to estimate the net transport which include:

- Wave asymmetry
- Lagrangian drift
- Wave streaming
- Undertow on the profile
- Bed ripples,
- Different bed material
- Wave decay due to breaking
- Wave set-up
- Variation in sediment size corresponding to a profile.

The equations are resolved in the longshore and cross-shore momentum balance equation to provide a time-varying and time averaged profile of eddy-viscosity, concentration, velocities, bed and suspended load both gross and net updrift and downdrift. An example of longshore sediment transport distribution can be seen in Figure 4-1.

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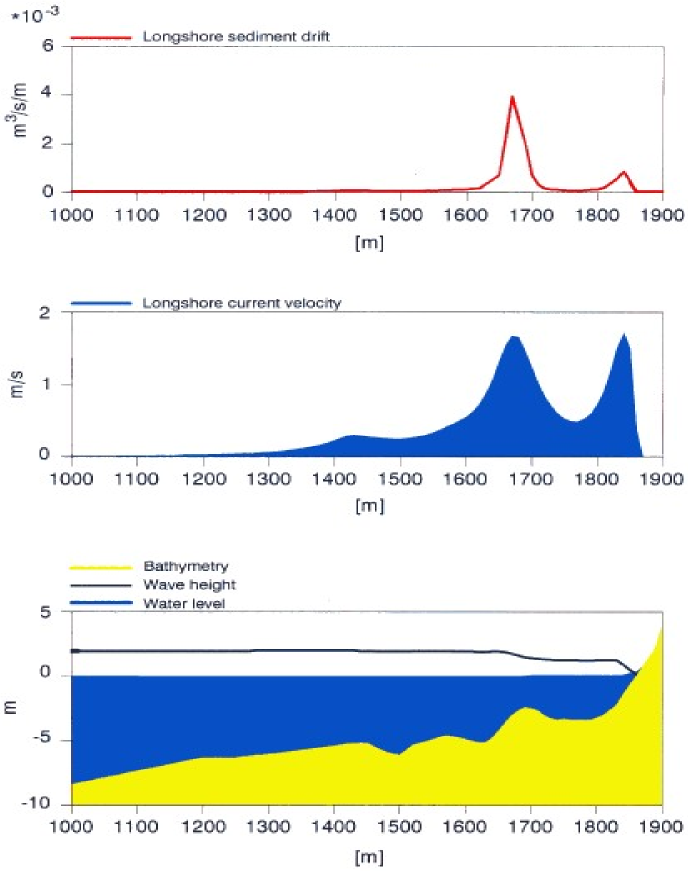

Figure 4-1: Illustration of the modelled long-shore drift distribution.

LITLINE uses the LITDRIFT calculation of sediment drift at each profile 'cell' along the coastline, calculating the shoreline change at each time-step and evolution over a period, by solving a continuity equation for the sediment in the littoral zone.

$$\frac{\partial y}{\partial t} = \frac{1}{D}\left(\frac{\partial Q}{\partial x} + q_s + q_0\right)$$

Where $\delta y/\delta t$ is the change in position of a point on the shoreline in a given time-step, and D is the active height/depth of beach profile, and $\delta Q/\delta x$ is the change in sediment transport in a profile slice and q_s and q_o are sediment source and sink parameters. The sediment transport rate Q is shown with respect to the profile in Figure 4-2. The equation above is then considered for a cell, calculating the changes over time to estimate the amount of shoreline movement δy, see Figure 4-3.

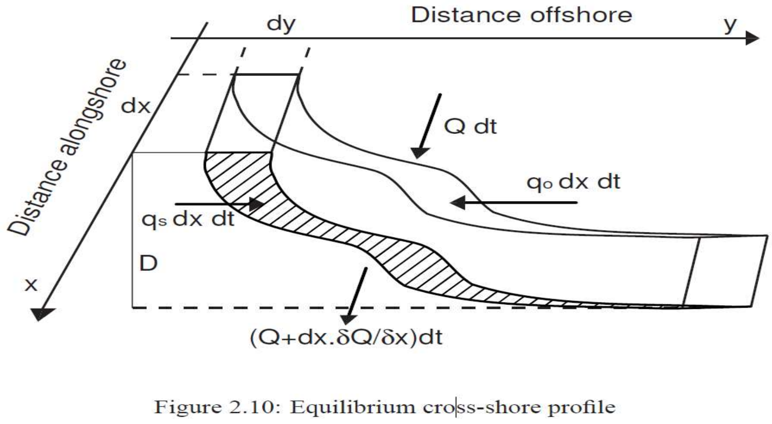

Figure 4-2: Illustration of one dimensional model cell

The model is setup as a series of slices, combined to represent the coastline. The input of sediment from one cell into another is calculated using the sediment transport equations, shown in Figure 4-3. In the case of this study, profile sections were established at 10m intervals over a shoreline length of 4200m.

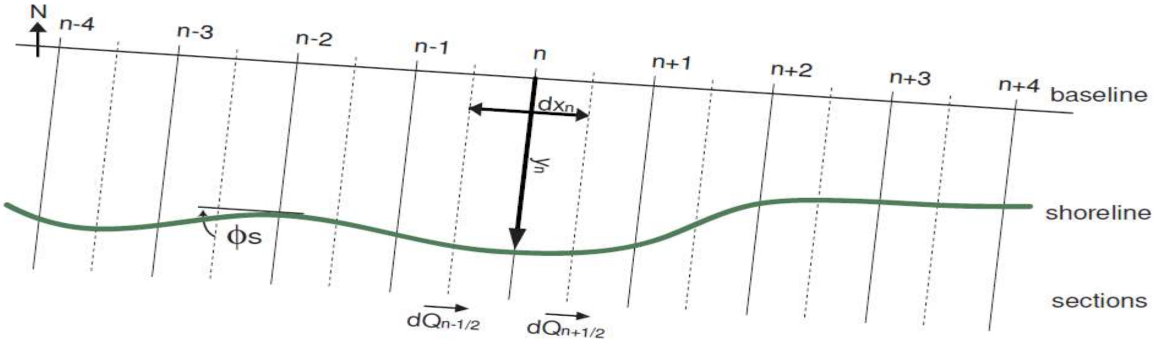

Figure 4-3: Illustration of coastline update method in one dimensional model.

Limitations:

- The central difference method of resolving the mass balance causes instabillity with the more oblique waves to the shore.
- The profile is fixed over time so that the short term profile change between different wave conditions is not represented.
- There is no calulation of on- or offshore transport so that offshore losses are not calculated within the model.

Critical inputs to the modelling are therefore:

- Overall topography and bathymetry.
- Sediment grading.
- Water levels.
- Wave climate (time series), and in relation to this,
- Coastline orientation.

4.1.2 Model set up

The first stage of the model set up was to establish simulations which match the current conditions of the beach. For this, based on current observations, it has been assumed that the conditions on the beach are in a relatively stable dynamic equilibrium (i.e. there is no significant progressive losses of sediment from the system). Therefore, whilst there may be periodic movement of shingle within the system and a change in the beach profiles, notably following a storm event, over time there is no dramatic erosion of the beach and during calmer conditions beach profiles recover to an extent.

The LITLINE model represents a single type of material. Whilst Sidmouth contained both sand and shingle as illustrated in Figure 4-4, the shingle sits on the steep upper beach and the sand sits at a shallower gradient in deeper waters. As the shingle is more effective at absorbing wave energy and thus at protecting the shoreline, the model was set up to only represent shingle.

Figure 4-4: Sidmouth Beach.

Notable model settings are summarised below:

- The model was driven by the outputs from the Met Office WAM4 hind cast model. The model was simulated using 36 years of data starting in 1980 and running in hourly time steps.
- Tidal currents were not used in the model as they are considered to be small at Sidmouth. The dominant sediment transport mechanism is via wave energy.
- The model was simulated using a constant 2m AOD water level.
- A single beach profile was taken at the Jacob's Ladder beach surveyed as part of the EA regional monitoring programme.
- The MHWS line was taken to be the baseline contour line.
- The model was set up so that the overall shoreline from north to the offshore normal is at an angle of 160 degrees.
- A single groyne is included at the far western boundary of the model (at Jacob's Ladder) to represent the cliff and the boundary of the sediment cell.

- The existing groynes, breakwaters, revetment, sea wall and training wall were included in the model.
- No sediment sources or sinks were used in the simulations.

4.1.3 Model validation

4.1.3.1 Baseline model

The aerial photograph and the initial starting position are shown in Figure 4-5. These are annotated with a few key positions for reference (refer to figure below for location of bullet points).

a) River Sid step back behind the training wall
b) Sidmouth Town rock groynes
c) Shingle beach behind the offshore breakwaters
d) Connaught Gardens headland.

Figure 4-5: Extent of model

Figure 4-6**Error! Reference source not found.** presents the baseline model:

- The horizontal axis represents the chainage in meters along the coastline.
- The right vertical axis is cumulative drift rate in m^3 over the duration of the simulation and relates to the upper pink dotted line. This is the net amount of sediment that crosses a profile over time. A negative drift indicates a sediment movement to the east and a positive drift to the west.
- The left vertical axis is the offset from the beach contour line in meters and relates to the lower bright pink line on the graph.

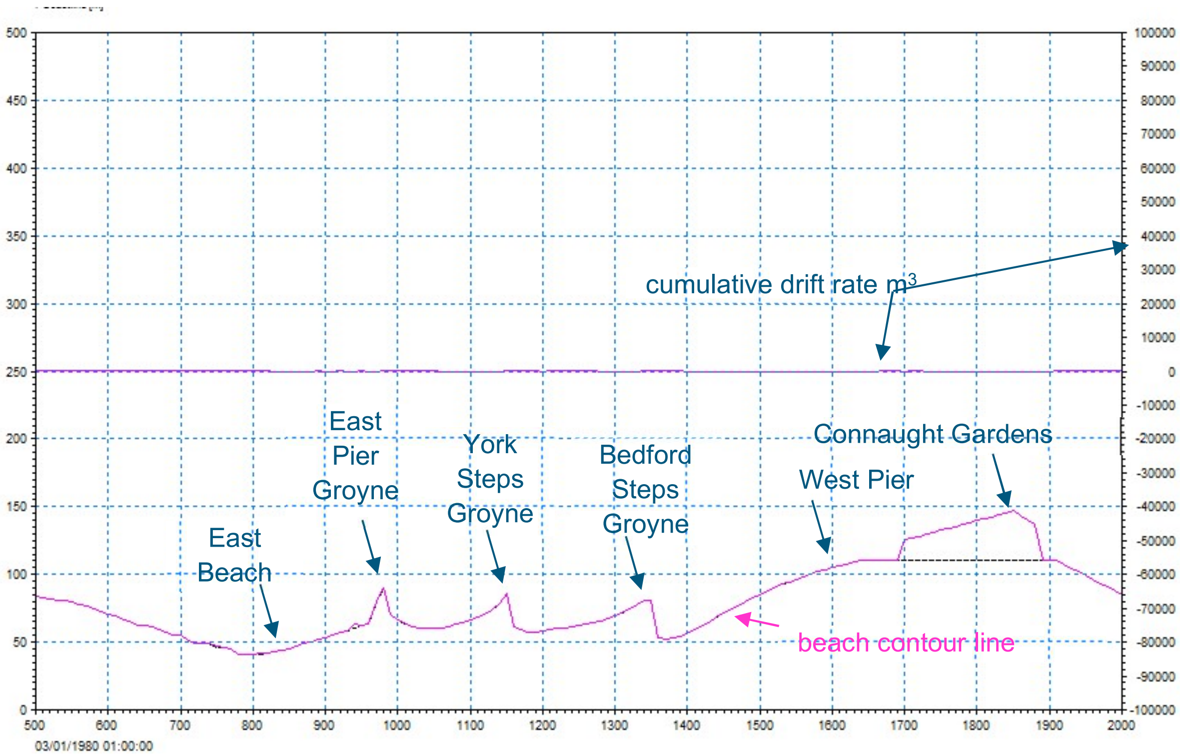

Figure 4-6: Baseline model starting position

Figure 4-7 presents the baseline evolution of the coastline after 5 years of simulations. The lower pink beach contour line represents the coastline after 5 years of simulations and the dotted black line represents the starting position of the simulation. The upper pink line represents the cumulative drift rate.

The initial simulation matches current observations providing confidence in the results. In particular (refer to figure below for location of bullet points):

a) There is a build-up of material on the west of the training wall and a cut back of material along East Beach.
b) The beach appears cut back at the seawall at Bedford Steps. This shows that the shingle profile is depleted and that the seawall is exposed.
c) The beach between East Pier Groyne and York Steps Groyne has accreted, suggesting material can bypass the groynes.
d) The influence of the offshore structures can be seen where the beach has accreted with negligible drift rates.

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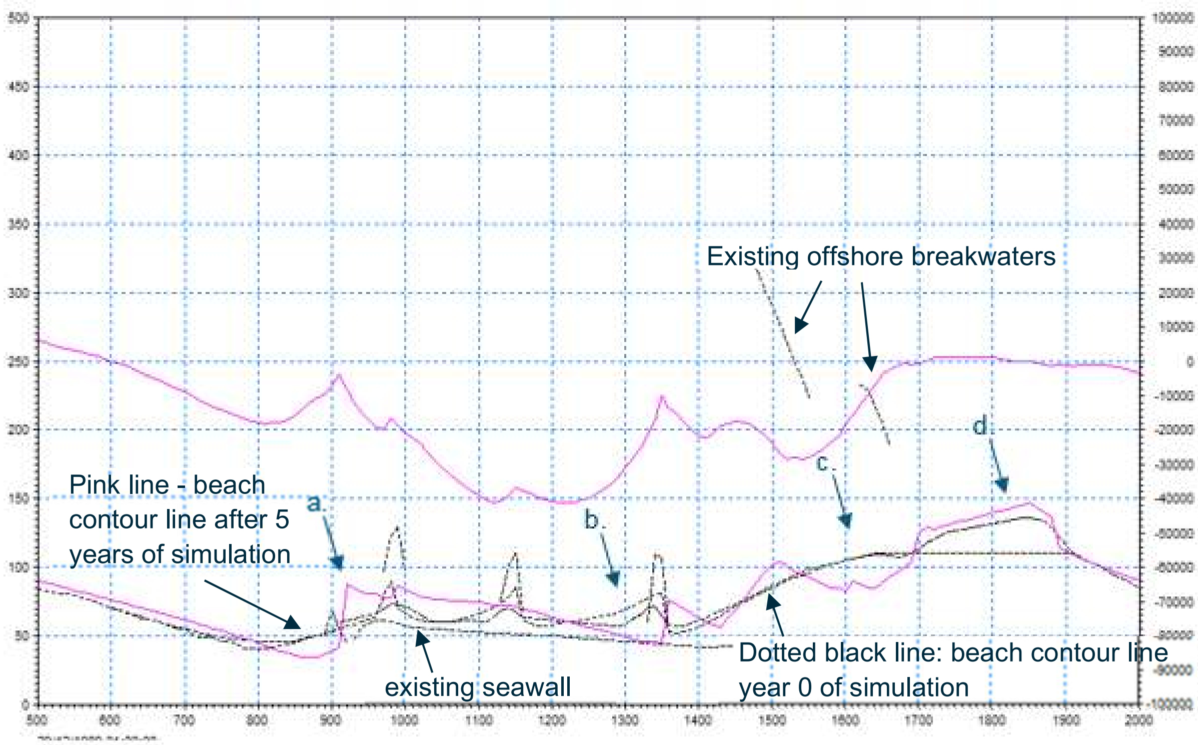

Figure 4-7: Baseline coastal evolution after 5 years.

4.1.3.2 Post 90s scheme construction

The second model scenario was to model the post 1990s scheme construction following nourishment of the frontage. This scenario was important as it provided a validation scenario that could be compared with the observation to date. The following assumptions have been made based on consultation with East Devon District Council and the Steering Group members.

- Based on assessment of the annual monitoring surveys and LiDAR surveys undertaken since 2000 there has been no significant accretion or loss of sediment along Sidmouth, indicating a relatively close cell system. However, shingle is known to move inside the cell during storm events.
- The initial beach nourishment was placed as part of the 1990s flood defence scheme and no significant recharging has been undertaken since.
- The beach is thought to be stable since 2000 and that most material was lost in the first 10 years since the original recharge.
- There has been a gradual build-up of shingle behind the breakwaters which is understood to occur during easterly and south easterly events.

Figure 4-8 illustrates year 0 of the simulation, with the 1990s beach recharged scheme (orange line) and the current beach contour (pink line – 2017). Figure 4-9 shows the simulation after 5 years.

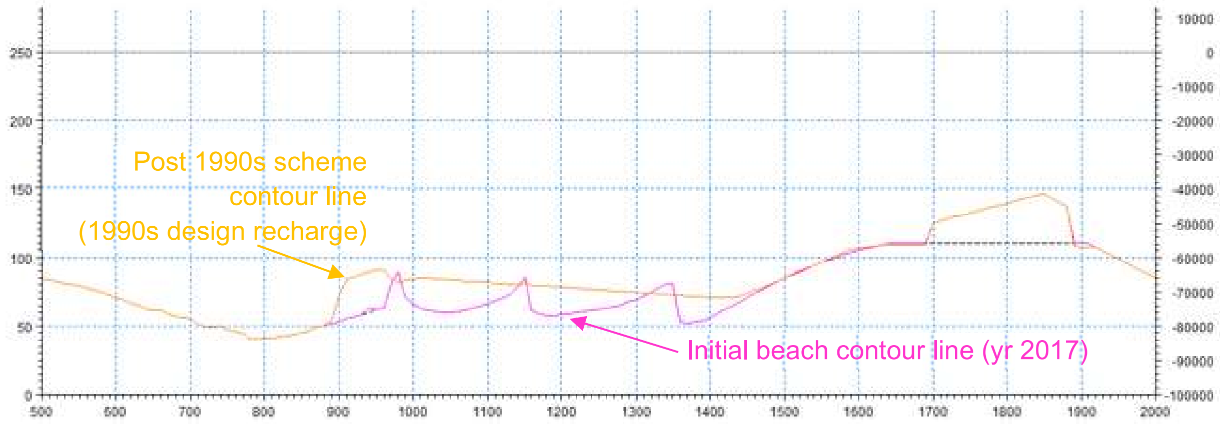

Figure 4-8: Model showing design beach nourishment following construction in the 90's

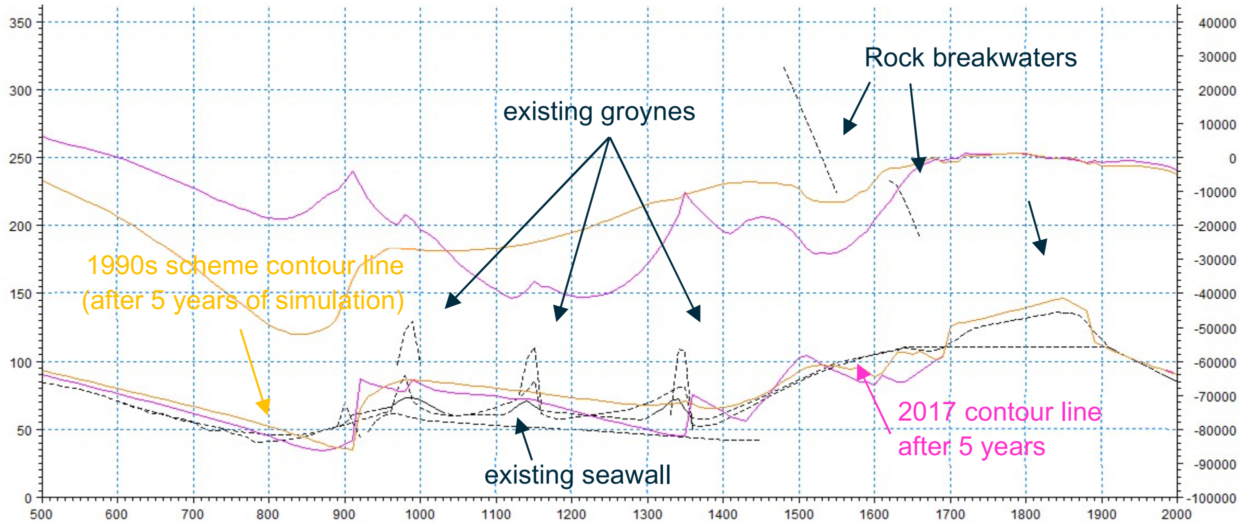

Figure 4-9: Evolution of model after 5 years

Notable observations are highlighted below:

- The bottom orange line on Figure 4-9 (seawards of the bottom pink line) indicates that beach material has deposited to the east on East Beach which matches well comments from the Steering Group that there used to be more beach in front of East Beach.
- The town frontage is influenced by the groynes. However their influence is limited when the beach profile is high and there is more bypassing of shingle around the structures. This suggests that the current groynes are not long enough to prevent beach loss to the east.
- The breakwaters have a strong influence on the beach when there is more sediment available and therefore drift rates are lower. The breakwaters in the simulation cause shingle build-up behind them.
- Material travels east, therefore supplying material to East Beach. This trend may be reverted during south easterly events.

Figure 4-10 illustrates the simulation after 10 years. Of notice are:

a. Sidmouth town frontage is further depleted and is close to matching the current 2017 baseline model profile. Material remains stable behind the breakwaters.
b. The beach at the training wall has no difference compared to the baseline model suggesting that the nourishment has had little benefit after 10 years.

c. The beach position on East Beach is close to the pink current position suggesting material continues to travel east and the protection to the cliff is reduced. It also suggests that sediment from the town frontage has depleted.

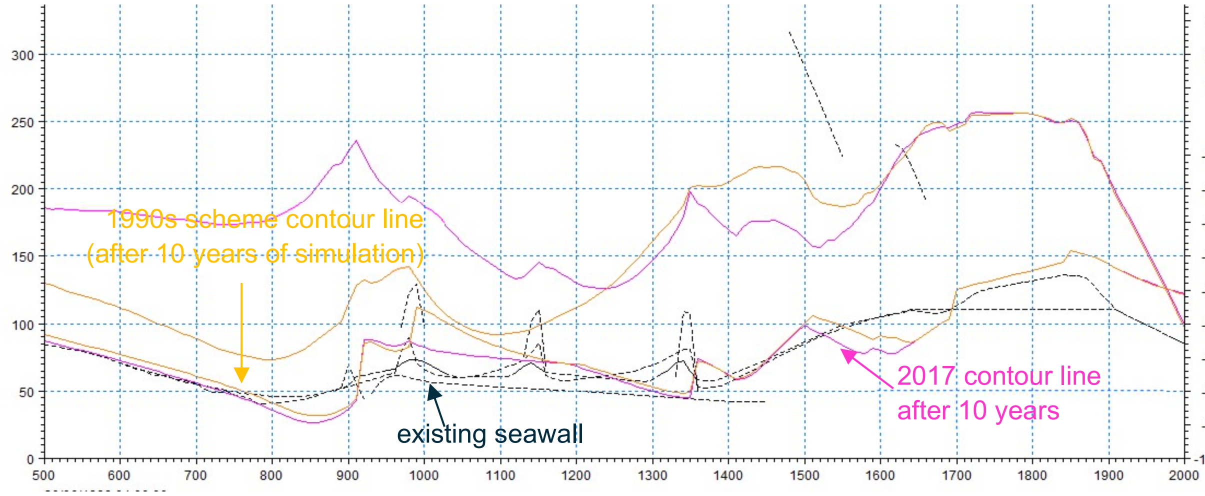

Figure 4-10: Evolution of model after 10 years

4.1.3.3 Preferred option simulation (no additional breakwater)

The preferred option was constructed in the model in a step by step manner to understand how the components influenced the results.

A new simulation with the same starting conditions as the 1990s town beach nourishment case described above was undertaken. In addition, a new groyne on East Beach was included but with initially, with no additional beach nourishment at this location. Figure 4-11 shows the development of this simulation after 10 years.

Notable observations are highlighted below (refer to figure below for location of bullet points):

a. The groyne at East Beach cliff collects sediment passing from the town frontage.
b. The town beach appears to be depleted similarly to the runs with no groyne. Therefore, the presence of a groyne on East Beach has limited influence on the behaviour of the town beach. Note the sediment drift rate lines are also similar for the town beach area.

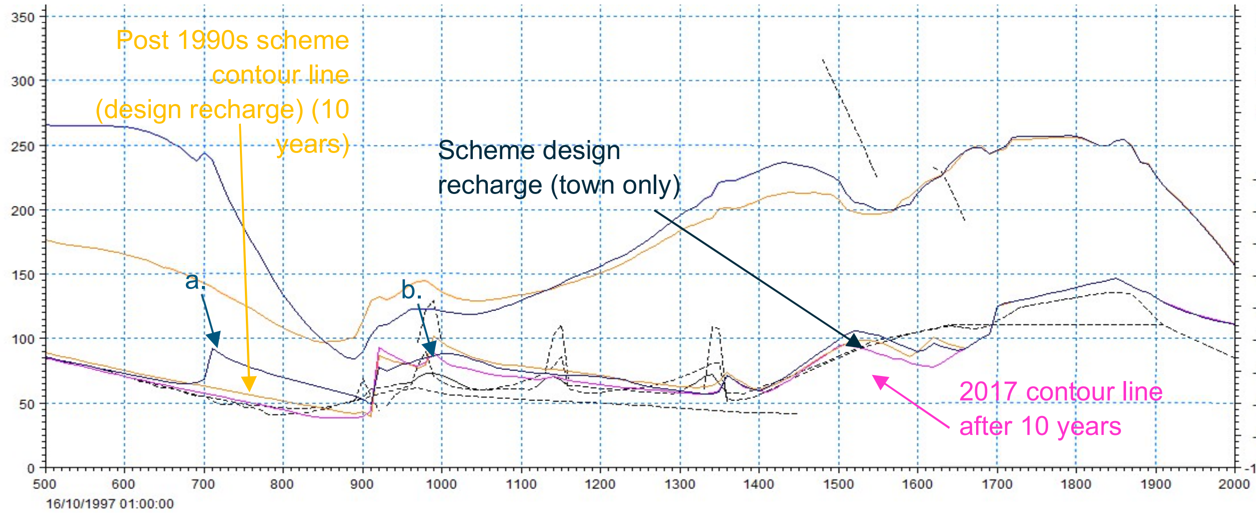

Figure 4-11: East beach Groyne and town beach nourishment

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Figure 4-12 shows the influence of nourishing the East Beach in the presence of a groyne.
Notable observations are highlighted below (refer to figure for location of bullet points):

a. After 10 years, there is little difference between the beach profile at East Beach before and after a beach nourishment. Therefore, the key benefits of renourishing East Beach are to provide a healthier beach which will reduce cliff erosion from the start.
b. The green drift line on the graphs indicates that material is still bypassing the East Beach groyne, and therefore, the groyne does not fully prevent shingle movement east.
c. There is still a build-up of beach at Pennington point which indicates the beach will be held in front of the cliffs. Note this should be taken as relative to the pink line and not to the original dotted shoreline, due to limitations of the model to represent the cliffs and a perched beach on a rocky foreshore.

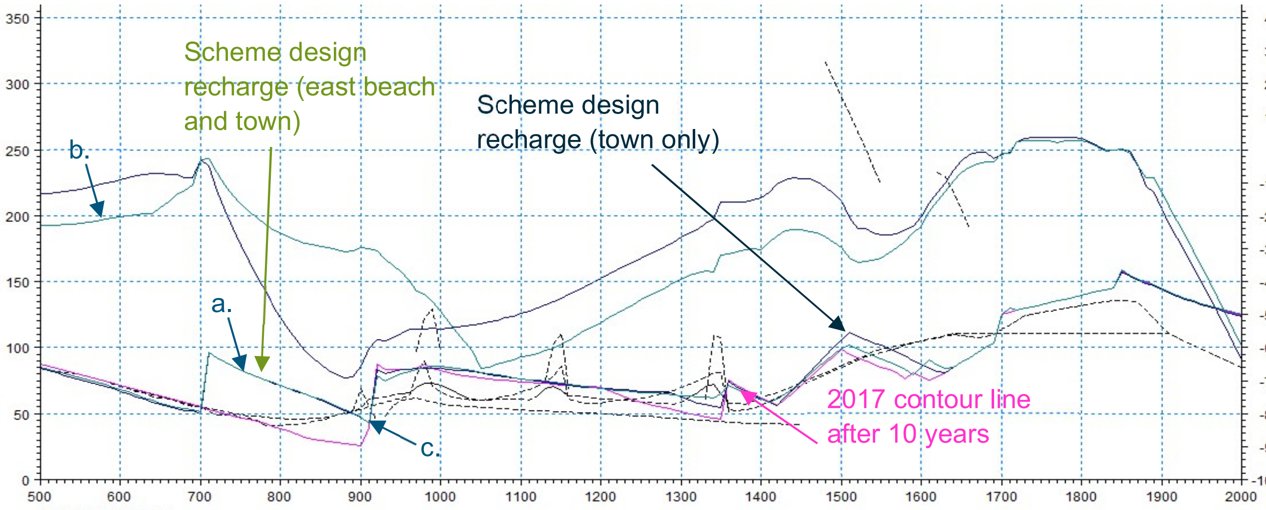

Figure 4-12: Evolution of the beach after 10 years with baseline – pink, east beach groyne only – blue and east beach groyne and nourishment -green.

4.2 Wave overtopping modelling

The preferred option along the Sidmouth Town frontage was tested and refined using the AMAZON wave overtopping model. As explained above, the model does not include the offshore breakwater opposite Frontage 4. Due to the size of the existing rock groynes, the maximum design beach the groynes can hold is a 10m wide berm with an elevation of +4.6mODN with a 1 in 7 seaward slope. This option was modelled in AMAZON and compared against the baseline 'Do Nothing' scenario.

The results indicated that, although wave overtopping reduced in comparison to the 'Do Nothing' scenario, this was not reduced sufficiently to provide adequate safety to pedestrians and cars along the esplanade. Moreover, without a raised splash wall, flood risk was not reduced to the required limit. As a result, the option of raising the existing splash wall was considered as a way of containing wave overtopping on the promenade and thus limiting flood water reaching the road.

Figure 4-13 shows the different frontages used in the model.

Figure 4-13 Sidmouth Frontages

Table 4-1 presents the wave overtopping results for the design beach and splash wall option considered along Sidmouth Town frontage for the present day, south west events (being the predominant wave direction) and Table 4-2 in 2117 with climate change. As the impact of the breakwaters on the incoming waves cannot be accurately represented in SWAN model, overtopping in the lee of the breakwaters is disregarded at this stage (Frontages 1, 2 and the westernmost section of Frontage 3). Moreover, the existing high retaining wall on Frontages 1 and 2 would prevent overtopping onto the properties and thus any propagation in to the town from this area.

Taking Frontage 4 as an example, it is noticeable that overtopping of the existing splash wall occurs during all present day return periods, with significant rates during events greater than the 1 in 20 year return periods. By raising the splash wall by ~1m, overtopping is contained seawards of the splash wall for return periods up greater the 1 in 100 year return period. Only a small amount of overtopping can be seen over Frontage 4 during the 1 in 100 year return period. This is likely to be further reduced with the construction of the offshore breakwater.

With climate change in 2117, wave overtopping is contained seawards of the splash wall for return period events lower than 1 in 100 years.

Table 4-1: Pre and Post scheme wave overtopping along the Esplanade – 2017 – South west event.

Return Period	Frontage 3 (east)		Frontage 4		Frontage 5		Frontage 6	
	Pre scheme l/s/m	Post scheme l/s/m	Pre scheme l/s/m	Post scheme l/s/m	Pre scheme l/s/m	Post scheme l/s/m	Pre scheme l/s/m	Post scheme l/s/m
1	0.14	0.00	0.20	0.00	0.30	0.00	3.01	0.00
10	0.15	0.00	1.17	0.00	0.52	0.00	4.35	0.00
20	0.42	0.00	1.82	0.00	0.78	0.00	5.50	0.00
50	1.12	0.00	3.22	0.00	1.53	0.00	7.22	0.00
75	1.56	0.00	3.95	0.00	1.95	0.00	7.99	0.00

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			4.26	1.73	2.20	0.17	9.02	0.00
200	2.34	0.00	4.53	2.17	2.69	0.18	10.03	1.29
1000	5.00		8.18		6.00		15.98	

Table 4-2: Pre and Post scheme wave overtopping along the Esplanade – 2117 – South west event.

Return Period	Frontage 3 (east)		Frontage 4		Frontage 5		Frontage 6	
	Pre scheme	Post scheme l/s/m	Pre scheme l/s/m	Post scheme l/s/m	Pre scheme l/s/m	Post scheme l/s/m	Pre scheme l/s/m	Post scheme l/s/m
1	1.48	0.00	8.57	0.00	1.85	0.00	10.92	0.00
10	5.91	0.00	21.54	0.00	5.07	0.00	18.13	0.00
20	7.67	0.00	21.89	0.00	5.11	0.00		0.00
50	11.68	0.00	25.97	3.23	9.44	0.00		1.36
75	13.54	0.00	29.84	4.38	10.68	0.00		3.55
100	14.36	0.00	30.64	5.14	14.38	3.47		4.68
200	15.56	0.00	36.52	6.06	15.55	3.57	31.54	5.84
1000	20.95	0.00	51.46	12.90	25.87	7.52	47.30	12.04

Appendix A

Royal HaskoningDHV is an independent, international engineering and project management consultancy with over 138 years of experience. Our professionals deliver services in the fields of aviation, buildings, energy, industry, infrastructure, maritime, mining, transport, urban and rural development and water.

Backed by expertise and experience of 6,000 colleagues across the world, we work for public and private clients in over 140 countries. We understand the local context and deliver appropriate local solutions.

We focus on delivering added value for our clients while at the same time addressing the challenges that societies are facing. These include the growing world population and the consequences for towns and cities; the demand for clean drinking water, water security and water safety; pressures on traffic and transport; resource availability and demand for energy and waste issues facing industry.

We aim to minimise our impact on the environment by leading by example in our projects, our own business operations and by the role we see in "giving back" to society. By showing leadership in sustainable development and innovation, together with our clients, we are working to become part of the solution to a more sustainable society now and into the future.

Our head office is in the Netherlands, other principal offices are in the United Kingdom, South Africa and Indonesia. We also have established offices in Thailand, India and the Americas; and we have a long standing presence in Africa and the Middle East.

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