REPORT

Numerical Modelling Report

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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 the 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 substantial 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 of seawalls, rock revetments, splash wall, rock groynes, offshore rock breakwaters, a river training wall / arm 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 that is situated 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 the design level (which was set as part of the 1990s schemes) and in places the toe of the seawall is now exposed. This results in wave reflection on the exposed vertical seawall which exacerbates wave overtopping and increases the risk of subsequent flooding. The 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. The cause of erosion is primarily 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 now posing a risk of outflanking and overtopping of the Sidmouth Town defences; erosion of the cliff is resulting in the gradual increased exposure of the western river wall of the River Sid to coastal sea storm conditions.

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 option identified with the Sidmouth and East Beach Management Plan.

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 and this alongside the use of the wave overtopping modelling was used to appraise and develop the preferred option.

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 which would 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.

3.1.1 Water level analysis

Extreme water levels are a key parameter when running numerical models for coastal conditions as it is this level that establishes the depth 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[1] (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.

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

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 derived, which include sea level rise and surge, are presented in Table 3-3.

[1] *UKCP09 - http://ukclimateprojections.metoffice.gov.uk/*

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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

3.1.2 Extreme wave heights

The coastal frontage of Sidmouth is orientated in a north east to south west direction. Waves approach the coastline from the south east, south and south west 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 for days at a time as presented below.

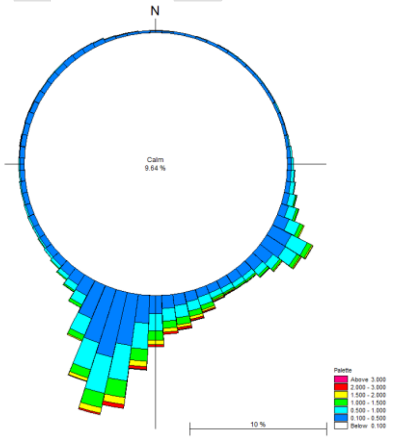

Figure 3-2: Met Office WaveWatch III Hindcast wave record

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.

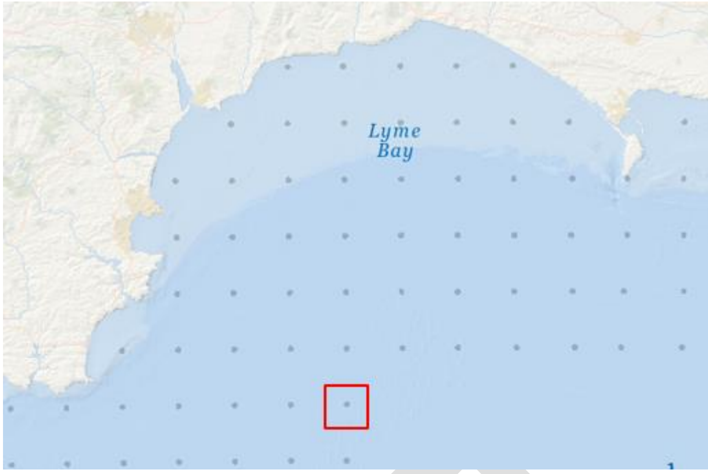

Figure 3-3: Location of Wavenet Hindcast output point

A statistical analysis was undertaken to derive extreme offshore wave heights and 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 derived.

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

Direction Section / Return Period	South East 90 / 150deg	South 150 / 210 deg	South West 210 / 270 deg
T1	3.49	4.54	6.87
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 extreme wave heights derived were applied across all four time epochs (2017, 2037, 2067 and 2117).

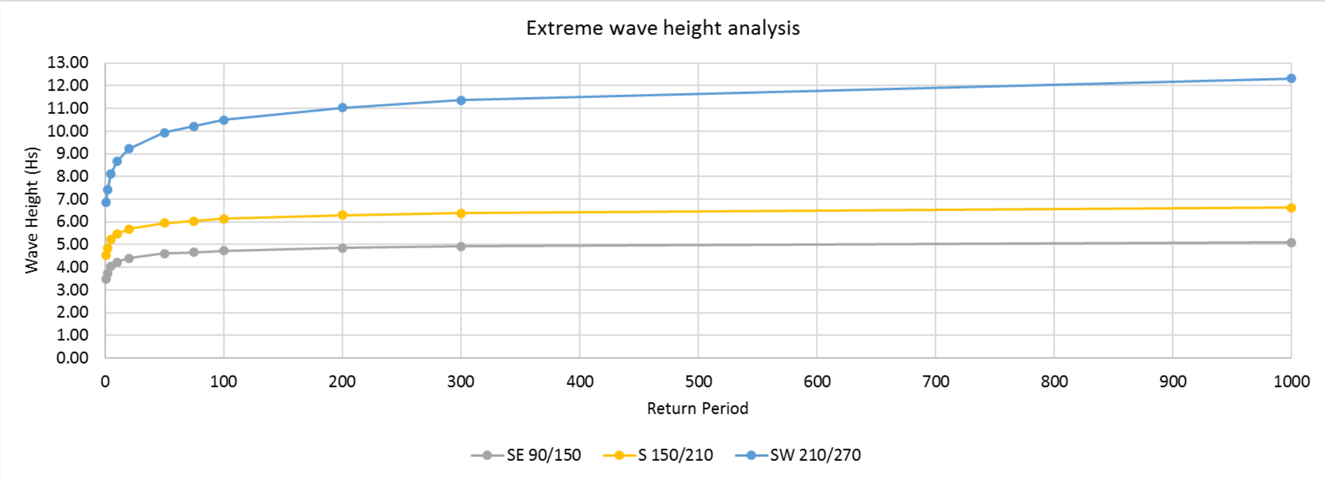

Figure 3-4: Extreme wave height analysis plot

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.

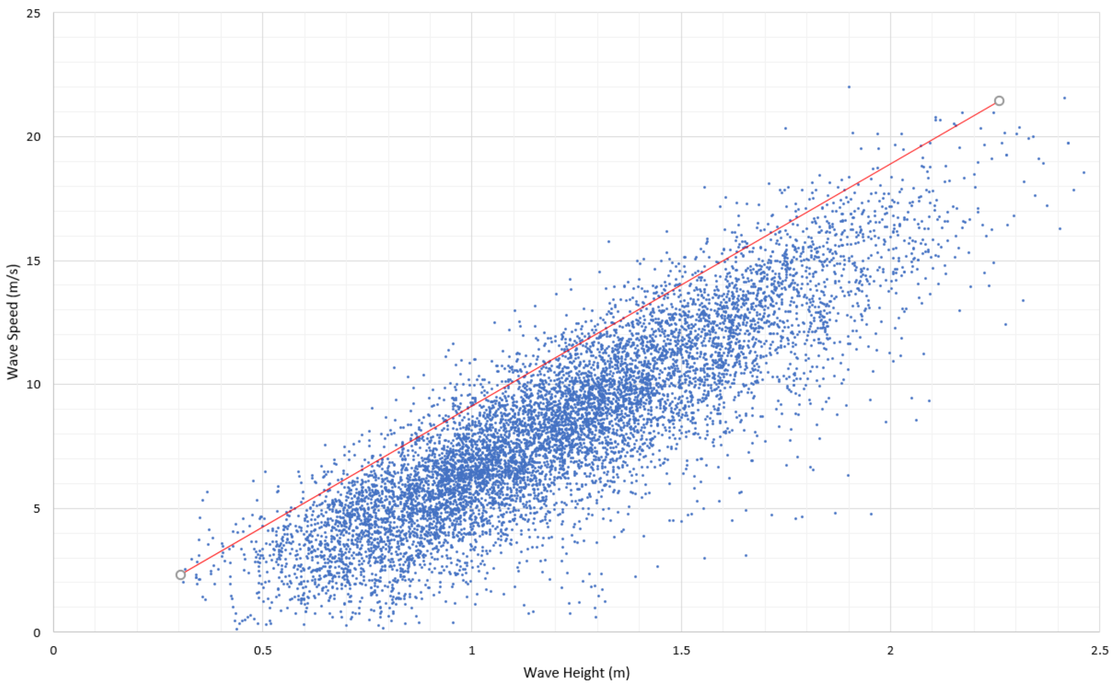

Figure 3-5: Hindcast dataset analysis between wind speed and wave height

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.

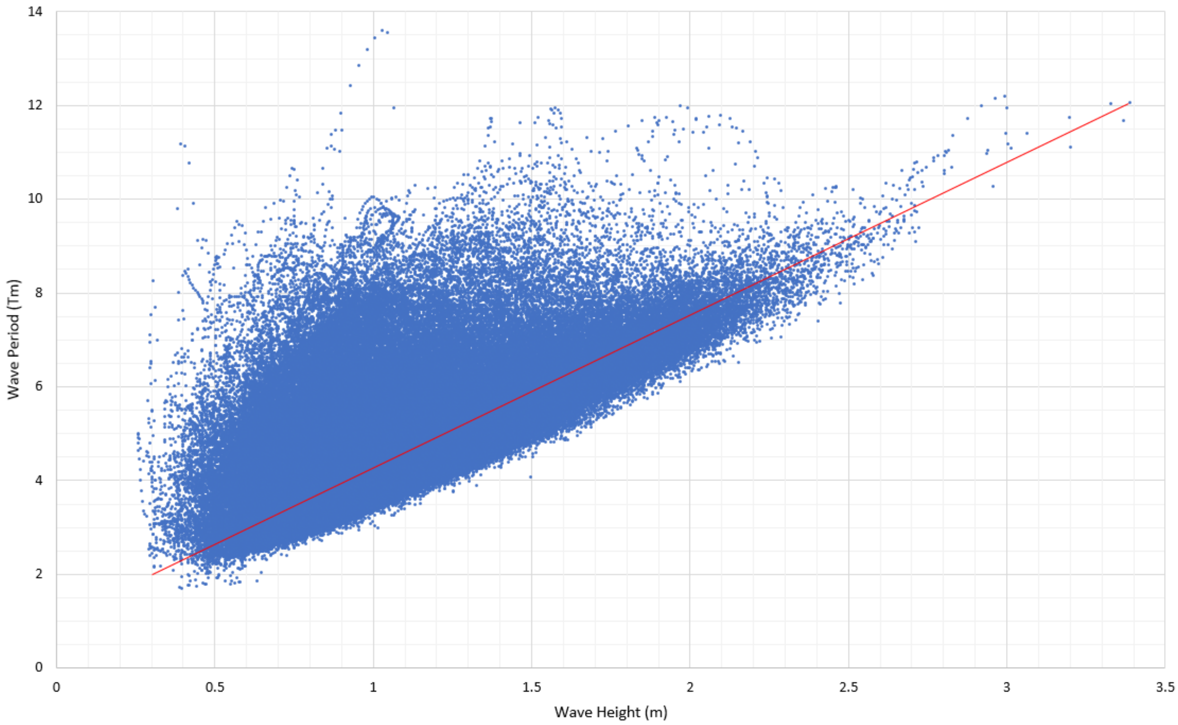

Figure 3-6: Hindcast dataset analysis between wave period and wave height

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 when estimating a 100 year coastal flood event to assume that say, a one hundred year wave will occur with a one hundred 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 level 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.

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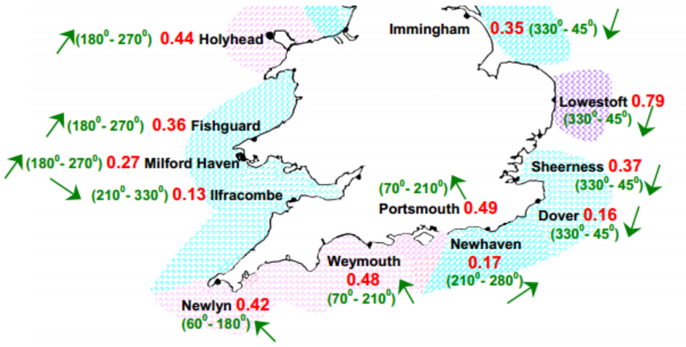

Figure 3-7: Correlation Coefficient (p, wave height and sea level), wave direction section in which dependence is higher.

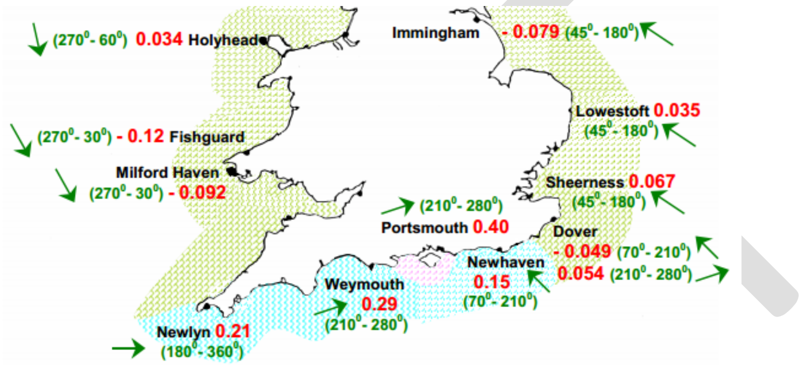

Figure 3-8: Correlation Coefficient (p, wave height and sea level), wave direction section in which dependence is lower

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

A existing SWAN model was made available from 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.

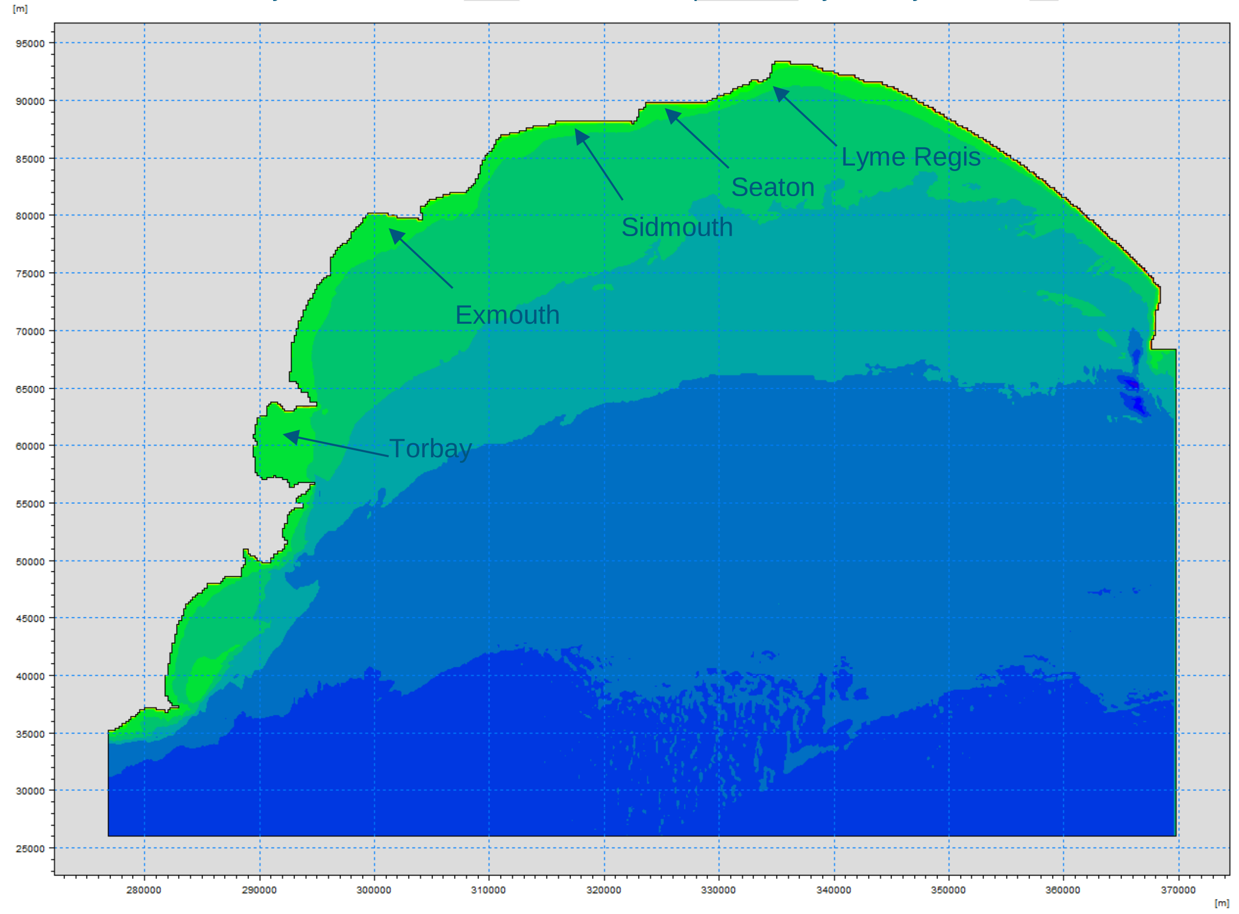

Figure 3-9: SWAN model domain

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.

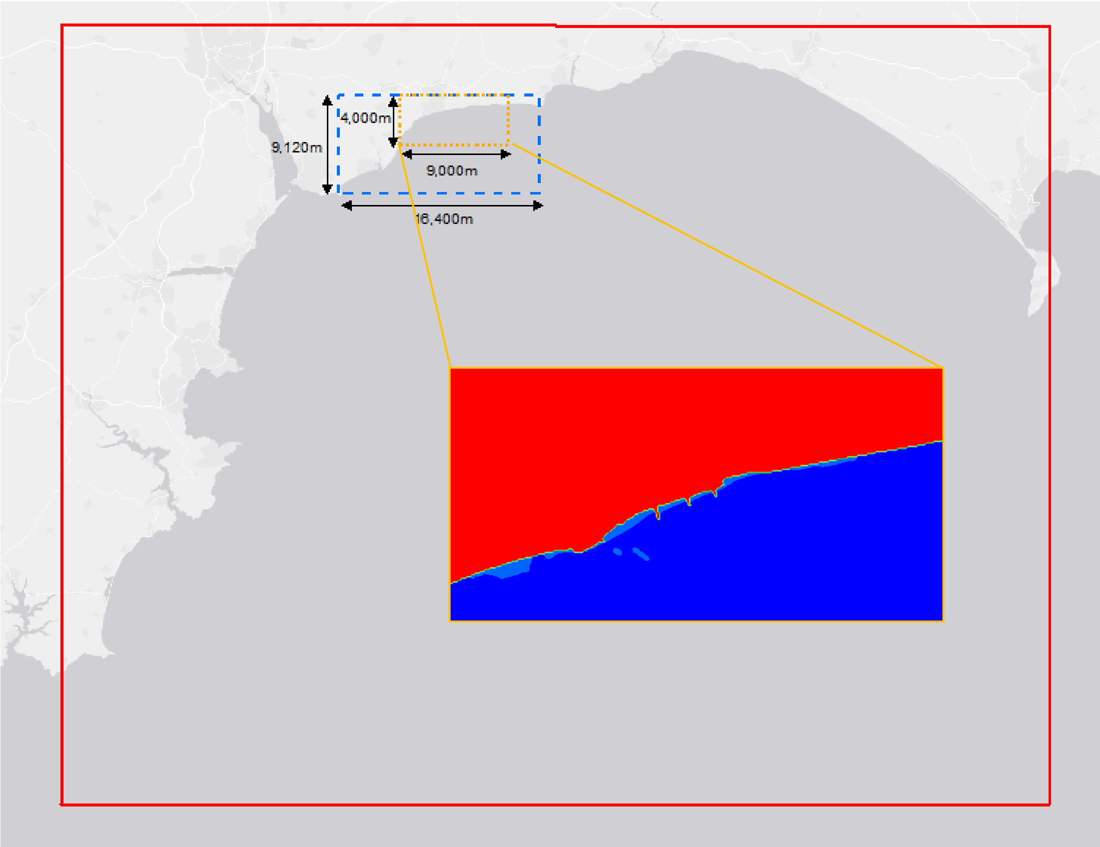

Figure 3-10: Swan model nested mesh

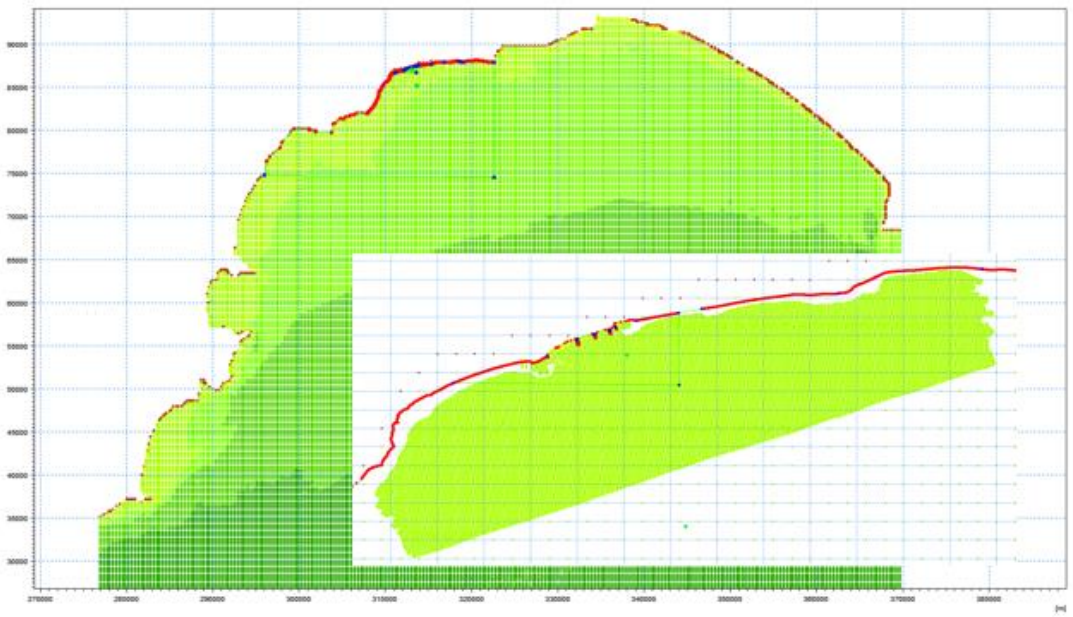

Figure 3-11: Bathymetric data coverages within the model.

3.2.3 Model calibration and validation

The SWAN model used 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 was 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 comparing to the measured data and illustrates that as expected the model performs well.

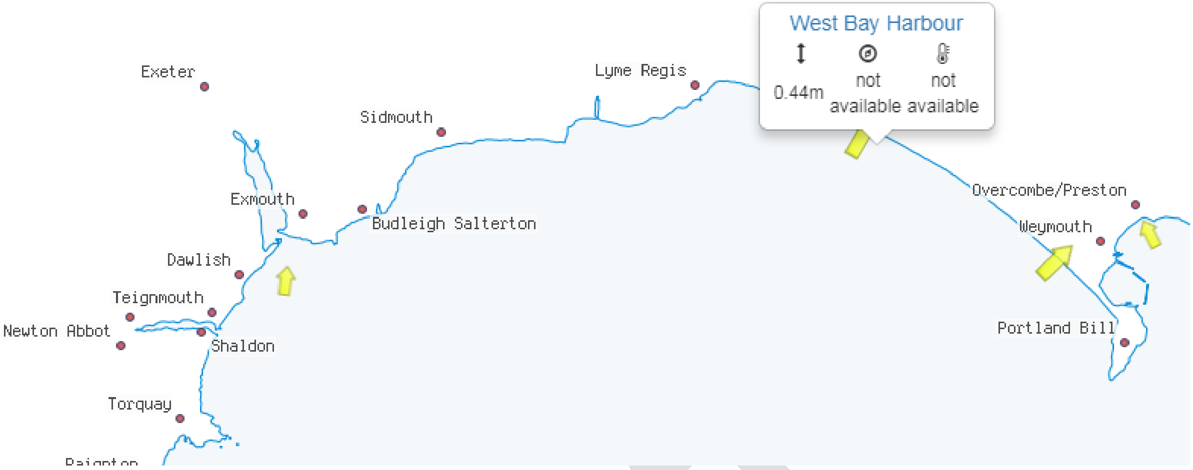

Figure 3-12: Location of West Bay wave buoy

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 the south west, however at times storm events can come from the south and south east. These storm events expose the River Sid training wall and East Beach cliffs, where wave overtopping can occur along the training wall. For this reason, wave transformation modelling was undertaken for both a South West event and a South South East event.

3.2.5 Nearshore extreme wave heights

The study area was divided into 8 frontages as illustrated in Figure 3-13. 6 frontages were located along the Sidmouth Town frontage and 2 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 outlined in Section 5.3.3. Table 3-9 presents an example of the modelled nearshore wave heights at 280m offshore. As discussed in the Section 6.3.2 for each wave overtopping run nearshore wave heights were extracted at one wave length from the shoreline.

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Figure 3-13: Sidmouth frontages

Table 3-9: Modelled nearshore wave heights for a 2017 South West 1 in 200 year return period 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

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 in order to assess the present and future flood risk. The overtopping analysis included an assessment of the following:

- Overtopping risks at present and in 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.

Figure 3-14: Wave overtopping along Sidmouth

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).

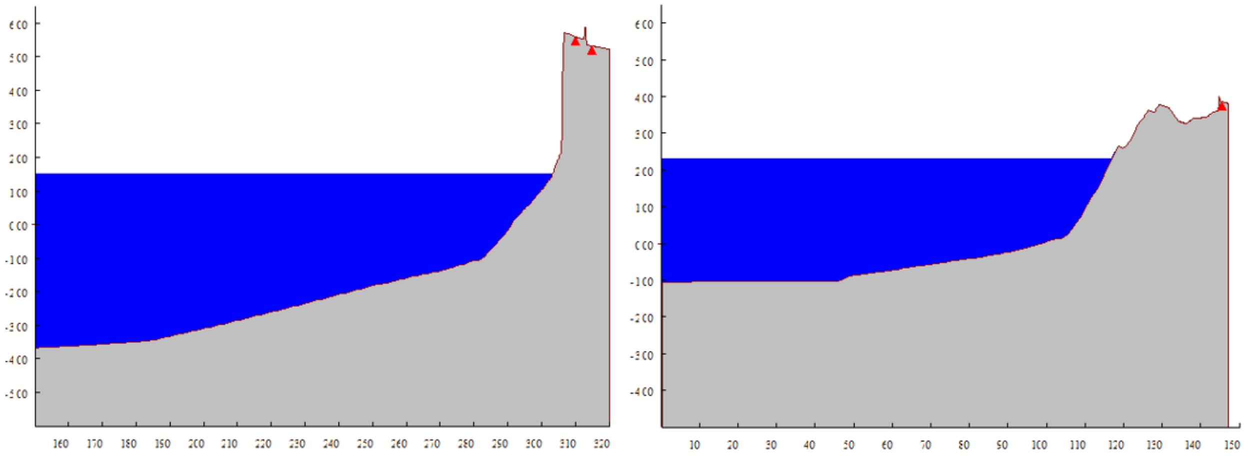

Figure 3-15: Example of AMAZON model run for profile 1 and profile 5.

3.2.9 Model set up

The Sidmouth frontage was divided into 8 frontages. The sub-frontages allowed accurate representation of the different beach profiles and defence sections along the frontage. Available beach topographic survey profiles were obtained from the Plymouth Coastal Observatory 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 created from LiDAR. The profiles were analysed and an appropriate average profile was chosen for present day overtopping modelling. For climate change it was assumed that beach levels would slowly drop under a ‘Do Nothing’ scenario. 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 Profile 3 under a scenario of a gradually lowering beach as time goes on.

Figure 3-16: Sidmouth sub-frontages and profiles

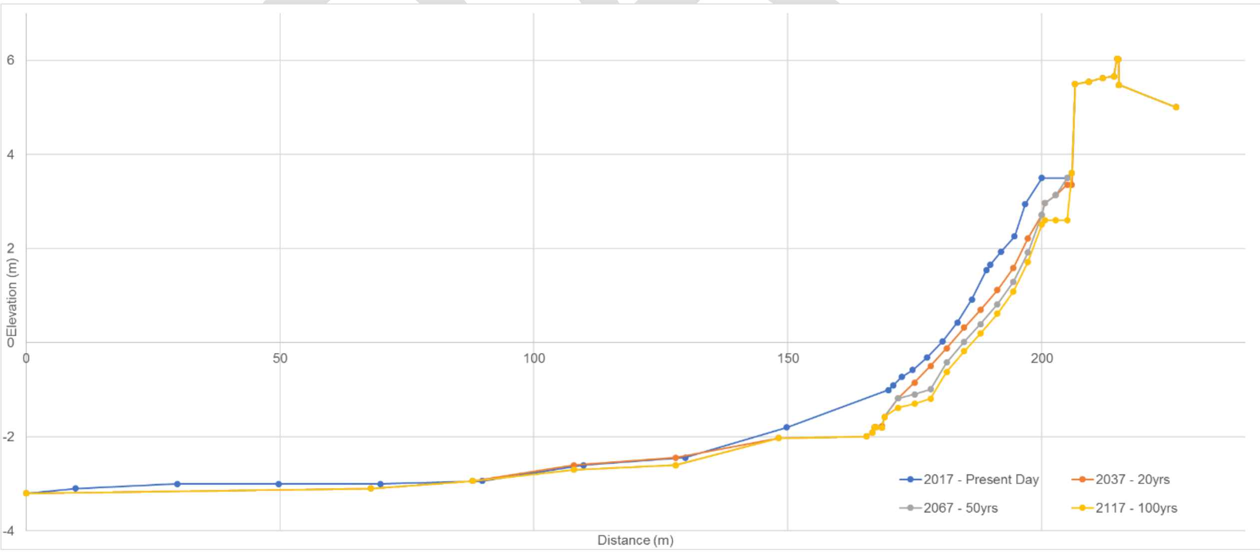

Figure 3-17: Profile 3 beach profiles used in the wave overtopping assessment.

For each model simulation wave heights were extracted in front of each profile location. The wave heights were extracted from a distance offshore to match the wave length being simulated in AMAZON (i.e. if the wave length was 200m the nearshore wave height 200m from the shoreline was extracted). This method is preferred when using AMAZON, compared to obtaining a wave height at a single point for all runs, as it ensures the correct wave height is used for the representative wave length and location, therefore allowing sufficient wave run up 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 Frontage 3 and 4. The analysis identified the worst case joint probability combination to use in the wave overtopping assessment per return period and identified that it is the wave height that has a greater influence on wave overtopping rather than water level.

Table 3-10: Wave overtopping assessment along Profile 3 and 4 – 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 F3 (l/s/m)**		**Wave Overtopping F4 (l/s/m)**	
1	10.49	13.75	25.05	2.35	6.028	2.3982	5.4041	1.8797
2	10.42	13.71	24.96	2.41	8.4638	4.2585	6.2224	2.2039
3	9.71	13.28	24.02	2.47	5.8502	2.8018	3.9819	1.2586
4	9.16	12.94	23.27	2.55	3.8177	1.7732	2.5118	0.689
5	8.62	12.58	22.51	2.61	2.3477	1.0358	1.5875	0.4503
6	7.90	12.10	21.46	2.69	1.6564	0.706	0.9925	0.2304
7	7.36	11.72	20.64	2.75	1.1442	0.4998	0.609	0.1193
8	6.82	11.33	19.79	2.81	0.8345	0.3509	0.3596	0.0501
9	6.10	10.79	18.61	2.91	0.7344	0.3331	0.3226	0.0742
10	5.57	10.36	17.67	2.98	0.704	0.3601	0.241	0.0395
11	5.03	9.90	16.69	3.05	0.6498	0.3547	0.1384	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[2]. 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 where present behind the splash wall in order to match the location where overtopping would be applied in the TuFLOW model.

3.3 Flood propagation modelling

The fourth stage in the modelling process was to undertake flood inundation modelling. This involves simulating how the wave 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.

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

3.3.1 Model set up

An existing TuFLOW flood inundation model was developed during the BMP and this was obtained from EDDC. The model domain selected covered the low lying land across the town of Sidmouth and tied into high ground to ensure all potentially vulnerable low lying land was 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[3] 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

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

Fence shrubs	0.05	Roads tracks and paths manmade	0.02
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. 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.

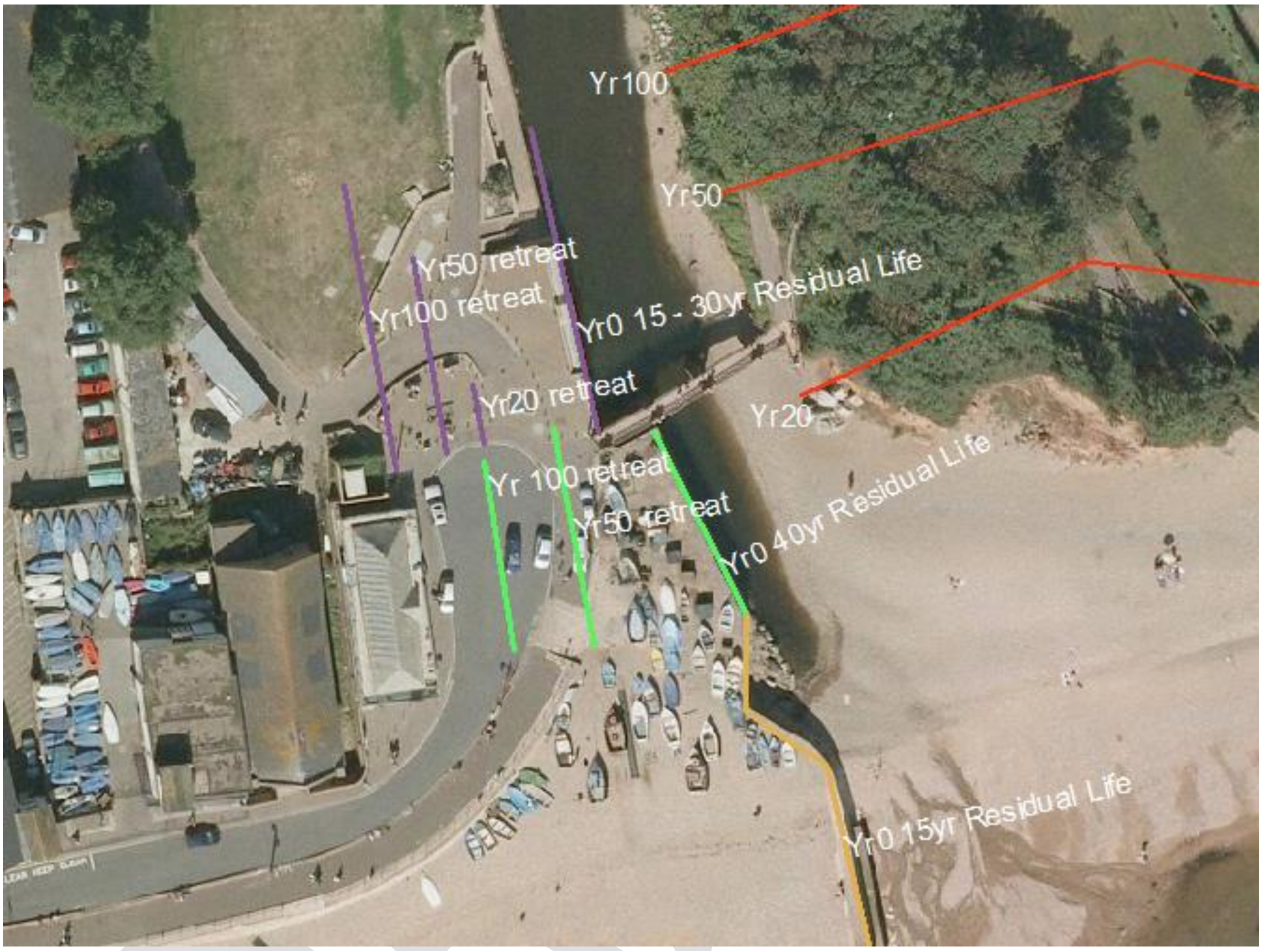

Figure 3-20: Do Nothing cliff retreat and defence failure over 100 years.

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.

The wave overtopping calculated from the AMAZON wave overtopping modelling for each event was the input into the TuFLOW model. Wave overtopping was calculated in AMAZON per metre for 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.

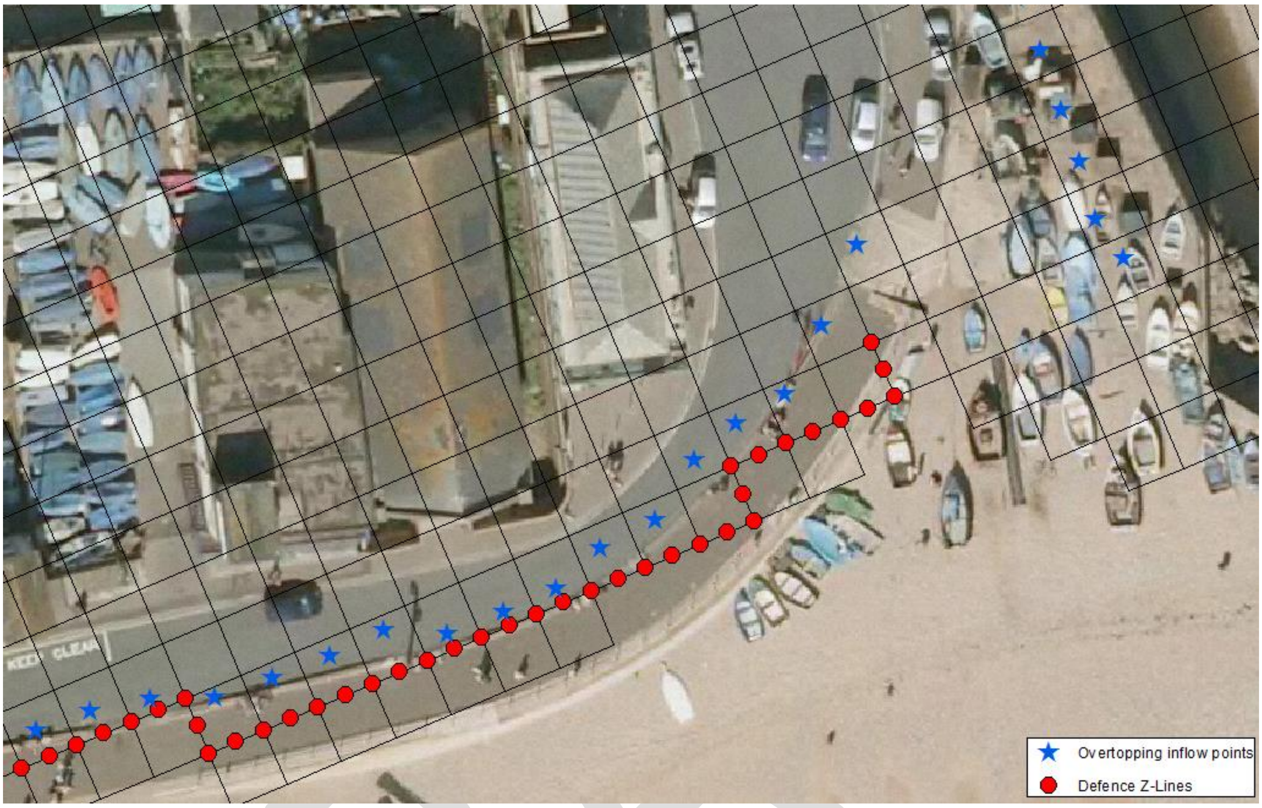

Figure 3-21: Wave overtopping inflow.

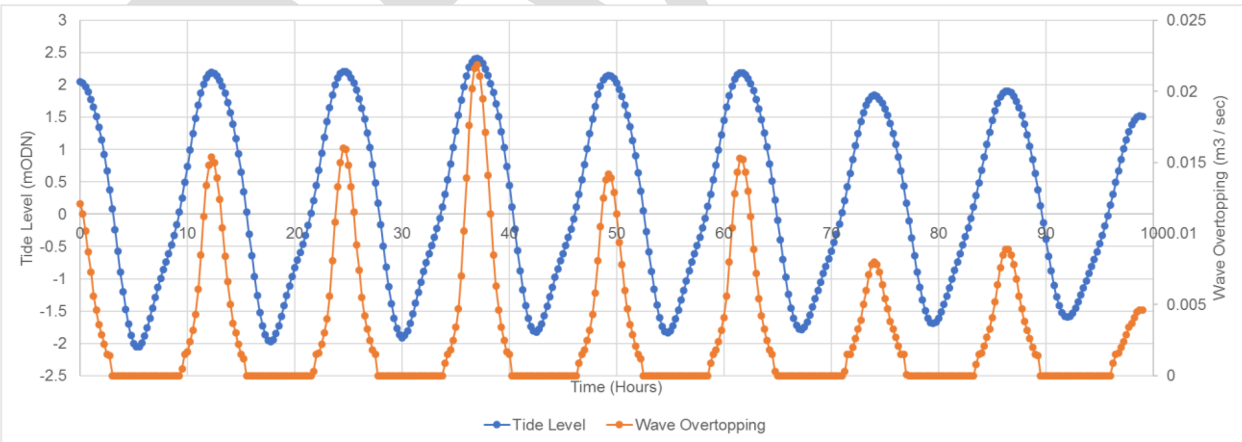

Figure 3-22: Tide vs overtopping

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. As discussed and agreed with EDDC and the Environment Agency the model was simulated for 48 hours to represent a typical extreme storm event along Sidmouth.

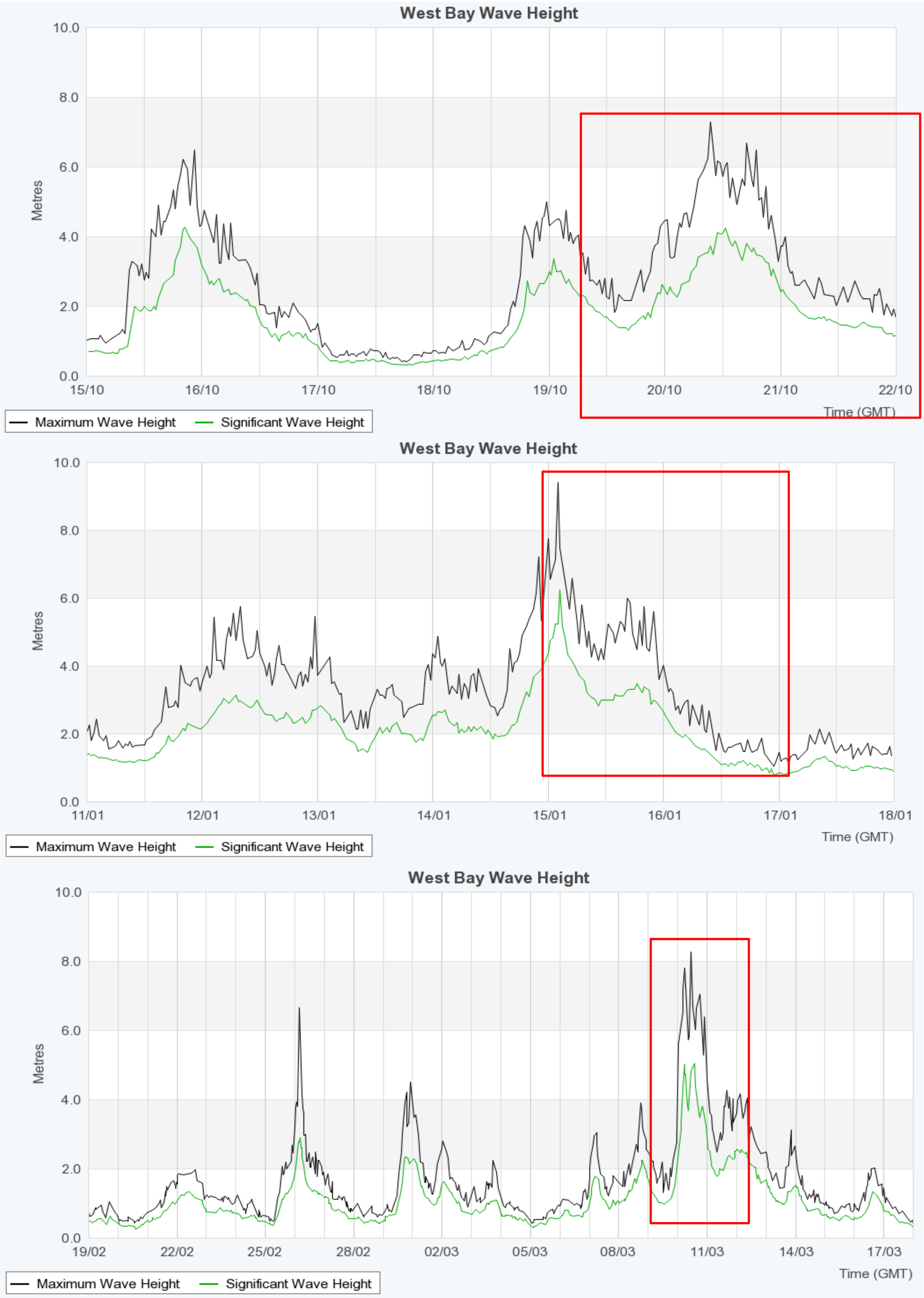

Figure 3-23: Measured wave heights at West Bay wave buoy during recent storm events. (A: Storm Brian, 2017, B: Winter Storms January / February 2015, C: February / March 2008).

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 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.
- There is no allowance for surface water drains that may drain any overtopped water back out to sea.
- Wave height remained the same for each run over topping. Only the change in water level over a tide cycle was considered.
- 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, this allows for breaking/non-breaking waves and currents. This can include for different 1-d wave propagation theories to transform the waves from the nearshore to the element on the profile grid. These 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.

This is 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.

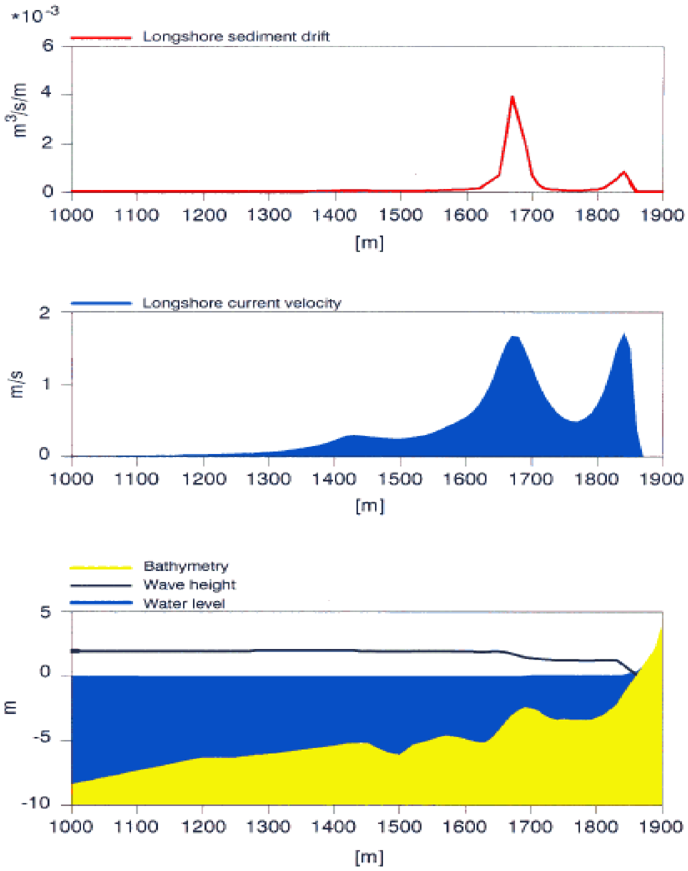

Figure 4-1: Illustration of the modelling of long-shore drift distribution.

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 dy/dt 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 dQ/dx is the change in sediment transport in a profile slice and qs and qo 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 dy, see Figure 4-3.

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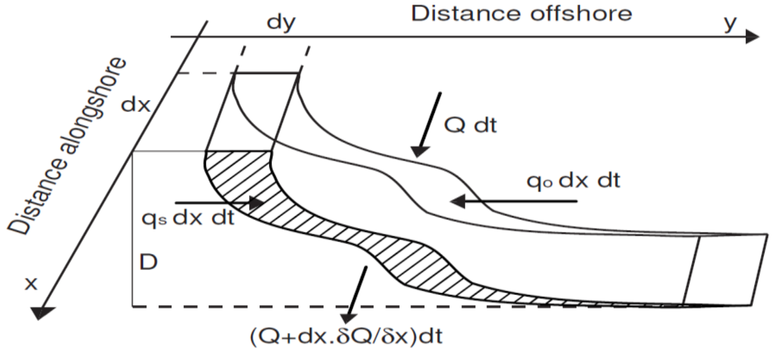

Figure 2.10: Equilibrium cross-shore profile

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. In the case of this study profile sections were established at 10m intervals over a shoreline length of 4200m.

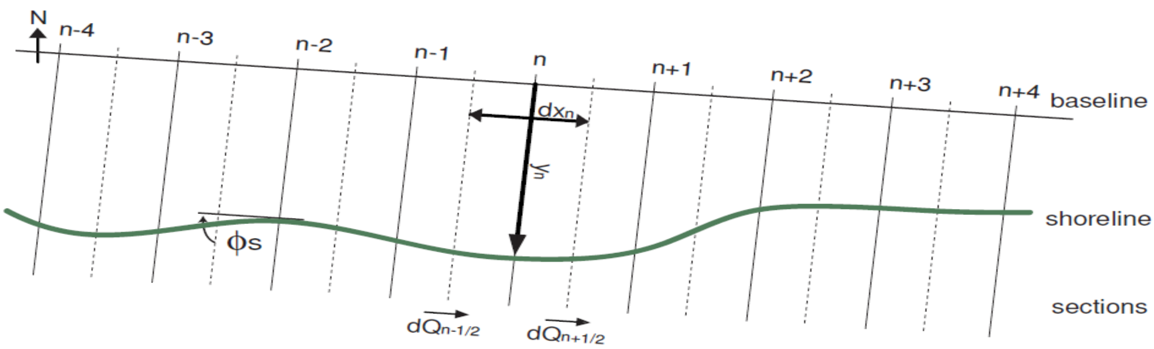

Figure 4-3: Illustration of coastline update method in one dimensional model.

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 on the beach. For this, 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 the shallower gradient and lower down the beach profile. It is the shingle that is more effective at absorbing wave energy and the material that is most effective at protecting the shoreline. For this reason, the model was set up only to represent the shingle area of the beach.

Figure 4-4: Sidmouth Beach.

Notable model settings are outlined 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 2mAOD water level.
- A single beach profile was taken from a typical full profile 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.

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- The model was set so the overall shoreline from north to the offshore normal is at an angle of 160 degrees from north.
- 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 was 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.

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

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Figure 4-6 presents the baseline model and Figure 4-7 presents the coastal evolution after 5 years.

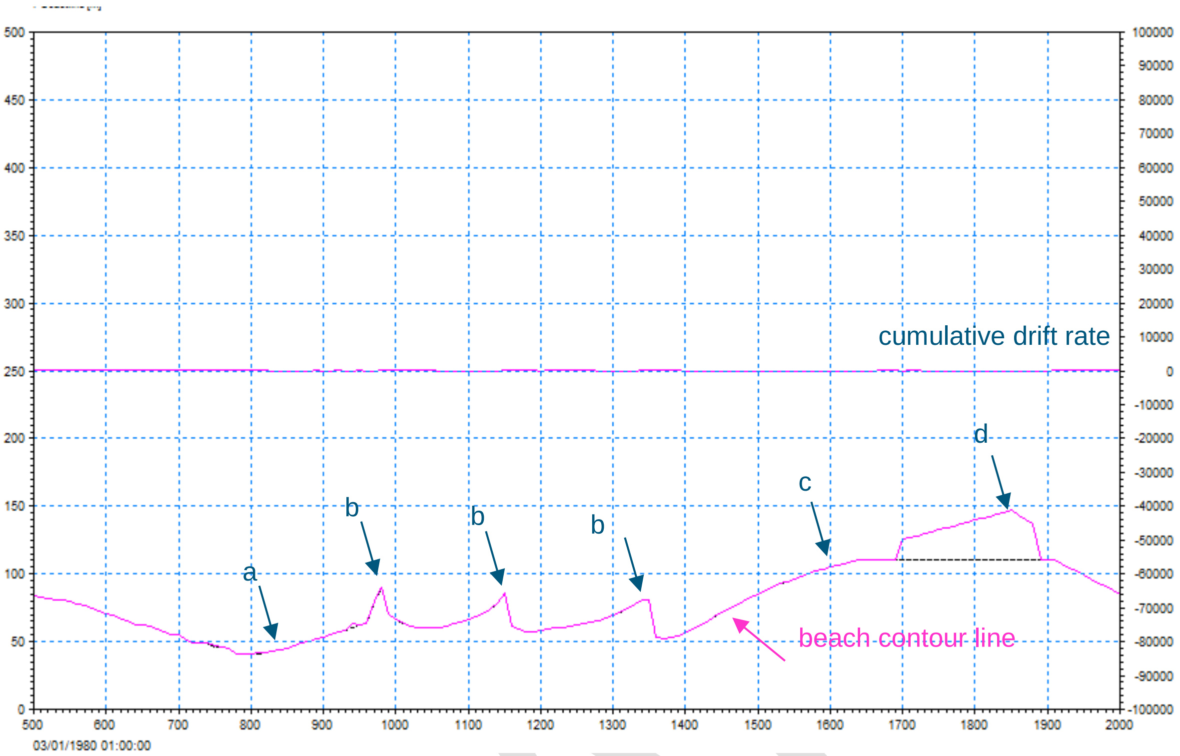

Figure 4-6: Initial starting conditions.

Figure 4-6 has two lines the lower line representing the beach contour line. In this you can see the position of the three-main rock groynes (b) on the beach at chainage 970, 1130, 1340 and the Connaught Gardens headland (d). The horizontal axis is a chainage along the baseline in meters.

The right-hand axis is cumulative drift rate in m3 over the duration of the simulation and relates to the upper line on the plot. This is the total amount of sediment net that has passed a profile over time. The net means it is based on direction, so in this model the negative drift indicates a movement to the east and positive drift a movement to the west.

The Left-hand axis is the offset from the baseline in meters and relates to the lower line on the plot. This is similar for all the following graphs.

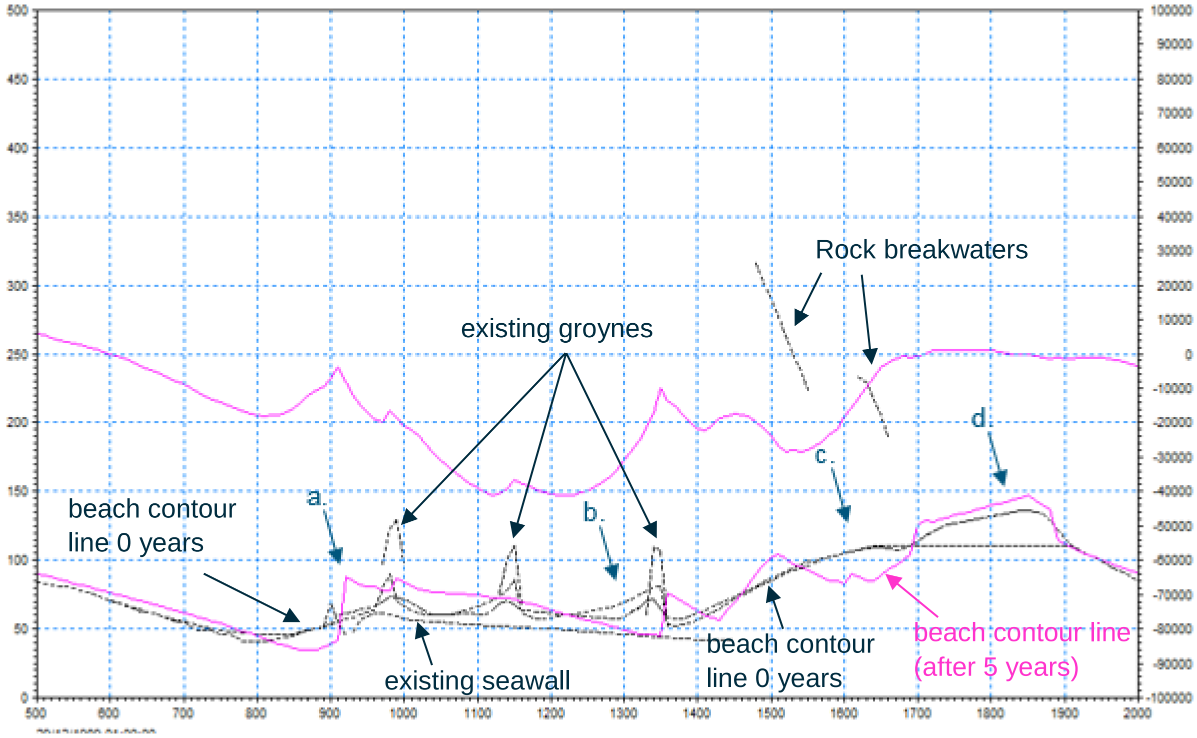

Figure 4-7: Baseline coastal evolution after 5 years.

Figure 4-7 shows the baseline evolution of the coastline after 5 years of simulations. The pink line represents the coastline and the dotted black line represents the starting positon of the simulation. Considering the coastline, a few points are highlighted.

The main town beach behaves as expected from the observed behaviour of the beach. Watch this over several time-steps the beach material can vary in position as the waves change direction.

a) There is build-up of material on the west of the training wall and a cut back of material along East Beach. The up-drift influence is stronger than observed though not sufficient flexibility in the model to reduce this any further. It should be noted here that this model only simulates the beach movement and it assumes the whole coast is an infinitely deep beach infinitely inland. Though the beach is thin and is backed by a cliff. Therefore, the model should only be considered in terms of relative beach movement and that effects here are not predictions of reality. The training wall in the model is acting more effectively as a groyne than in reality so this cut back is exaggerated.

b) The pink line here shows the beach position at a moment in time after 5 years of simulation. At this point in the model simulation the beach has been cut back as far as the seawall (black dotted line). This shows that shingle profile is depleted and that the seawall is exposed. The area of material in that groynes bay also suggest that it is less than the beginning of the run indicated by the black dotted line. Slightly further east around chainage 1020 there appears to be a build-up of material suggesting the material has bypassed the groynes. This is also supported by the shape of the drift curve above where the peaks in the line are the groynes effectively holding the material.

c) This shows the influence of the offshore structures on the beach behind blocking incoming waves. This is balanced in the model with the positioning of the points to create a similar influence if this was smoothed as would occur in reality given that these structures have a height of about 3.6mAOD and waves will further diffract and spread behind them softening the shadow they create.

d) At Connaught Gardens the coastline is fixed in the model and a revetment is present which prevents erosion.

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 to simulate as it informed the performance of the model to that experienced on site. 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. However, shingle is known to move within the cell during storm events.

- The full design beach nourishment was placed in the 1990s and no significant quantities of material has been added or removed following this date.

- Given that there was a significant amount of material was added in the 1990s and the beach is thought to be currently stable since 2000. Most the beach nourishment added in the 1990 has been lost in the 10 years following the nourishment. This gives the basis of validation for the model as otherwise it is not possible to measure longshore drift rates and the beach monitoring records are not complete back to that date.

- There has been a gradual build-up of shingle behind the breakwaters.

Figure 4-8 illustrates the starting position for the baseline simulation (pink line) and the position post 1990s scheme (orange line). Figure 4-9 shows the simulation after 5 years. The orange line is seaward of the pink line on East Beach which suggests the nourishment of the town beach has also supplied some material to the east. This matches well with observations from the Steering Group members with comments that there used to be more beach along East Beach in the past. Therefore, this suggests that if the town beach had not been nourished in the past the erosion along East Beach is likely to have been worse.

Comparing the position of the two coastline positions after 5 years of simulation the following can be concluded:

a) The town frontage is influenced by the groynes however the influence is limited when the beach is full and there is more natural bypassing of shingle around the structures. this suggests that the current groynes hold a beach when full but do not prevent loss of sediment to the east.

b) 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.

c) Material travels east therefore supplying material to east beach.

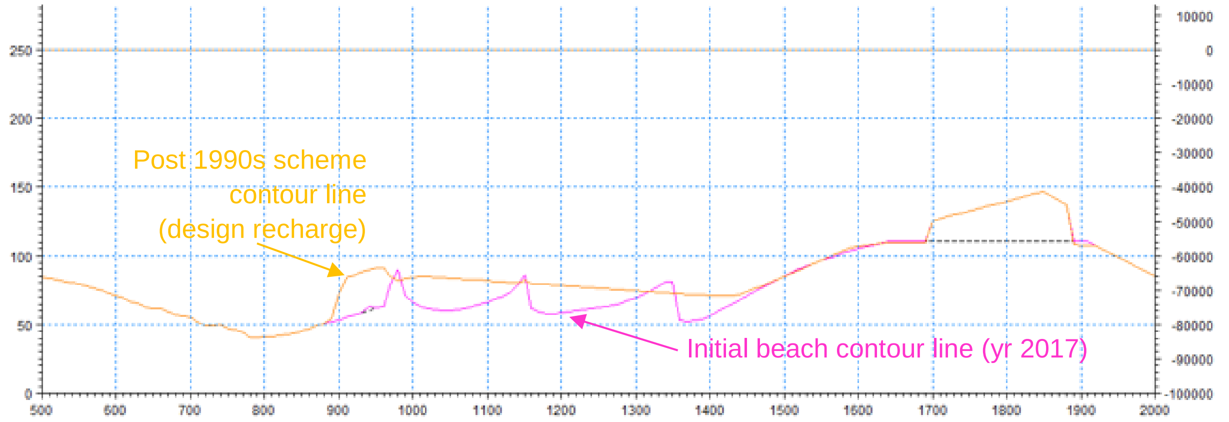

Figure 4-8: Model showing design beach nourishment following construction in the 90's

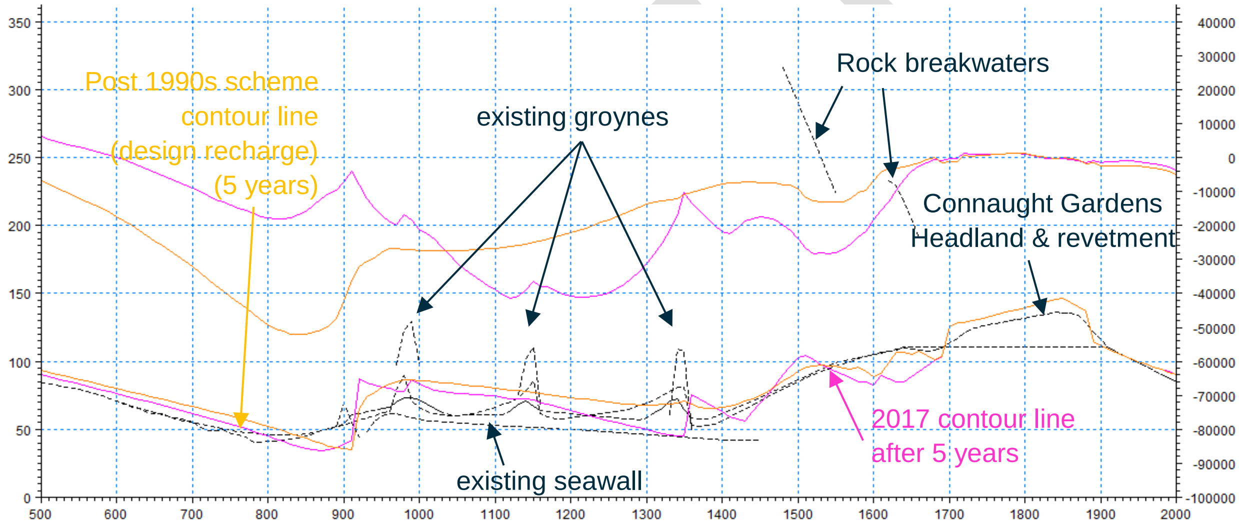

Figure 4-9: Evolution of model after 5 years

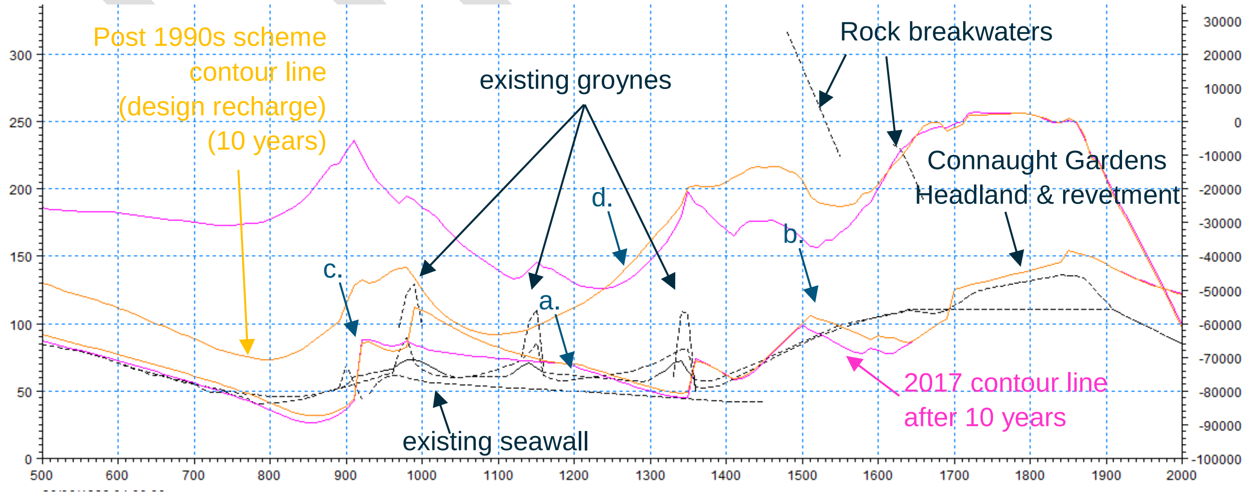

Figure 4-10: Evolution of model after 10 years

Comparing the position of the two coastline positions after 10 years (Figure 4-10) of simulation the following can be concluded:

a) Sidmouth town frontage is further depleted and is close to matching the baseline mode position.

b) Material remains stable behind the breakwaters.

c) 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.

d) The beach position is close to the pink pre-1990s 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.

4.1.3.3 Preferred option simulation

The preferred option was constructed in the model in a step by step manner to understand how the components influenced the results, therefore a groyne was added with no additional nourishment, then later with nourishment.

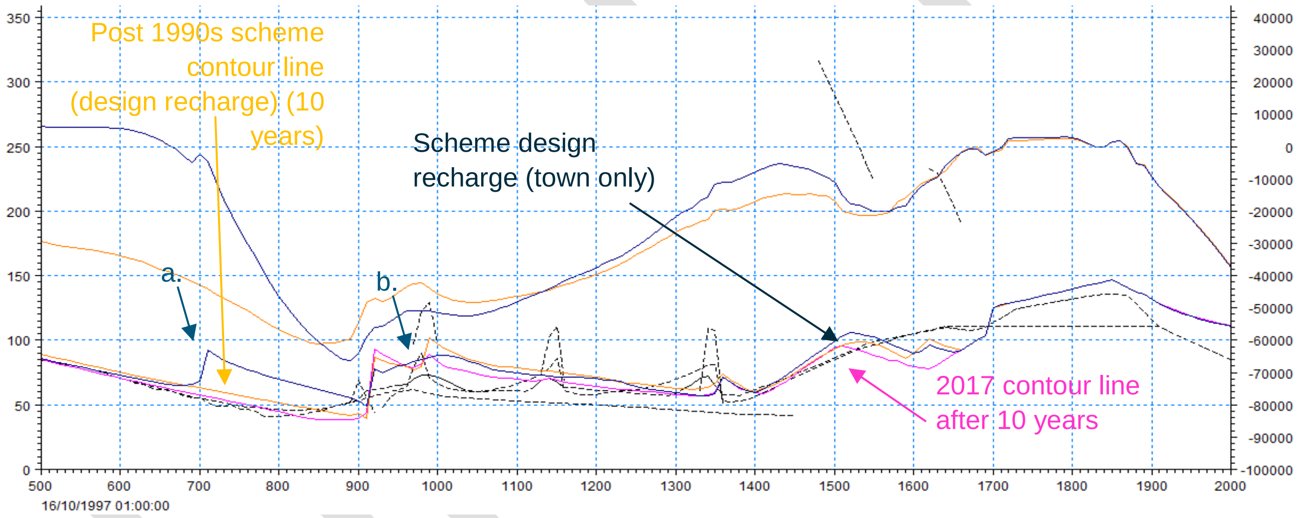

Figure 4-11: East beach Groyne and town beach nourishment

Figure 4-11 shows the development of the simulation after 10 years. This has the same starting conditions as the Town beach nourishment case described above and indicated by the orange lines. The difference in the blue line is the presence of a new groyne at east beach, this has no further additional nourishment.

a) Here it can be seen the presence of the groyne collects sediment passing from the town frontage. Note the apparent erosion to the down drift side is exaggerated due to the model being unable to represent cliffs within the model and it assumes there is further beach available. In effect the beach will be bare/limited due to lack of material as is evident in the baseline run. Also the peak in the drift line above indicating the prevention of movement of sediment around the end of the structure.

b) Here the town beach appears to be depleted similarly to the run with no groyne. Therefore, the presence of a groyne on each 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.

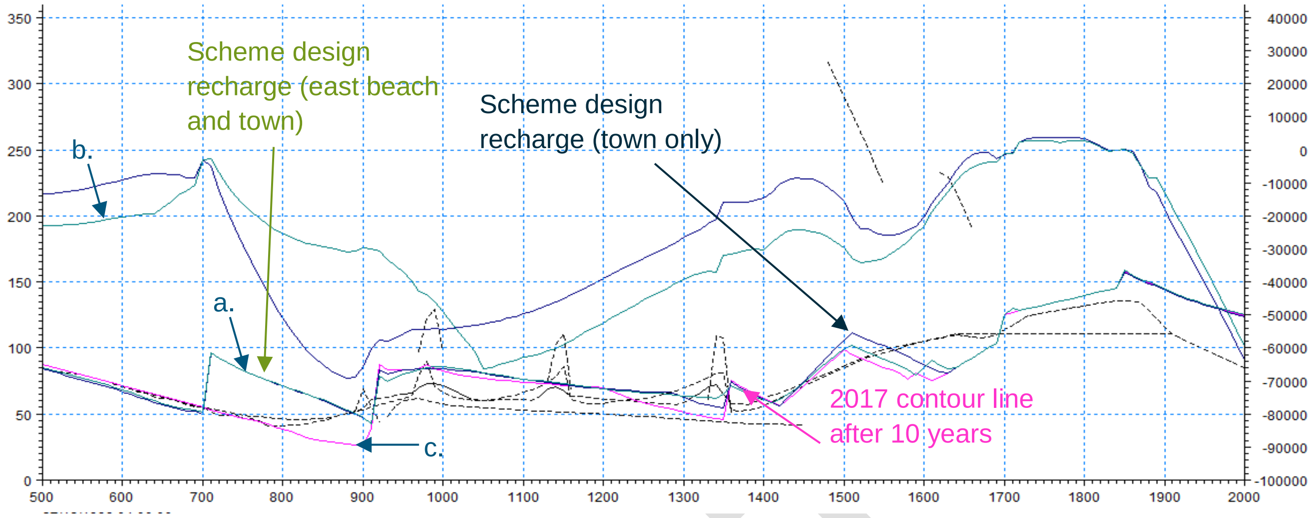

Figure 4-12: Evolution of the beach after 10years with baseline – pink, east beach groyne only – blue and east beach groyne and nourishment -green.

Figure 4-12 shows the influence of nourishing east beach in the presence of a groyne.

a) From this it can be concluded that there is little difference in outcome after 10 years whether east beach is nourished or not. Therefore, the benefit in nourishing east beach occurs initially when there is no sediment and there is not yet time for sufficient shingle to build up from the town beach. The initial nourishment ensures that the cliff is protected from the start.

b) The green drift line also shows that there is more material bypassing the east beach groyne and supplying material further to the east, i.e. 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 the limitations of the model to represent the cliffs and a perched beach on a rocky foreshore.

4.1.3.4 Sensitivity check on influence of groyne length

In an idealised case the east beach groyne would be infinitely long to hold all the material travelling east. This material could then be recycled back onto the town beach.

The baseline case has limited beach material and a limited supply of shingle to the east. Therefore, when the proposed groyne is constructed it is likely to have little impact on the cliff frontage further east of the groyne but will ensure that the majority of additional material being recharged onto the town frontage is held within.

If artificial supply of material is required further eastwards, then the east beach groyne would need to be shortened until material will fill up the bay and start to bypass the structure. This is likely therefore to form a beach in front of the adjoining cliffs and reduce the erosion of these.

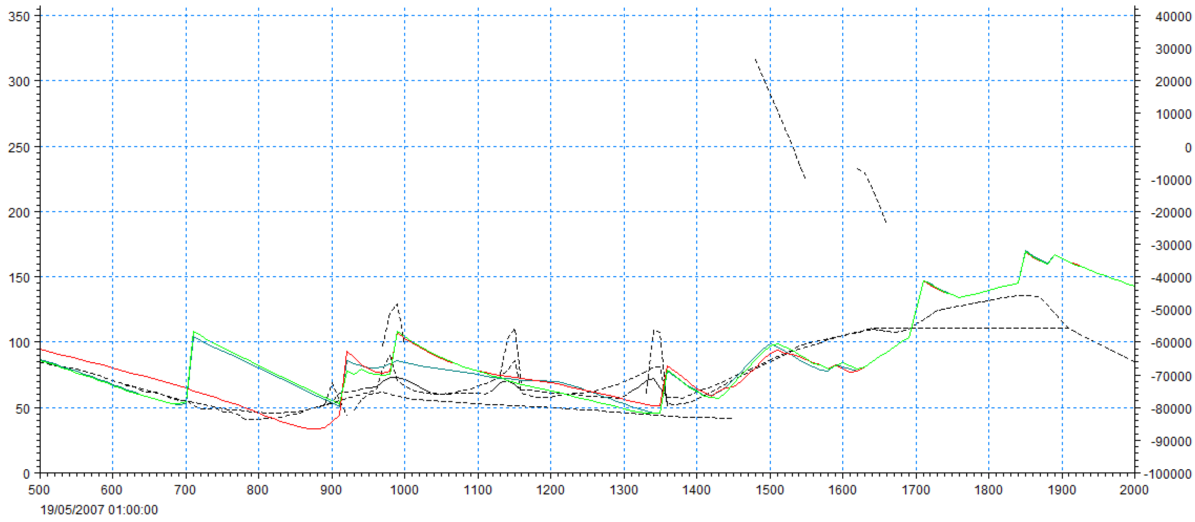

Figure 4-13: Graph showing influence of groyne length

Figure 4-13 show the difference in groyne length on shoreline position. The dark green represents a long groyne of 80m in length, the light green represented a long groyne of 110m in length and the red line represents a short groyne of 60m in length. The results indicate that there is little difference and therefore little benefit of having a long groyne of 110m in length and that a short groyne of 60m in length fails to hold sufficient beach material with the majority of material bypassing to the east.

4.2 Wave overtopping modelling

The preferred option along the Sidmouth Town frontage was tested and refined using the AMAZON wave overtopping model. 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 of this indicated that, although wave overtopping reduced in comparison to the 'Do Nothing' scenario, it was not reduced significantly to provide adequate safety to pedestrians or to reduce flood risk. As a result, the option of raising the existing splash wall was considered as a way of reducing wave overtopping reaching the road. Table 4 1 presents the wave overtopping results for the design beach and splash wall option considered along Sidmouth Town frontage for present day and
Table 4 2 in 2117 with climate change.

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

Return Period	Frontage 1		Frontage 2		Frontage 3		Frontage 4		Frontage 5	
	Pre-scheme	**Post-scheme**	**Pre-scheme**	**Post-scheme**	**Pre-scheme**	**Post-scheme**	**Pre-scheme**	**Post-scheme**	**Pre-scheme**	**Post-scheme**
1	0.26	0.00	0.14	0.00	0.20	0.00	0.30	0.00	3.01	0.00
10	0.97	0.00	0.15	0.00	1.17	0.00	0.52	0.00	4.35	0.00
20	1.63	0.00	0.42	0.00	1.82	0.00	0.78	0.00	5.50	0.00
50	3.15	0.00	1.12	0.00	3.22	0.00	1.53	0.00	7.22	0.00
75	3.49	0.00	1.56	0.00	3.95	0.00	1.95	0.00	7.99	0.00
100	4.57	0.00	2.08	0.00	4.26	1.73	2.20	0.17	9.02	0.00
200	4.84	0.00	2.34	0.00	4.53	2.17	2.69	0.18	10.03	1.29
1000	8.52	0.00	5.00	0.00	8.18	4.48	6.00	1.32	15.98	1.35

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

Return Period	Frontage 1		Frontage 2		Frontage 3		Frontage 4		Frontage 5	
	Pre-scheme	**Post-scheme**	**Pre-scheme**	**Post-scheme**	**Pre-scheme**	**Post-scheme**	**Pre-scheme**	**Post-scheme**	**Pre-scheme**	**Post-scheme**
1	5.00	0.00	1.48	0.00	8.57	0.00	1.85	0.00	10.92	0.00
10	8.89	0.00	5.91	0.00	21.54	0.00	5.07	0.00	18.13	0.00
20	10.34	0.00	7.67	0.00	21.89	0.00	5.11	0.00	18.96	0.00
50	12.57	0.00	11.68	0.00	25.97	3.23	9.44	0.00	24.45	1.36
75	13.00	0.00	13.54	0.00	29.84	4.38	10.68	0.00	23.51	3.55
100	13.41	0.00	14.36	0.00	30.64	5.14	14.38	3.47	30.55	4.68
200	22.87	0.00	15.56	0.00	36.52	6.06	15.55	3.57	31.54	5.84
1000	27.34	0.00	20.95	0.00	51.46	12.90	25.87	7.52	47.30	12.04

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