Weymouth Coastal model updates

Final Report

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This report describes work commissioned by Oran Balazs, on behalf of Dorset Council, by an email from Chris Osbourne on the 13 December 2021. Dorset Council's representative for the contract was Oran Balazs. Bethany Shenton and Lewis Brazell of JBA Consulting carried out this work.

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Purpose

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Abbreviations

AEP	Annual Exceedance Probability
BODC	British Oceanographic Data Centre

CCO	Channel Coastal Observatory
CFB	Coastal Flood Boundary
DTM	Digital Terrain model
FDP	Third joint probability scenario
FDT	Fluvially dominated
FRIS	Flood Reconnaissance Information System
LIDAR	Light Detection and Ranging
NaFRA	National Flood Risk Assessment
NCLD	National Coastal Loads Database
RCP	Representative Concentration Pathway
SFRA	Strategic Flood Risk Assessment
SoN	State of the Nation
SWAN	Simulating Waves Nearshore
TDT	Tidally dominated
UKCP	UK Climate Projections

1 Introduction

This report summarises the updates to the Weymouth coastal and fluvial model (JBA 2019) to map the flood risk for the Level 2 Strategic Flood Risk Assessment (SFRA). The report covers offshore statistics, wave transformation, wave overtopping and flood inundation. The modelling focuses on the area of the River Wey through Weymouth, the town centre and the Lodmoor Nature Reserve, as shown in Figure 1-1.

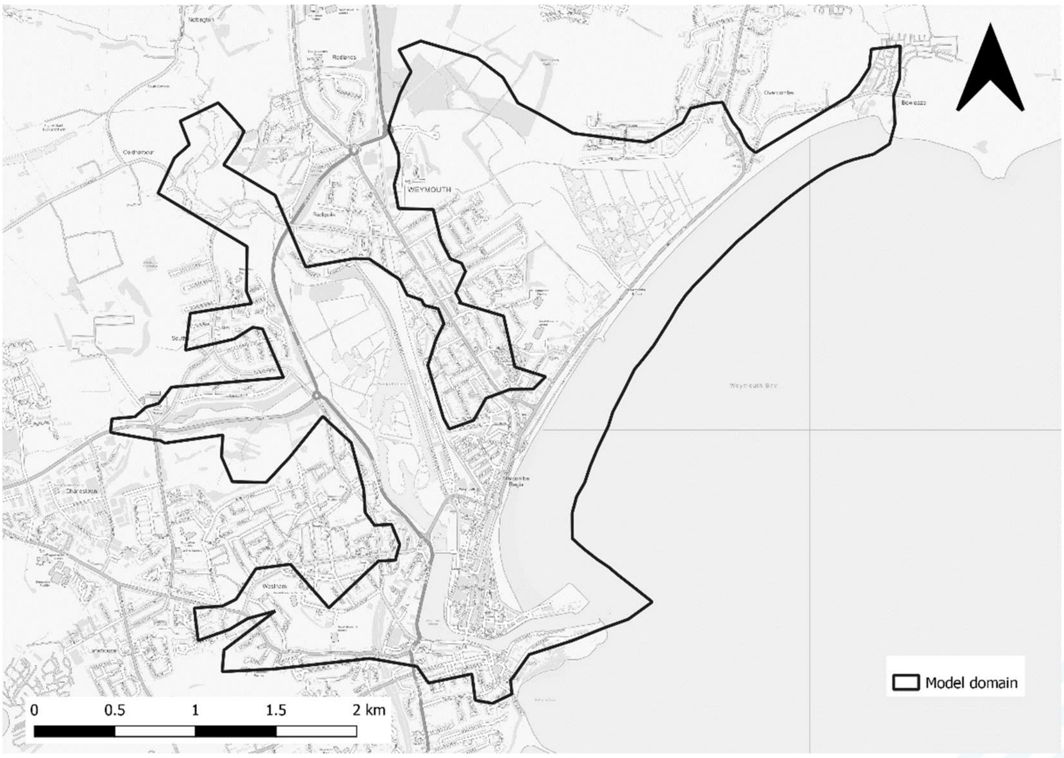

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Figure 1-1: Weymouth coastal and fluvial model domain.

1.1 Coastal flood risk drivers

Coastal flood inundation is caused by extreme still water level (tidal) and wave overtopping discharge rates (coastal). Although still water levels provide the background conditions resulting in a flood event, a considerable proportion of coastal flooding can be attributed to waves overtopping defences. An accurate representation of the effects of wave overtopping is therefore crucial.

Wave overtopping must be calculated separately as there is no single model capable of simulating both still water flooding and wave overtopping. The method outlined in the State of the Nation (SoN) National Flood Risk Assessment, NaFRA (2014) project was used to calculate the wave overtopping volumes. Wave overtopping volumes were calculated at seven defences along the esplanade at Weymouth. A full description of the methodology used for the State of the Nation project update can be read in 'The State of the Nation Flood Risk Analysis, Coastal Boundary Conditions' report[1]. The following sections provide an overview of the assumptions that were made for the wave overtopping process, and the methodology followed specifically for this study.

1.1.1 Assumptions

The behaviour of waves in the nearshore and surf zone is highly complex and the subject of research. As such, several assumptions were made to represent wave overtopping at the

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1 Environment Agency 2015 'State of the Nation Flood Risk Analysis, Coastal Boundary Conditions' MCR5289-RT025-R02-00

model boundary for the appropriate design conditions. Firstly, for the purposes of a flood inundation model, it is unnecessary to incorporate details of individual wave processes but rather to represent average wave conditions, such as the significant wave height and the worst-case overtopping conditions at each defence for each event.

The most important assumption is that wave conditions remain consistent throughout the tidal cycle. This approach is appropriate for modelling design events as it simulates the conditions at the boundary of the model where extreme tides, surge levels and waves occur at the same time. Changes in overtopping are therefore a result of the depth limiting effects of the changing water level, rather than any changes in the incident wave conditions.

Wave overtopping was calculated over a 24-hour period in line with the Environment Agency Flood and Coastal Risk Management Modelling Guidance[2] which recommends modelling wave action over a 12 to 24-hour period, as the waves will diminish as the storm passes and the wind changes direction.

1.2 Method

The method for calculating wave overtopping is a multistage process, which can be broken down into four key stages;

- Defence schematisation
- Wave transformation modelling
- Wave model emulation
- Wave overtopping

1.2.1 Defence schematisation

This study has modelled flood defence structures situated along the open coastal frontage of Weymouth. The coastline was initially separated into six individual stretches that were theorised as having a specific overtopping risk. Following the calibration, profile 1 was split in two to give a seventh profile. Schematisation of the defence profile is principally required for calculating the wave overtopping discharge rate during the wave overtopping calculations stage. However, it is also used at the wave transformation stage, as it defines the locations of the required nearshore wave conditions.

1.2.2 Wave transformation modelling

To transform offshore wave conditions into the nearshore, wave transformation modelling is required. In this study, there are separate locations each with differing risks in terms of wave exposure and bathymetric features that could impact on the incoming waves. To account for the differences, the approach to the wave transformation modelling was tailored to each wave overtopping profile using 1D surf zone models (five 1D wave models were used to calculate the wave data for the seven overtopping sections). The Simulating Waves Nearshore (SWAN) modelling package was used for all locations. SWAN is a third-generation wave model incorporating complex physics for the description of nearshore processes. It is an open-source package used widely for research and commercial applications, developed by internationally recognised experts at the Delft University of Technology[3]. The wave model was used to simulate 10,000-year synthetic storm event sets from the State of the Nation (SoN) project. This event set provides pre-calculated nearshore water level and wave conditions for 435,000 individual storm events, that represent 10,000-years of event data. All 435,000 events have been transformed through five individual SoN SWAN 1D models. A

2 2 Environment Agency 2010. `Computational modelling to assess flood and coastal risk` Doc No 379_05 Version 2

3 Delf University of Technology, 2015 ‘SWAN User manual, SWAN cycle III version 41.10’

full description of the methodology used for the SoN Project can be read in `The State of the Nation Flood Risk Analysis'[4].

1.2.3 Wave overtopping calculations

The method outlined in the European Overtopping Manual (EurOtop) was used to calculate the wave overtopping discharges for this project. The manual includes methods and guidelines on the prediction of wave overtopping at complex structures. For this study, the Neural Network I methodology was used. Neural Network requires a set of input parameters for the calculations which comprise the still water level at the defence toe (including wave set-up), the incident wave conditions, and the defence profile geometry. The nearshore wave conditions calculated from the wave transformation modelling, were run through the Neural Network tool. The resulting wave overtopping discharges were analysed and assigned an event probability. Each defence is treated independently, therefore, a storm that results in a 0.5% AEP event at one defence may result in more or less severe flooding at an adjacent defence. The orientation of the defence and the defence geometry in relation to the tide and wave conditions controls the amount of wave overtopping. From the 10,000-year event set, the data is analysed and the wave conditions and overtopping discharges for each event of interest are extracted. These are then used to calculate wave overtopping over a tidal cycle to produce a time series of wave overtopping for inclusion in the flood inundation modelling.

A time series of wave overtopping volumes for each event were then derived. These events are the 50, 10, 4, 2.5, 2, 3.3, 1, 0.5 and 0.1 % Annual Exceedance Probability (AEP) events. Wave overtopping volumes were also calculated for climate change events for the 50, 10, 4, 2.5, 2, 3.3, 1, 0.5 and 0.1 AEP for 2098 and 2138 per UKCP18 estimation for change using the Representative Concentration Pathway (RCP) 8.5 at the 95th percentile.

2 Flood history

There is limited reporting of coastal flooding along the open-coast frontage of Weymouth (Weymouth Beach Management Plan, Jacobs, July 2019). However, according to the Environment Agency's Flood Reconnaissance Information System (FRIS) significant incidents are recorded to have occurred in and around the town centre in 1955, 1977, 1979, 1983 and 2008 from a mixture of tidal, fluvial and surface water sources. Given the location of the town centre, tidal flooding represents the main flood risk combined with the effects of wave overtopping (Weymouth Flood Risk Strategy Report, EA, June 2010). Weymouth town centre is particularly exposed to tidal flooding due to significant proportions of the area being considered low lying, in particular sections of the Park District varying between 0.5m and 1m in elevation.

In Weymouth there were no recorded flood outlines or historical flood records available from the Environment Agency, but there are photos and videos of flooding online. Photos and videos of flood risk are available from events on 10 March 2008, 14 December 2012, 3 January 2014, 05 February 2014, 14 February 2014 and 12 March 2020, all mainly focusing on still water flooding around the harbour in the Commercial Road area. The events of the 5 and 14 February 2014 both also show some evidence of wave overtopping along the seafront. Figure 2-1 shows a screenshot taken from a Channel 4 news video (https://www.channel4.com/news/storm-uk-wind-tide-sea-weather-flood-weymouth-dorset-video), showing wave overtopping during the 05 February 2014 event.

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4 Environment Agency 2015 `State of the Nation Flood Risk Analysis, Coastal Boundary Conditions` MCR5289-RT025-R02-00

Figure 2-1: Spray wave overtopping from 5th February 2014.

High tide levels in March 2008 resulted in a breach of defences and the flooding of the road beneath Town Bridge (Weymouth Flood Risk Strategy Report, EA, June 2010), Figure 2-2.

Figure 2-2: Still water flooding under Town Bridge in March 2008.

At Preston beach flooding was an issue along the B3155 and into the Lodmoor Nature Reserve before a flood defence scheme was finished in 1996. The flood defence scheme included a raised concrete promenade and rear defence wall built on top of the existing defences, and fronted by a re-nourished beach and rock armour to protect the defence toe. After the 2014 winter storms much of the shingle beach had been eroded and work was undertaken to place 4,000 tonnes of rock under the shingle to maintain the crest width and renourish the beach. Following further storm damage, the rock armour was reinstated in January 2022. With the improved defences the wave overtopping has been limited to shingle being thrown onto the promenade (Figure 2-3) but no evidence of overtopping onto the road.

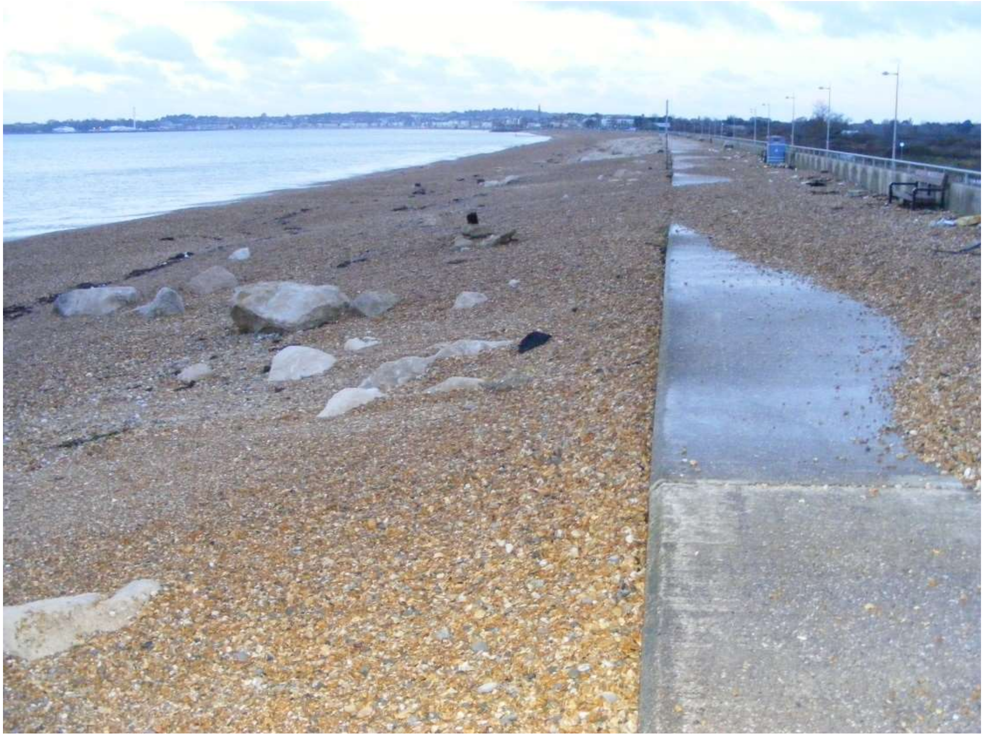

Figure 2-3: Shingle on the promenade of the Preston Beach defences in 2014

3 Multivariate statistical analysis of coastal extremes

When considering extreme coastal events, it is important to assess the likelihood of conditions occurring simultaneously. Coastal surges, winds and waves are generated by the same atmospheric processes, as such there are relationships between these variables. Extreme waves occurring during a high sea level pose a far greater coastal flood risk than if they occurred during lower sea levels. The probability of concurrent processes needs to be carefully assessed through dependence relationships between each variable, in addition to independent processes such as astronomical tides.

SoN data was available to provide the boundary conditions for the SWAN 1D wave transformation models. These data consist of nearshore water level and wave conditions (wave height, period and direction) and are taken from the National Coastal Loading Database (NCLD). The wave data in the NCLD were calculated using offshore 2D SWAN wave models to transform the offshore waves into the nearshore. Three NCLD datasets were used as boundary conditions to the five 1D wave models. Adjustments were made to the sea-levels, wind speeds and wave heights to account for climate change projections for the present-day and future scenarios. Further details of climate change adjustments and the wave transformation modelling can be found in Section 5.

4 Schematisation of coastal defence structures

Flood defence structures along the coastline of Weymouth were analysed. These defences were divided into seven separate stretches that were theorised as having a specific wave overtopping risk. Schematisation of the defence profile is principally required for calculating the wave overtopping discharge rate during the wave overtopping calculations stage. However, it is also used at the wave transformation and emulation stage, as it defines the locations of where the nearshore wave conditions are required.

This study has applied the Neural Network I methodology to calculate the wave overtopping discharge rates at each defence, the details of which are outlined in the European Overtopping Manual (EurOtop). The Neural Network One tool in EurOtop was developed by the European CLASH programme. EurOtop uses a large database of results from physical modelling tests to derive a solution based on complex defence profiles. The schematisations of the seven defences describe the components of the profile using the parameters required by the Neural Network tool. A total of 18 parameters are used to describe the defence structure, the defence toe level, defence crest, defence slope, roughness and orientation.

Detailed cross-sectional data were used to accurately parameterise the defence profiles. Data used included:

- 1m Digital Terrain Model (DTM) based on LIDAR
- Cross-sectional beach surveys by the Channel Coastal Observatory (CCO)
- Data sources including as-built drawings, photographs and notes taken during site visits.

Analysis of this defence cross-section data was carried out to parameterise a preliminary model schematisation for each defence section. A total of 18 parameters are required as input for the Neural Network tool. These include: crest height (Rc); armour height (Ac); armour width (Gc); berm elevation (hb); berm width (B); upper slope (αu); lower slope (αd); and roughness (γf). A typical beach profile and the parameters required for the schematisation of a Neural Network profile are summarised in Figure 4-1: .

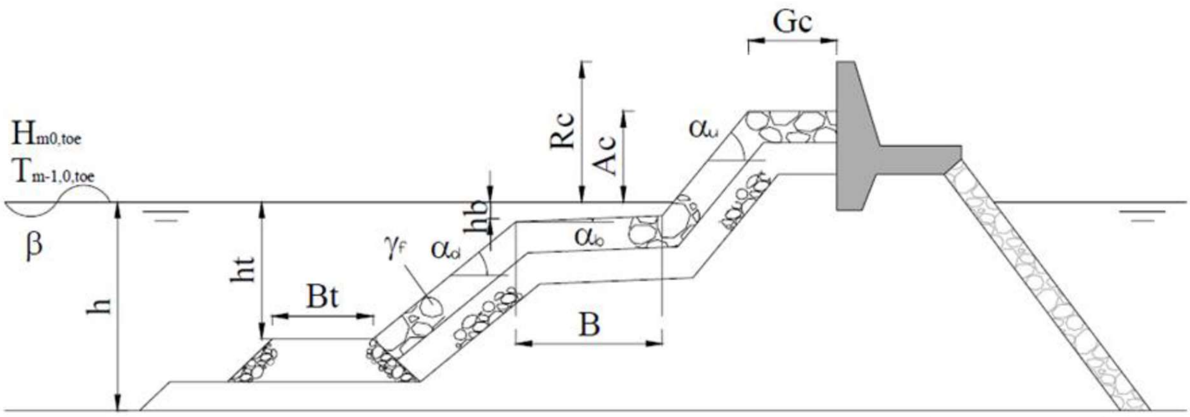

Figure 4-1: Schematisations of a typical beach profile for analysis using the Neural Network overtopping tool (Source: EurOtop manual)

The defence profile schematisation can be separated into three main sections; the upper section or crest, middle section or berm and the lower section or toe. It is important to note that the berm should be set at an elevation that is within the range of the water level plus ±1.5 multiplied by the wave height at the toe of the structure, so that it is kept within range of the Neural Network tool. If the berm was more than 1.5 multiplied by the wave height lower than the water level, then it would become the toe of the structure. In some cases, usually a simple sloping structure or vertical wall, no single point is representative of the berm. In this instance the berm is set to 'floating' (a value of -9) and in the wave overtopping calculations the berm level is set to equal the water level.

A defence schematisation QA sheet is supplied alongside this report which gives details on each defence schematisation and the resultant input parameters for Neural Networks. Examples of the baseline defence schematisations are shown in Figure 4-2: and Figure 4-3: . Another limitation of the wave model parameterisation is that the lower slope can be no shallower than 1 in 10, therefore, for profile one the lower slope was set to 1 in 10, even though the actual beach is much shallower. For profile one the toe level was set to -0.91mAOD and this resulted in just over 11,000 events predicting overtopping, with overtopping starting at around a 100% AEP (once a year). Profile seven was split off from profile one to further reduce the wave overtopping in the more sheltered corner of the bay. Profile seven used a higher toe level of 0.8mAOD and this resulted in only 215 events predicting overtopping from the 10,000-year event set, with overtopping starting at the 2% AEP (1 in 50-year return period). The toe level could be set to the base of the wall at

2.5mAOD, but at that level, all events with a water level below 2.5mAOD would be ignored because the toe of the defence is dry. With a toe level of 2.5mAOD there would only be 12 events that were considered from the 10,000-year event set, and this would ignore potentially big events, such as those with a 2.3mAOD water level and 1.85m wave height. The overtopping models are therefore sensitive to the location of the toe level used.

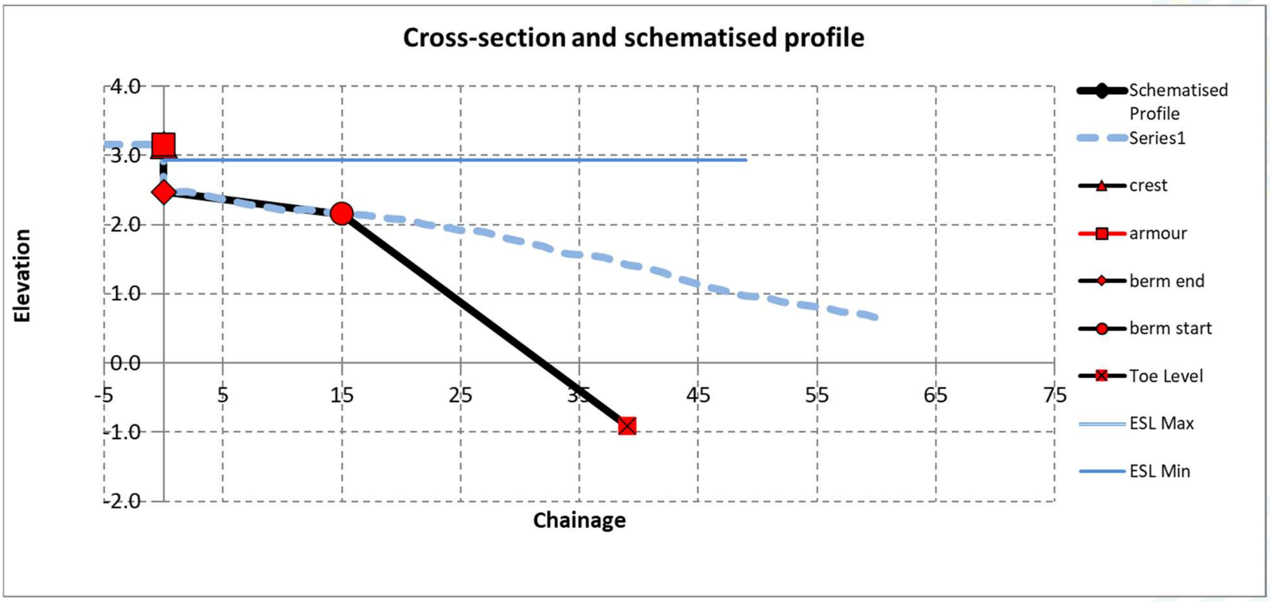

Figure 4-2: Neural Network Schematisation, Example Profile 1, Baseline Scenario

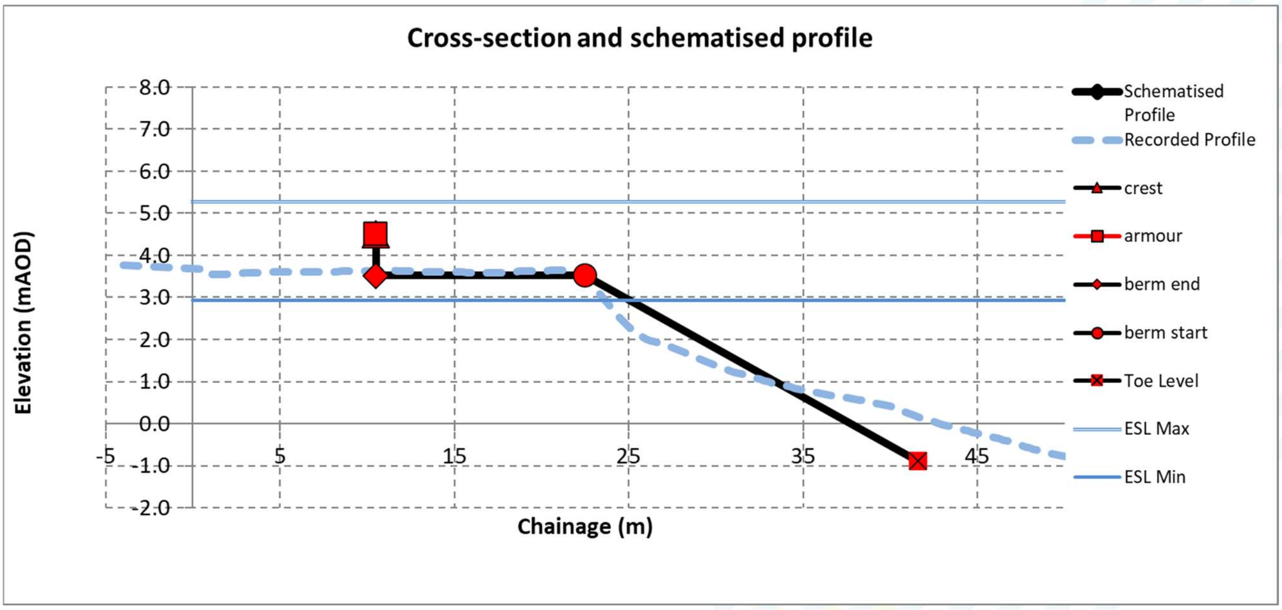

Figure 4-3: Neural Network Schematisation, Example Profile 4, Baseline Scenario

The location of the seven wave overtopping profiles is displayed in Figure 4-4.

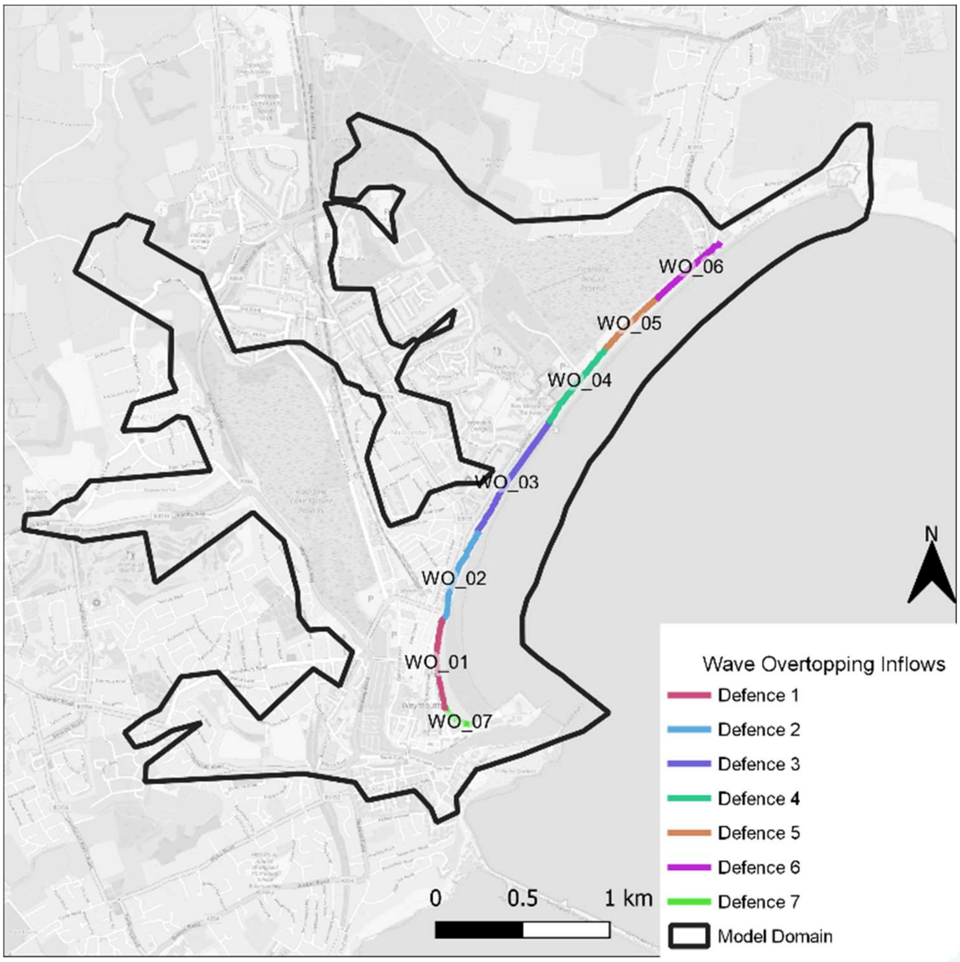

Figure 4-4: Wave overtopping profile locations (profile 7 was split from profile 1 after model calibration).

4.1 Undefended wave overtopping profiles

For the undefended scenario, the wave overtopping schematisations were amended to represent the removal of the defences. For the purposes of this study, only sections of coast with raised defences were amended for the undefended scenarios, for example, the Esplanade remains the same for both the baseline and undefended scenarios. An example of an undefended schematisation is provided in Figure 4-5: . Further details of the undefended defence schematisations, and the data used are summarised in the schematisation sheets.

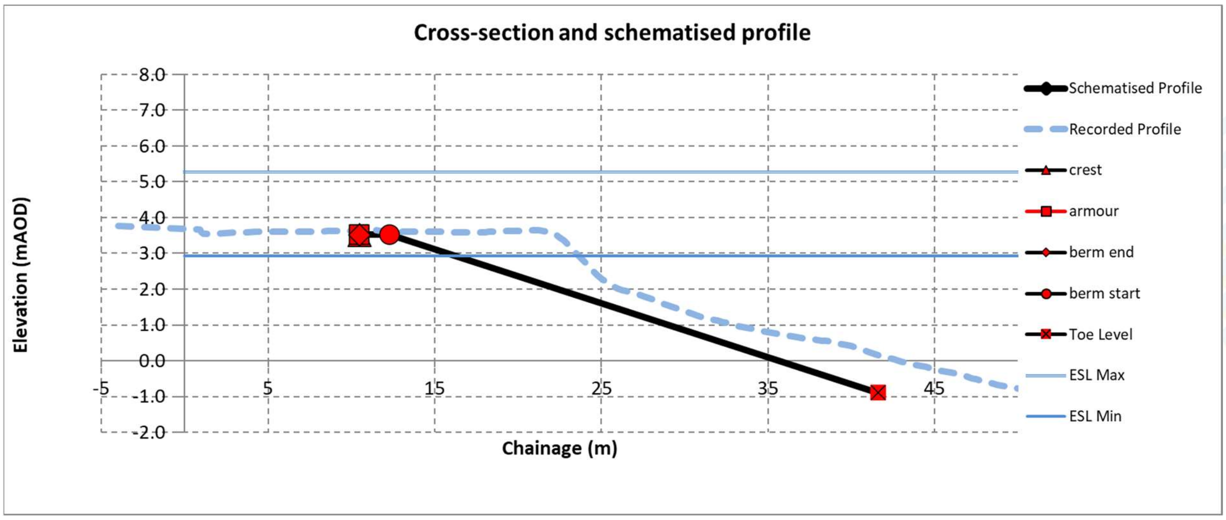

Figure 4-5: Neural Network Schematisation, Example Profile 4, Undefended Scenario (compare to Figure 4.3 with defence).

4.1.1 Eroded beach wave overtopping profiles

For the eroded beach scenario, the wave overtopping schematisations were amended to represent the erosion of the beach fronting the esplanade and Preston beach. To schematise these defences, a minimum profile was extracted, taking account of available CCO profile data including post-storm profiles. The minimum profile was based on the minimum elevation at each survey point from all surveyed profiles, rather than the lowest of all individual profiles. An example of an undefended schematisation is provided in Figure 4-6: .

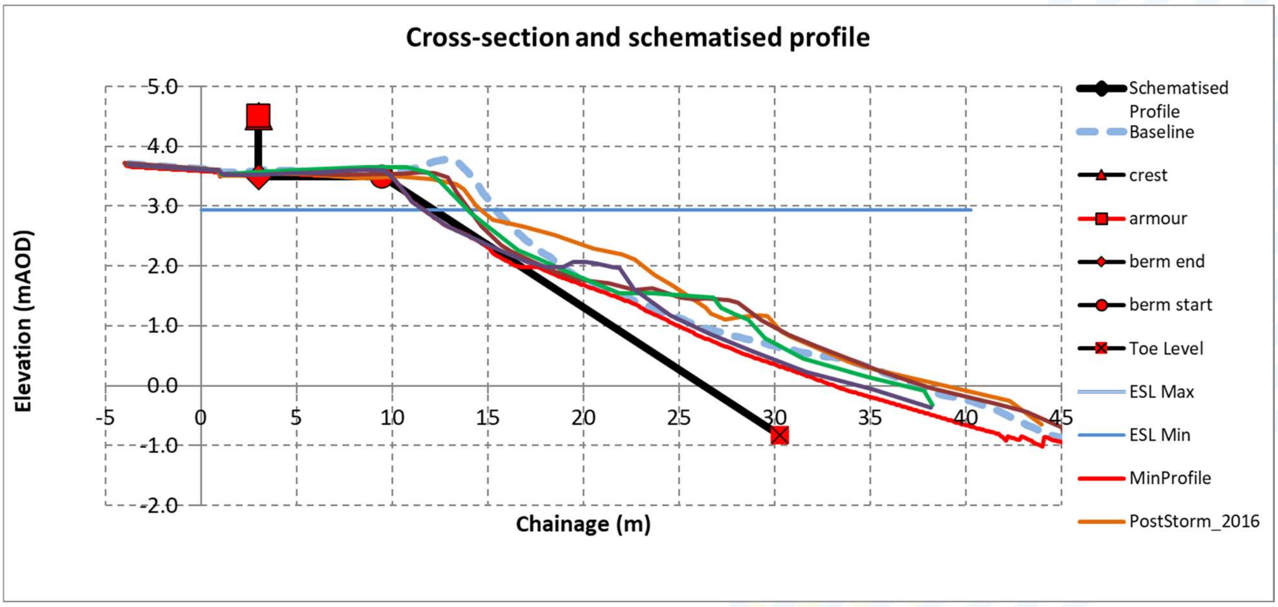

Figure 4-6: Neural Network Schematisation, Example Profile 5, Eroded Beach Scenario

4.1.2 Future defence wave overtopping profiles

For the future defence scenario, the wave overtopping schematisations were amended to represent improvements to the esplanade. This comprises of; an uplift of the Esplanade level by 0.5m and the construction of a set-back wall 1m high at the back of the Esplanade but no change to the beach profile. An example of the future defence schematisations is provided in Figure 4-7: .

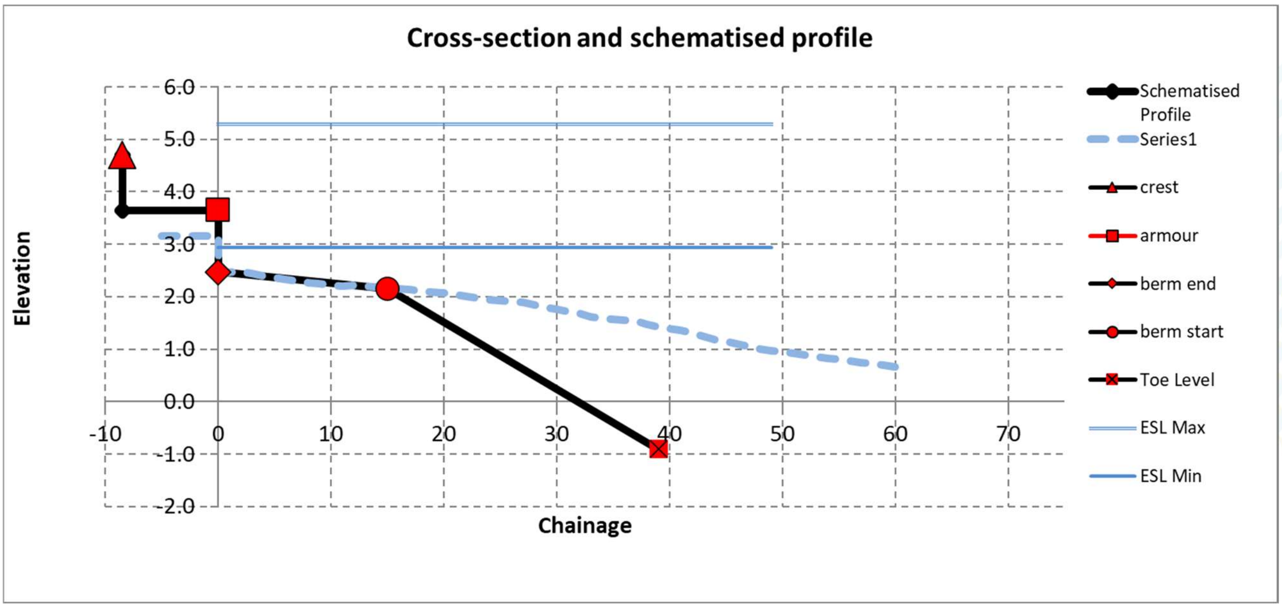

Figure 4-7: Neural Network Schematisation, Example Profile 1, Future Defence Scenario.

5 Wave Transformation modelling

Five existing SoN 1D wave transformation models were used for this study. They were used to transform waves from the nearshore points to the defence toe for the seven individual defence sections. These results were used to calculate wave overtopping discharges.

5.1 Modelling approach

This project uses five individual SoN SWAN 1D wave transformation models. Nearshore water level and wave conditions, taken from the NCLD database are transformed to seven defence toes. The NCLD datasets comprise of 'extremes' and 'lower peaks' and provide the following conditions for the SWAN 1D modelling; water level, wave height, wave period (Te and Tp), wave direction, directional spreading and frequency spreading. Table 5-1: summarises the SWAN 1D models and corresponding NCLD dataset.

Table 5-1: Selected SWAN 1D transect IDs and corresponding Nearshore Point ID

Catchment	Transect ID	Nearshore Point ID	Sea bed elevation (mAOD) of nearshore point	Used for wave overtopping profile
4400	152	1083	-5.78	WO4
4400	154	1082	-6.96	WO3
4400	156	1082	-6.96	WO1, WO7
4400	158	1082	-6.96	WO2
4400	260	1084	-5.82	WO5, WO6

The locations of the SWAN1D wave transformation model transects, the nearshore NCLD points and the wave overtopping inflow lines are shown in Figure 5-1.

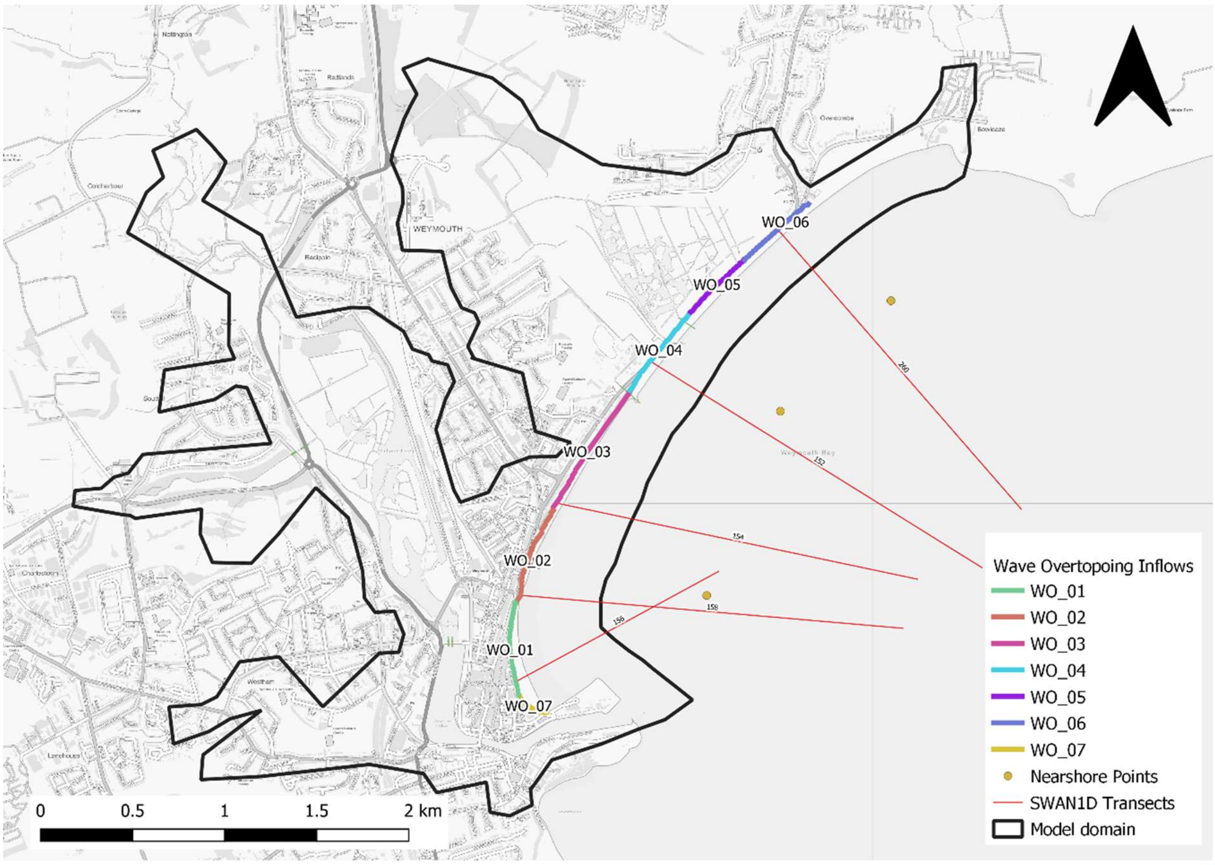

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Figure 5-1: SWAN1D wave model transects, nearshore NCLD points and wave overtopping inflow lines.

5.2 Boundary conditions

Each of the five SWAN 1D models were driven by water level and wave conditions from a corresponding NCLD event set, as described in Section 5.1. The NCLD event set were uplifted to present-day (2022) and for the climate change epochs (2098 and 2138) using UKCP18 RCP 8.5 95th percentile uplifts. The climate change adjustments are summarised below in Table 5-2: .

Table 5-2: UKCP18 RCP 8.5 95th percentile climate change uplifts

Epoch	Water Level (m)	Wave height (%)
2014 to 2022	0.03	0
2022 to 2098	0.98	10
2022 to 2138	1.71	10

5.3 Toe Depths

The wave results will be extracted in shallow water at the toe of the defence structure. It is important that the toe depths in the 1D wave transformation models are the same as the toe depths in the schematised wave overtopping defence profile. Checks were made for each model to confirm that the output points were within 0.1m of the toe depths.

5.4 Calibration and validation

As the SoN SWAN 1D models are already calibrated and validation and the models are being used with no updates, no further calibration or validation was undertaken as part of this project.

6 Wave overtopping calculations

6.1 Method

The wave overtopping estimates for this study were calculated using the methods outlined within the industry standard EurOtop manual[5]. The manual outlines various methods and guidelines for the prediction of wave overtopping for different structure types. For this study, the Neural Network I approach was used. This method allows for the rapid assessment of complex multi-component defence structures, which are characteristic of many of the defences. Although the Neural Network II tool is a more recent model, it was not applied as previous experience has shown the online executable to be unreliable when calculating overtopping for large datasets such as the 10,000-year Monte Carlo sample. The Neural Network I tool requires the following input data to derive a mean wave overtopping discharge rate:

- Nearshore wave conditions at the toe of a defence structure
- Defence geometry
- Still water level

Wave overtopping discharges were calculated along the coastal frontage at seven key sites along the Weymouth esplanade. The wave overtopping method calculates a wave overtopping discharge, quantified by the parameter 'q', in $m^3/s/m$. This method can be described as follows.

The output from the SWAN 1D transformations (present-day and climate change) were used as inputs in the Neural Network overtopping prediction tool, along with the still water level and defence schematisation parameters, in order to derive a wave overtopping discharge associated with each event in the SoN 10,000-year offshore dataset. Each defence is treated independently, therefore, a storm that results in a 0.5% AEP event at one defence may result in more or less severe flooding at an adjacent defence. The orientation of the defence and the defence geometry in relation to the tide and wave conditions controls the amount of wave overtopping.

The wave overtopping rates derived for each event in the SoN 10,000-year offshore dataset at each defence toe were ranked, and assigned a cumulative probability via the formula:

$$p_i = \frac{i}{n+1}$$

The pi is the cumulative probability assigned to the ith smallest overtopping rate. The n is the overtopping rate. The cumulative probability values assigned to each event were then used to derive empirical return periods Ti using the formula:

$$T_i = \frac{n_y}{n(1-p_i)}$$

The ny is the number of years of simulation, which is 10,000. Once ranked, the wave and water level conditions could then be extracted for each desired AEP event, the largest being the overtopping rate associated with the 0.01% AEP event.

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5 EurOtop (2010) “Wave Overtopping of Sea Defence and Related Structures: Assessment Manual”, Overtopping Course Edition, November 2010. HR Wallingford

The nearshore wave and water level data was extracted for each event of interest and the overtopping calculated over a tidal time-series. This is achieved by keeping the wave conditions constant (assuming that a large storm event will produce winds and waves that are constant over the duration of a tidal cycle) and varying the water level through a tidal time-series. This generates wave overtopping discharges that vary through time for each present-day AEP event and climate change AEP events for 2098 and 2138 per UKCP18 RCP 8.5 upper end (UE) 95th percentile scenarios. Climate change overtopping was calculated for all scenarios.

In addition to normal overtopping the EurOtop Clash database was supplemented with equations from Nadal & Hughes (2009)[8], that calculate the level of overtopping discharge when the wave interacts with the defence crest. When the still water level is at or above the defence crest this results in zero or negative freeboard. These equations adjust the volume of overtopping calculated, which prevents double counting the volume of water overtopping the defence crest level from wave action with that from still water. In the extreme, the overtopping discharge rate is reduced to zero when green water or the still water level is the sole cause of overtopping. As still water flooding is applied within the TUFLOW inundation model, it is essential that it is removed from the overtopping inflows to prevent double counting and an over-estimation of flood risk.

6.2 Undefended wave overtopping calculations

Wave overtopping discharge rates were required to assess the flood risk impact in each community should the defences completely fail or be removed. The wave conditions at the toe of the defence remain the same as for the defended scenario. The undefended overtopping discharge rates were calculated using the undefended schematisation parameters, detailed in Section 4.1.

6.3 Eroded beach wave overtopping calculations

Wave overtopping discharge rates were calculated to assess the flood risk impact in each community should the beach be eroded. The eroded beach overtopping discharge rates were calculated using the eroded beach schematisation parameters, detailed in Section 4.1.1.

6.4 Future defences wave overtopping calculations

Wave overtopping discharge rates were also required to assess the flood risk impact in each community should the esplanade defences be improved. The wave conditions at the toe of the defence remain the same as for the current defence scenario. The future defence overtopping discharge rates were calculated using the future defence schematisation parameters, detailed in Section 4.1.2.

6.5 Wave overtopping calculations

The wave transformation modelling calculated the wave conditions at the defence toe for each event in the 10,000-year offshore dataset. These wave conditions were run through the Neural Network tool to derive a wave overtopping discharge for each event in nearshore dataset. The overtopping rates were then ranked, the largest being the overtopping rate associated with the 0.01% AEP event. Once ranked, the wave and water level conditions could then be extracted for each desired AEP event and run through the Neural Network using a full tidal water level time-series; this generates the wave overtopping discharges that vary through time for each AEP.

The wave overtopping rates were then adjusted to remove the volumes associated with still water flooding, as this is also calculated in the inundation model and would result in double-counting. These adjustments can sometimes lead to inconsistencies in the overtopping rates between AEP events. In the overtopping models a single crest level is used, often taken as the lowest or average crest height along a section of coast. In some cases, the wave overtopping at a specific defence section could be discounted to very small volumes, sometimes to zero, as still water flooding is expected based on the extreme water level and average defence crest level. However, in the inundation model, the defence crest of a modelled overtopping defence section, can vary to some degree. This can lead to less still water flooding than expected across the defence section, as only short lengths of the defence will be at risk from still water flooding. Consequently, the adjustments can lead to smaller event simulations predicting more extensive flooding. To avoid inconsistencies in the wave overtopping and modelled outputs, the overtopping discharges are maximised between AEP events in the flood models, so that a larger event always has equal or more overtopping entering the model at each defence asset.

6.6 Climate change wave overtopping calculations

Wave overtopping discharge rates were also required for climate change events for the years 2098 and 2138. Sea level rise estimates were based on the latest UKCP18 sea-level change guidance using the RCP 8.5 at the 95th percentiles. The increases for sea level rise are shown in Table 5-2: . An increase of 10% was applied to all offshore wave heights and wind speeds for this study, as per the UKCP18 guidance.

The method used to calculate wave overtopping rates for the climate change epochs is a multistage process, as follows:

- The wave conditions at the defence toe were run through the Neural Network I tool
- This derived an estimated wave overtopping discharge rate that accounts for the impact of climate change for each event in the SoN 10,000-year offshore dataset.
- The overtopping discharge rates were ranked and associated with an AEP event, applying the same method outlined in Section 6.1. The wave and water level conditions were extracted for the AEP events required for the climate change scenarios.

Figure 6-1: shows the wave overtopping discharge profile for the 0.5% and 0.1% AEP events using UKCP18 projections for the year 2022.

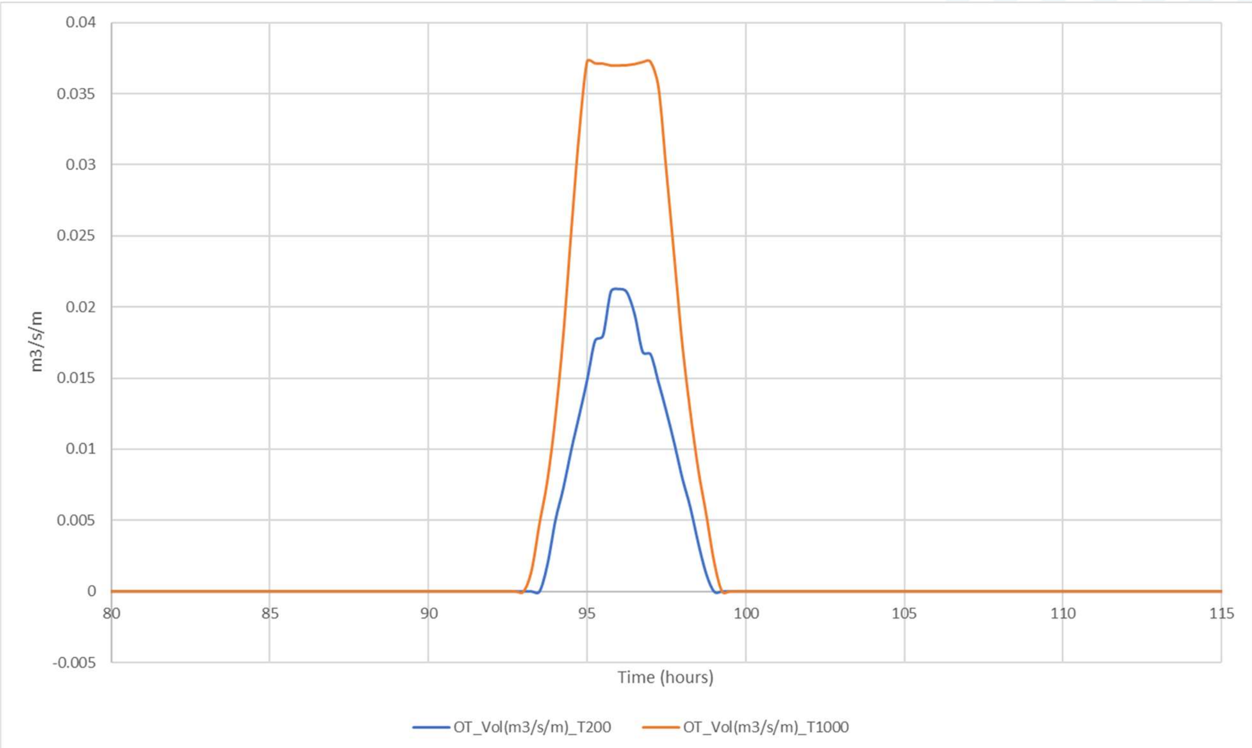

Figure 6-1: Wave overtopping volumes, profile 1, baseline scenario

6.7 Overtopping model limitations

The approaches taken in this study incorporate the most advanced methods currently available for wave overtopping modelling on the scale of the study area. However, the results are only as accurate as the input data that are used. Whilst all due care and diligence was taken to use appropriate data and methods, the results should be viewed with a margin of caution given the inherent uncertainty in the estimation of wave overtopping.

Several assumptions were made and there are elements of subjectivity throughout all stages of any modelling process. While the joint probability approaches use the most advanced statistical methods based on the Heffernan and Tawn (2004)[6] multivariate model, there is still the reliance on an extrapolation of 30-years of available data out to 10,000-years of synthetic data. In this context, even the most advanced methods are still limited by the amount and quality of the underlying data. As more data become available, the confidence in the extrapolation of the extreme values will increase.

Other assumptions and limitations include:

- The overtopping discharges assume that the wind and wave conditions remain constant throughout the duration of the tidal event.
- Offshore winds are accounted for in the offshore and surf zone wave transformation as these are included in the boundaries of the wave models. In the nearshore the local winds may also impact on wave overtopping discharge rates and the extent over which the overtopping impacts behind a defence when there is a strong onshore wind blowing spray over the defences. These local wind affects are not accounted for in the wave overtopping modelling.
- The beach profile is assumed to remain constant throughout the event. It is known that the beach profile can change during an event but this is not represented in the modelling.
- As discussed in the schematisation section, the wave overtopping rates are very sensitive to the selection of the toe level. If the toe level is taken at a low level, particularly on a shallow beach, the calculated waves could be a long way from the defence, and it could result in excessive overtopping. Conversely, selecting a high toe level rules out all events with a water level below the toe level. Testing is needed to select the most appropriate toe level.

6.8 Overtopping model validation

Evaluating the performance of wave overtopping models is difficult due to the paucity of observed wave overtopping data available for model verification. Recorded overtopping data does not exist in the way that recorded wave data do. A formal quantitative evaluation of the performance of overtopping models is therefore not generally possible and a more qualitative approach must be taken. Nevertheless, this element of the performance evaluation and optimisation process is an important step to provide confidence in the calculated overtopping rates.

As no formal overtopping data was available for Weymouth, the accuracy of the overtopping calculations was assessed by undertaking long term performance testing of the wave overtopping between February 1991 and May 2014. A frequency and threshold analysis were then performed using the derived hindcast wave overtopping estimates.

The hindcast analysis enables the identification of the largest events to have caused wave overtopping in the recent past and a comparison against the historical flood records, to confirm that overtopping is being predicted for the events where overtopping was reported. The frequency analysis check provides further information on whether the defences are overtopping as frequently or in-frequently compared against local evidence.

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6 Heffernan, J.E. and Tawn, J.A. (2004). A conditional approach for multivariate extreme values. Journal of the Royal Statistical Society: Series B (Statistical Methodology), 66(3): 497-546

Table 6-1: Largest overtopping events in hindcast period

Top events	WO1	WO2	WO3	WO4	WO5	WO6	WO7
1	27/12/2003 21:45	27/12/2003 21:45	27/12/2003 21:45	N/A	N/A	N/A	N/A
2	30/03/2013 08:45	30/03/2013 08:45	30/03/2013 08:45	N/A	N/A	N/A	N/A
3	31/07/1991 21:00	31/07/1991 21:00	N/A	N/A	N/A	N/A	N/A
4	31/12/2000 20:15	18/02/2012 15:15	N/A	N/A	N/A	N/A	N/A
5	21/12/2003 17:00	23/03/1999 22:30	N/A	N/A	N/A	N/A	N/A
6	31/03/2012 11:45	21/12/2003 17:00	N/A	N/A	N/A	N/A	N/A
7	30/04/1991 20:00	31/03/2012 11:45	N/A	N/A	N/A	N/A	N/A
8	11/05/2001 09:00	01/07/2011 19:00	N/A	N/A	N/A	N/A	N/A
9	28/01/2006 05:45	11/05/2001 09:00	N/A	N/A	N/A	N/A	N/A
10	23/03/1999 22:30	23/08/2013 20:45	N/A	N/A	N/A	N/A	N/A
Number events in hindcast period	41	11	2	0	0	0	0
Frequency	1.78	0.48	0.09	0	0	0	0

For profiles 4, 5, 6 and 7 the toe or crest levels were higher and there was no overtopping for any event in the hindcast period. For profiles 1 to 3 the largest events were 27 December 2003 followed by 30 March 2013. Profiles 1 and 2 had the highest number of events where wave overtopping was predicted over the hindcast period. There is very little evidence of wave overtopping along the seafront in Weymouth, other than images from two videos from 5 February and 14 February 2014, which both show small amounts of wave overtopping. In the hindcast testing, no wave overtopping was predicted for these two events. The hindcast data is forced with wave data from the Met Office WAVEWATCH III (WW3) model and for these events, the offshore Met Office model predicted low wave conditions, therefore, it was not possible to adjust the overtopping models to predict wave overtopping for these events.

For three events, the calculated overtopping volumes were used to provide boundary conditions for calibration of the flood inundation model. Full details of the TUFLOW model calibration can be found in Section 7.2.

Following the wave model validation and the inundation model calibration, it was identified that too much wave overtopping was being predicted at the southern end of the Esplanade. Wave overtopping profile WO_01 was split into two and a new profile (WO_07) was added. Adding WO_07 to the model reduced overtopping in this location to a more representative volume.

Table 6-2: Calculated wave overtopping rates for the baseline event

AEP Overtopping (%)	WO1 ($m^3/s/m$)	WO2 ($m^3/s/m$)	WO3 ($m^3/s/m$)	WO4 ($m^3/s/m$)	WO5 ($m^3/s/m$)	WO6 ($m^3/s/m$)	WO7 ($m^3/s/m$)
50.00	0.001	0.002	0.002	0.000	0.001	0.000	0.000
10.00	0.004	0.006	0.005	0.002	0.005	0.000	0.000
5.00	0.007	0.009	0.008	0.003	0.008	0.000	0.000
2.50	0.011	0.012	0.011	0.005	0.011	0.000	0.000
2.00	0.012	0.014	0.012	0.006	0.012	0.000	0.001
1.33	0.014	0.016	0.014	0.007	0.015	0.000	0.002
1.00	0.016	0.018	0.016	0.008	0.018	0.000	0.003
0.50	0.021	0.023	0.021	0.012	0.025	0.013	0.006
0.10	0.037	0.042	0.037	0.025	0.048	0.070	0.014

7 Flood inundation model

An existing model of Weymouth from the 2019 Weymouth inundation modelling study was re-used and updated to model the flood risk for this study.

The model is a 1D-2D ESTRY-TUFLOW model, which was updated with the latest defence information, a new LIDAR Digital Terrain Model (DTM), updated building thresholds, improvements to the representation of sluice gates and outfalls and updated boundary conditions.

7.1 Model Updates

The 2D TUFLOW inundation model was updated with the following:

- Most recent version of TUFLOW at the time of the simulations (2020-10-AB)
- Most recent version of LiDAR (2020, 1m res) – open-source LIDAR composite.
- The model domain was extended to encompass all regions of the DTM lower than 4.3mAOD elevation (the maximum modelled extreme water level value).
- The still water level boundary was updated and applied from offshore Coastal Flood Boundary (CFB) point (4734). This provides sea-level timeseries for present day (2022) and climate change scenarios (2098 and 2138). The latest UKCP18 RCP8.5 95th percentile climate projections, were used to update the sea-levels from the CFB 2017 base year.
- Inclusion of two, previously unmodelled culverts, connecting Lodmoor Nature Reserve to Weymouth Bay.
- Improvements to the representation of the culverts and sluice gates at Westham Bridge.
- Addition of a seventh wave overtopping boundary to reduce the amount of wave overtopping being predicted for the more sheltered southern end of the Esplanade.
- Improved representation of fluvial river channels within the model domain and additional fluvial inflows added for the two small watercourses that drain into the Lodmoor Nature Reserve, see Figure 7-1. The flow of the fluvial inputs was input so that the peaks of both tidal and fluvial events are aligned.

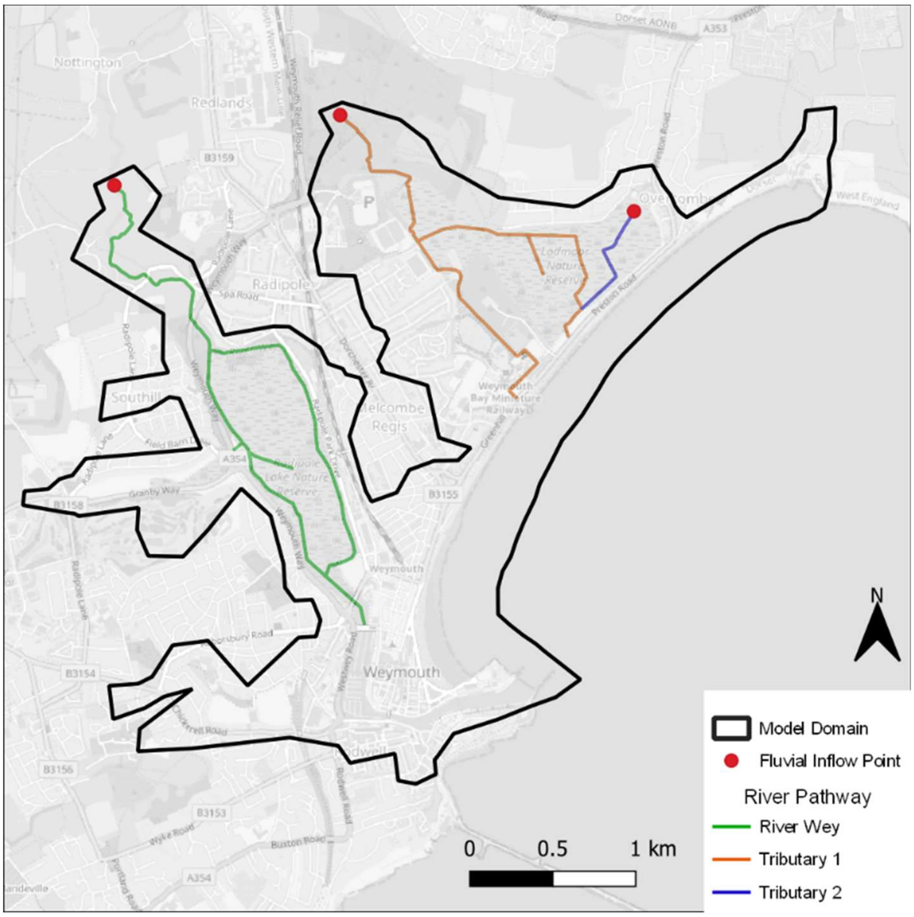

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Figure 7-1: Representation of fluvial channels and associated inflow points.

7.2 Flood inundation model calibration

Following the updates to the model and the new wave overtopping calculations, the flood inundation model was calibrated against three past events. In Weymouth there were no recorded flood outlines or historical flood records available from the Environment Agency, but there are photos and videos of flooding online. Photos and videos of flood risk are available from events on 10 March 2008, 14 December 2012, 3 January 2014, 05 February 2014, 14 February 2014 and 12 March 2020, all mainly focusing on still water flooding around the harbour in the Commercial Road area. The events of the 5 and 14 February 2014 both also show some evidence of wave overtopping along the seafront.

Three events were selected for the model calibration, two were still water events from the 10 March 2008 and the 14 February 2014. From the hindcast model testing, the event with the highest wave overtopping was the 27 December 2003, therefore this event was selected to calibrate the wave overtopping inflows. The TUFLOW model was calibrated against coastal events only. No fluvial flows were considered.

7.2.1 Still water boundary

For the three calibration events, data for the still water level boundary was extracted from the Weymouth tide gauge, provided by the British Oceanographic Data Centre (BODC). It was applied directly to model offshore water level boundary after being converted from Chart Datum to Ordnance Datum.

For each calibration, the model simulations were run for the duration of three tidal cycles, the middle cycle being representative of the peak still water level.

7.2.2 Wave overtopping boundaries

Wave overtopping of coastal defences during the 27th December 2003 event was calculated and was incorporated into the model in parallel with the water level time series.

Still water levels were provided by the design tidal graph extracted from the Weymouth tide gauge. Wave data was taken from the NCLD and transformed into the nearshore using the SWAN1D model. The overtopping models described in Section 4 and Section 6 were used to calculate the wave overtopping.

7.2.3 14th February 2014 calibration results

Figure 7-2: shows the results from the 14 February 2014 Calibration run. A video of the flooding shows water flowing north along Commercial Road, near the junction with Lower St. Alban Street, with depths of approximately 0.2m to 0.5m. The model shows flooding further south on Commercial Road but not in this location, with flood depths of 0.1m to 0.3m. The peak recorded water level at the Weymouth tide gauge was 2.00mAOD. The harbour defences alongside Commercial Road have a crest level of 2.1mAOD, rising to 2.46mAOD from the Harbour Slipway. The modelled water level does increase within the harbour from 2.00mAOD at the entrance to 2.04mAOD by Town Bridge but this is not high enough to overtop the defences. During the event there may have been increased water levels due to strong winds or small waves within the harbour that resulted in more water overtopping the defences than is shown in the model. It is also noted that the surface water drains backed up during the event and this exacerbated the flooding, but this is not represented in the modelling.

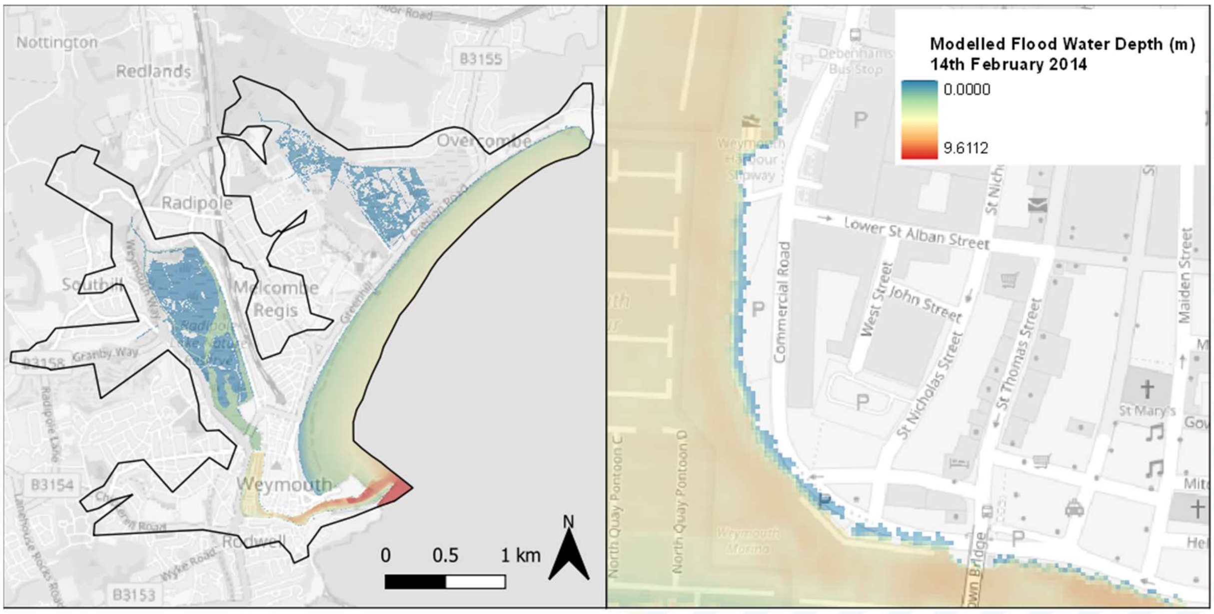

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Figure 7-2: Flood Extent, Calibration event 14th February 2014

This event, although being one of the largest on record, does not appear in the Lower Peaks dataset (which includes events that were extreme either due to water level, wave height or wind speed), which suggests that the event was underestimated in the offshore hindcast model data and the NCLD dataset. Therefore, no wave overtopping was calculated along the main Weymouth Promenade. There were images from a video showing overtopping during this event, but this could not be replicated in the model.

7.2.4 10th March 2008 calibration results

Figure 7-3: shows the associated results from the 10th March 2008 model simulation.

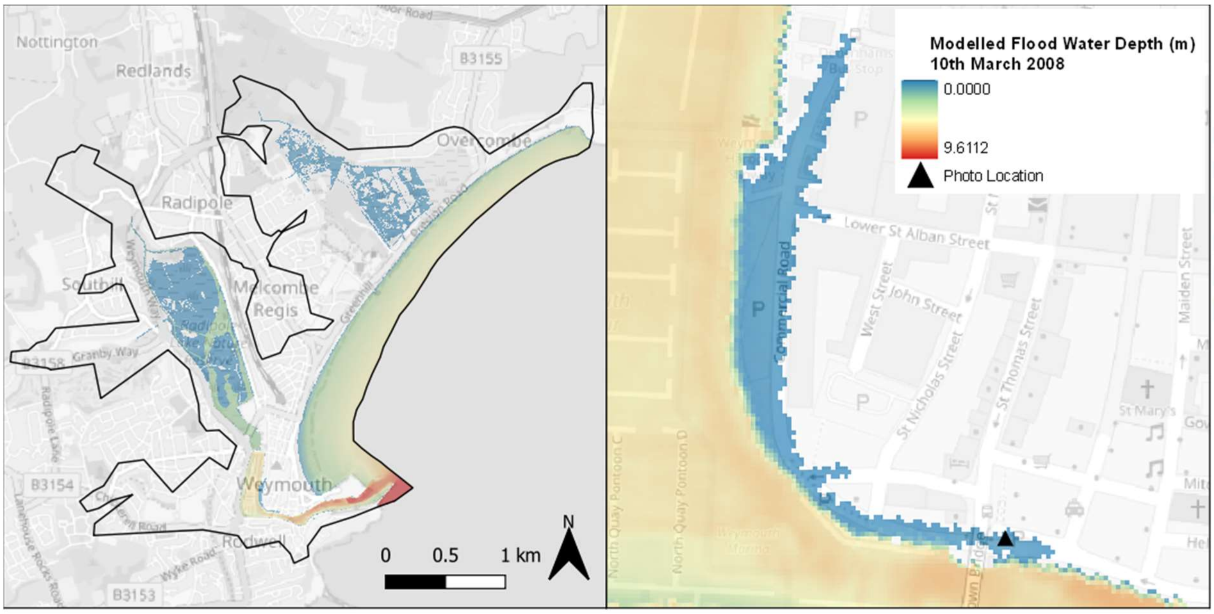

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Figure 7-3: Flood Extent, Calibration event 10th March 2008

The model results show flooding along Custom House Quay underneath the Town Bridge and up through to Cosens Quay Car Park and Commercial Road. This matches a photograph of the flooding taken during the event, Figure 7-4.

Figure 7-4: Flooding beneath Town Bridge on Custom House Quay in March 2008

The flood depths in the photo are estimated to be 0.1-0.2m deep and this matches with the modelled outputs.

7.2.5 27th December 2003 calibration results

The calibration event of the 27 December 2003 was selected as this was the event with the highest predicted wave overtopping discharges from the hindcast model testing. The flood depths from the model results are shown in Figure 7-5: .

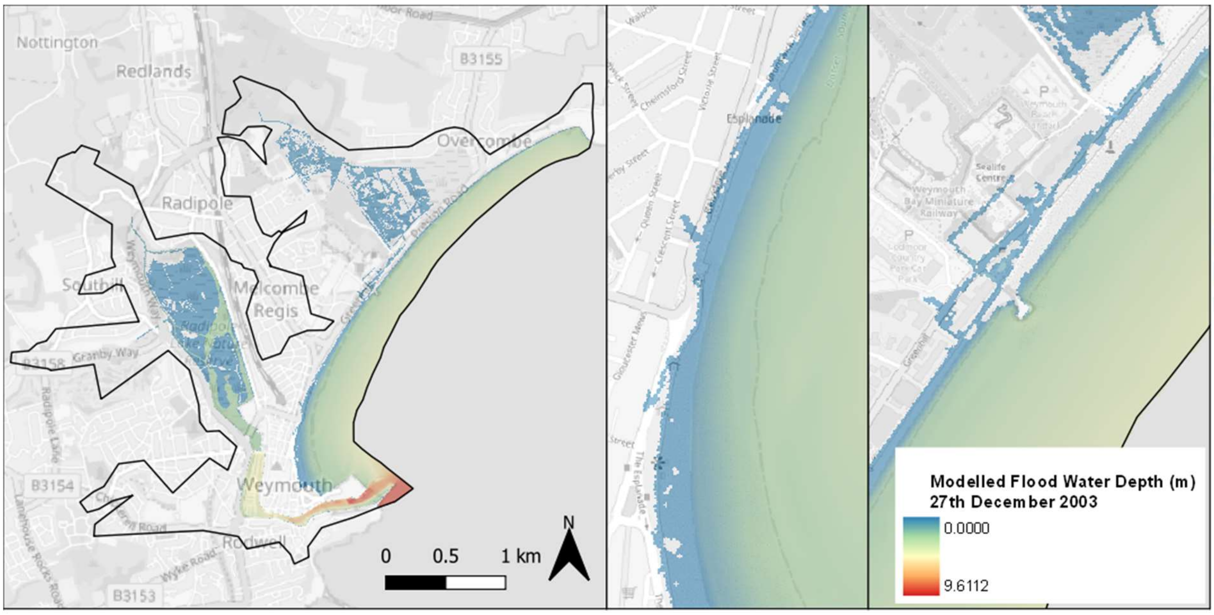

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Figure 7-5: Flood Extent, Calibration event 27th December 2003

The results show several instances of significant wave overtopping along the length of the Promenade. Most significant overtopping of defences occurs at wave overtopping profile three, located at Greenhill near the Greenhill Gardens, Tennis court and Bowls Club. Water initially overtops the main seawall, travelling into and filling, the Public Sandpits adjacent to Preston Beach. Once at capacity the sea water spills out on to Greenhill (B3155) and into the Sea Life Centre. Other instances of overtopping occur down the length of the Esplanade, with maximum flood depths of 0.13m. There are no videos or images of flooding to verify this event.

7.3 Sensitivity testing

The final model setup was tested for sensitivity to the downstream boundary conditions and to the model roughness. The model roughness was varied by ±20% and a simulation was completed for the defended 0.5% AEP event. The results showed that the model is not sensitive to the roughness, with no differences in the flood extents across most of the model (Figure 7-6). There are some minor differences in the flood extents around the Ferry terminal and the flood depths across the three simulations are within ±0.02m.

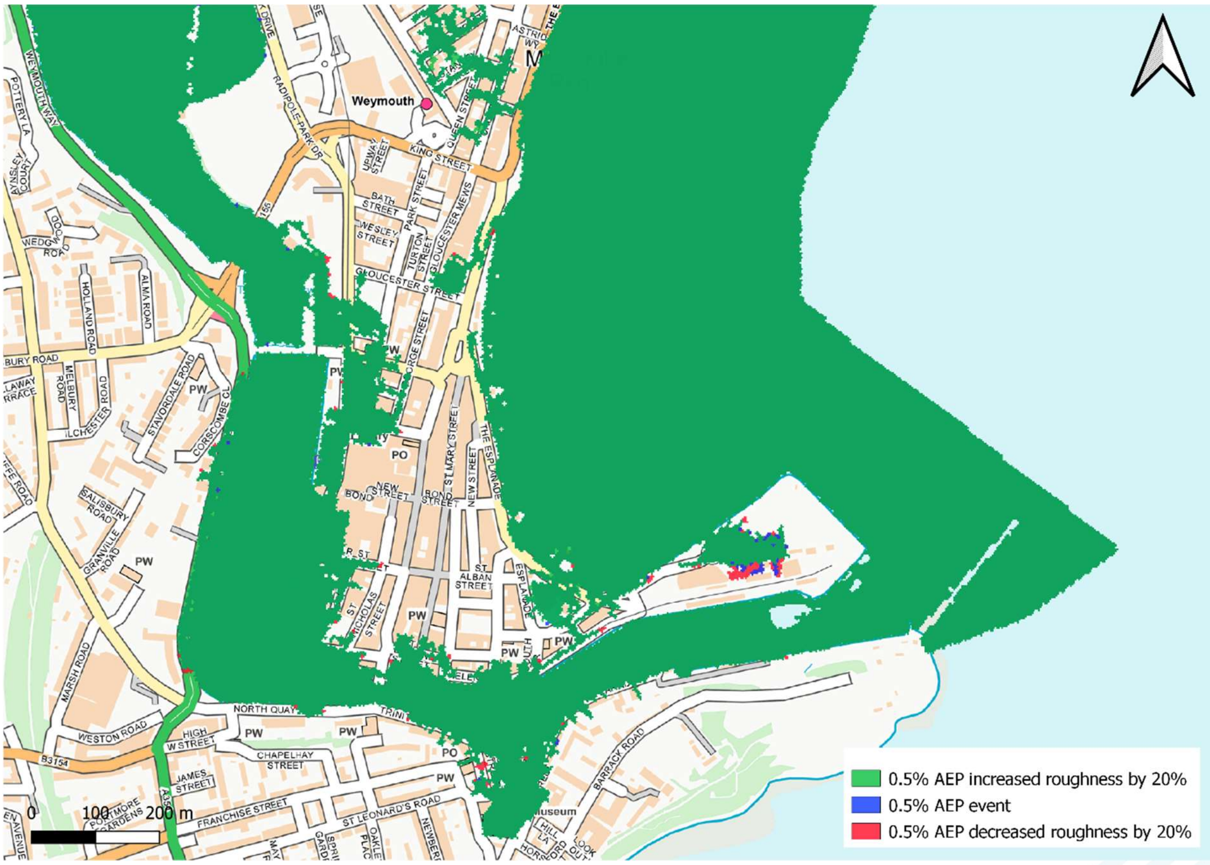

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Figure 7-6: Sensitivity of the model to ground roughness for the 0.5% AEP event

The downstream boundary conditions were tested for sensitivity by simulating the defended 0.5% AEP event using the upper and lower confidence intervals (CI) of the extreme sea levels from the CFB. Sensitivity tests were completed using the 0.5% AEP still water (SWL) scenario with no wave overtopping. The 0.5% AEP water level is 2.37mAOD, the lower CI 0.5% AEP sea-level is 2.28mAOD and the upper CI is 2.57mAOD. In comparison to the CFB levels, the lower CI is equivalent to a 1.33% AEP, whereas the upper CI is higher than the 0.1% AEP of 2.53mAOD. The Lodmoor area is not at risk from SWL flooding, therefore the flood extents in this area are identical. The biggest differences are around Weymouth Harbour with less flooding from the lower CI event to the south around Cove Row and to the east on Commercial Road and the Swannery Car Park. The upper CI simulation shows increased flooding in these areas plus additional flooding to the west of the harbour along Westway Road and around the Ferry terminal (Figure 7-7). The difference in the modelled levels reflects the differences in the input boundary conditions, with a confidence limit for the results of ±0.2m.

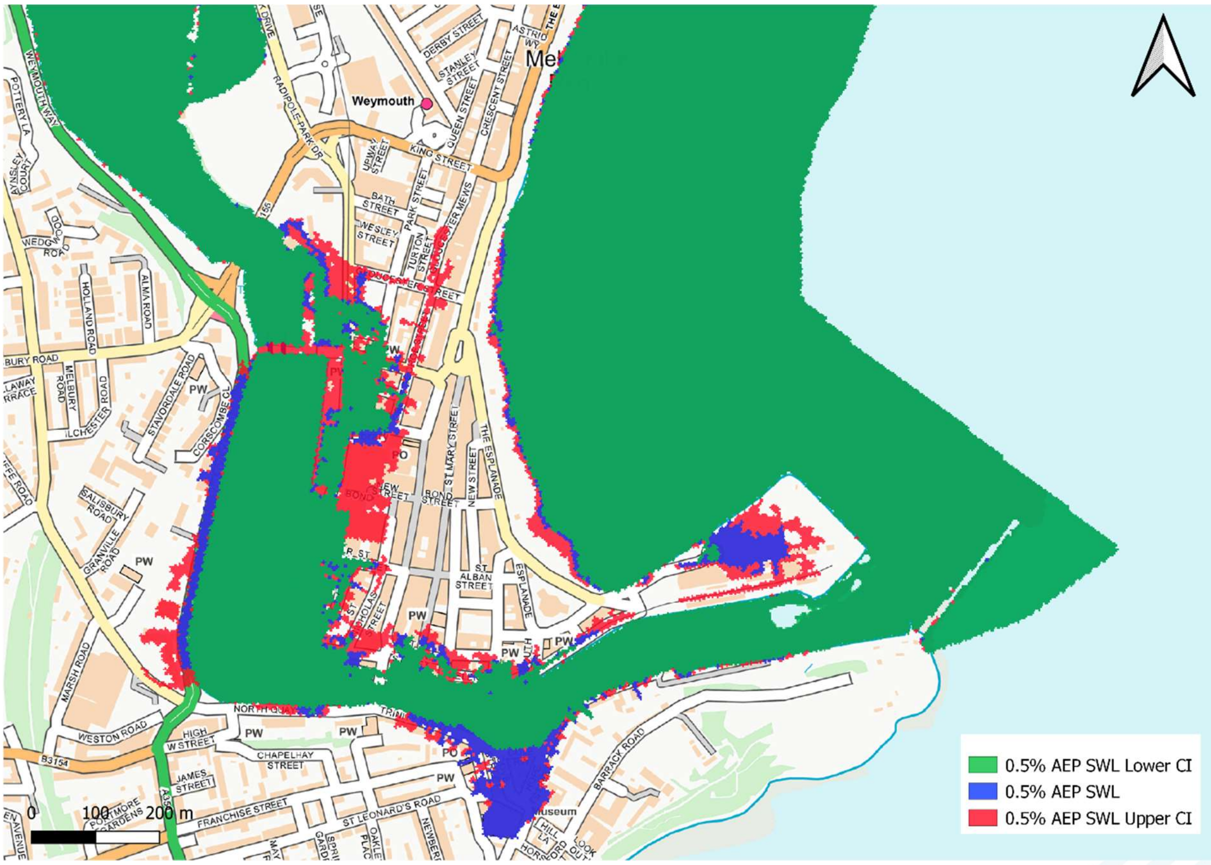

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Figure 7-7: Sensitivity to Coastal Flood Boundary Confidence Interval sea-levels for the 0.5% AEP event

7.4 Model scenarios

The model was used to simulate the flood risk for several different scenarios, these were:

- Defended – with all current defences in place
- Undefended – with all formal raised defences removed
- Defended no overtopping – as defended but without wave overtopping
- Future defences – with planned future defence heights updated to the specifications outlined within Appendix A and C of the FCRM strategy report, alongside direct correspondence from the client
- Defence breach – with breaches through the existing defences in five different locations
- Eroded beach – a scenario with increased wave overtopping due to a lowered beach profile
- Silt scenario – this scenario was run to assess the impacts of siltation in Radipole Lake.

Further details on the model scenarios are provided below.

7.4.1 Defended scenario

This scenario represents the current baseline defences with all existing defences included in the model. Flood risk sources include the still water flood risk from the offshore tidal boundary and the wave overtopping risk from overtopping inflow boundaries on the landward side of the defences.

7.4.2 Undefended scenario

In the undefended scenario all formal raised defences are removed from the model. Quaysides and the Esplanade that are not raised above surrounding ground levels are retained in the model. Any other defacto defences, such as road and rail embankments, where the primary purpose of the embankment is not to function as a flood defence, are retained in the model. The flood risk boundaries include the offshore tidal boundary and revised wave overtopping boundaries, which have the defence crests lowered for the wave overtopping calculations.

7.4.3 Defended no overtopping

This is the same as the defended scenario but without the wave overtopping inflows, so represents the flood risk only from still water, on a calm day with no additional wave overtopping.

7.4.4 Future defence scenario

For the climate change scenarios, this model setup represents the planned future defences updated to the specifications outlined within Appendix A and C of the FCRM strategy report, alongside direct correspondence from the client. The changes and additions to current defences were as follows:

- The raising of the Esplanade (wave overtopping profiles 1,2, 3 & 7), Figure 4-4, by a height of 0.5m. This was achieved by setting a standard elevation value across the defence crest and the associated wave overtopping profiles of 3.65m, 4.05m and 4.40m respectively.
- The addition of a rear 1m defensive raised wall along the length of the Esplanade (wave overtopping profiles 1,2, 3 & 7) the schematised heights of which can be seen in Figure 7-8.
- The upgrading of the defensive wall running adjacent to the B1355 Preston Road (wave overtopping profiles 4, 5 & 6, Figure 4-4) to a standard height of 5.4m, Figure 7-8.
- A staggered increase in the height of the defences within the Harbour and Esplanade regions under the two climate change scenarios (2098 and 2138). This method adds new defences within the two areas, as well upgrading existing defences to standard elevations of 2.92m and 3.74m in line with the climate change scenarios. The existing defences, as well as the planned future defences, within Weymouth Harbour can be seen in Figure 7-9.

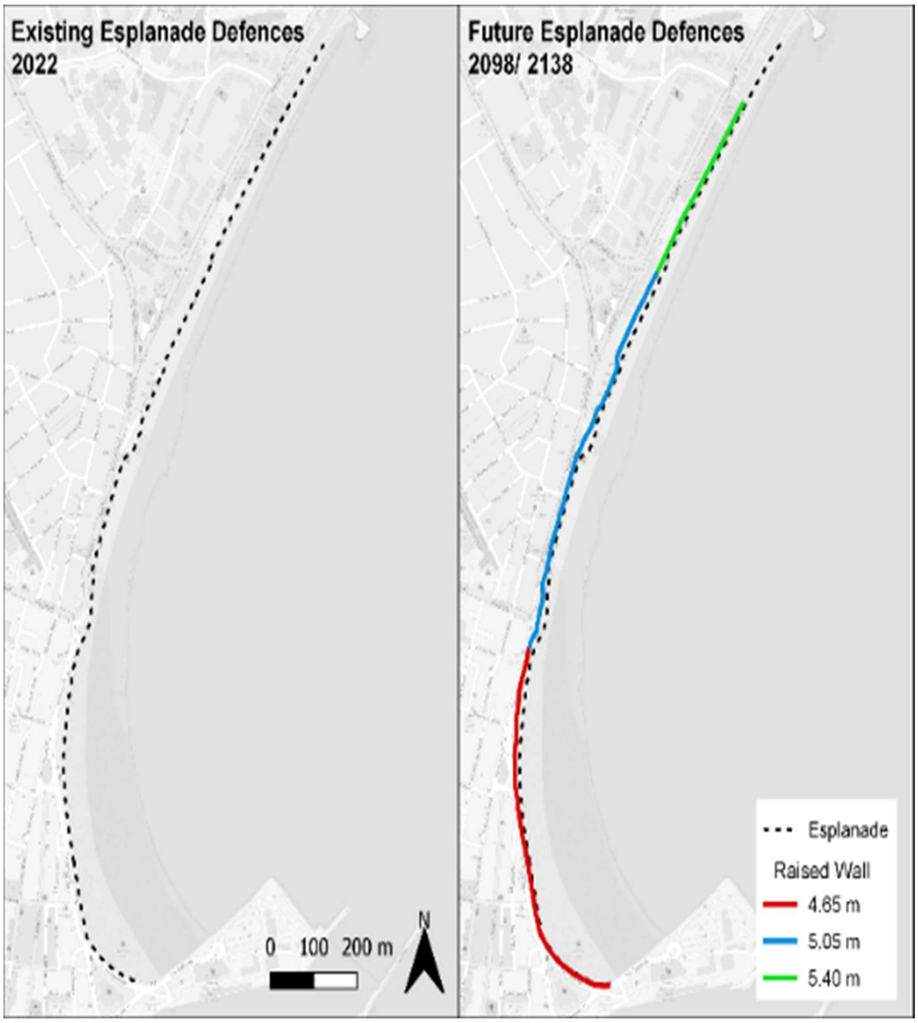

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Figure 7-8: Proposed heights of the raised wall to the rear of the Esplanade

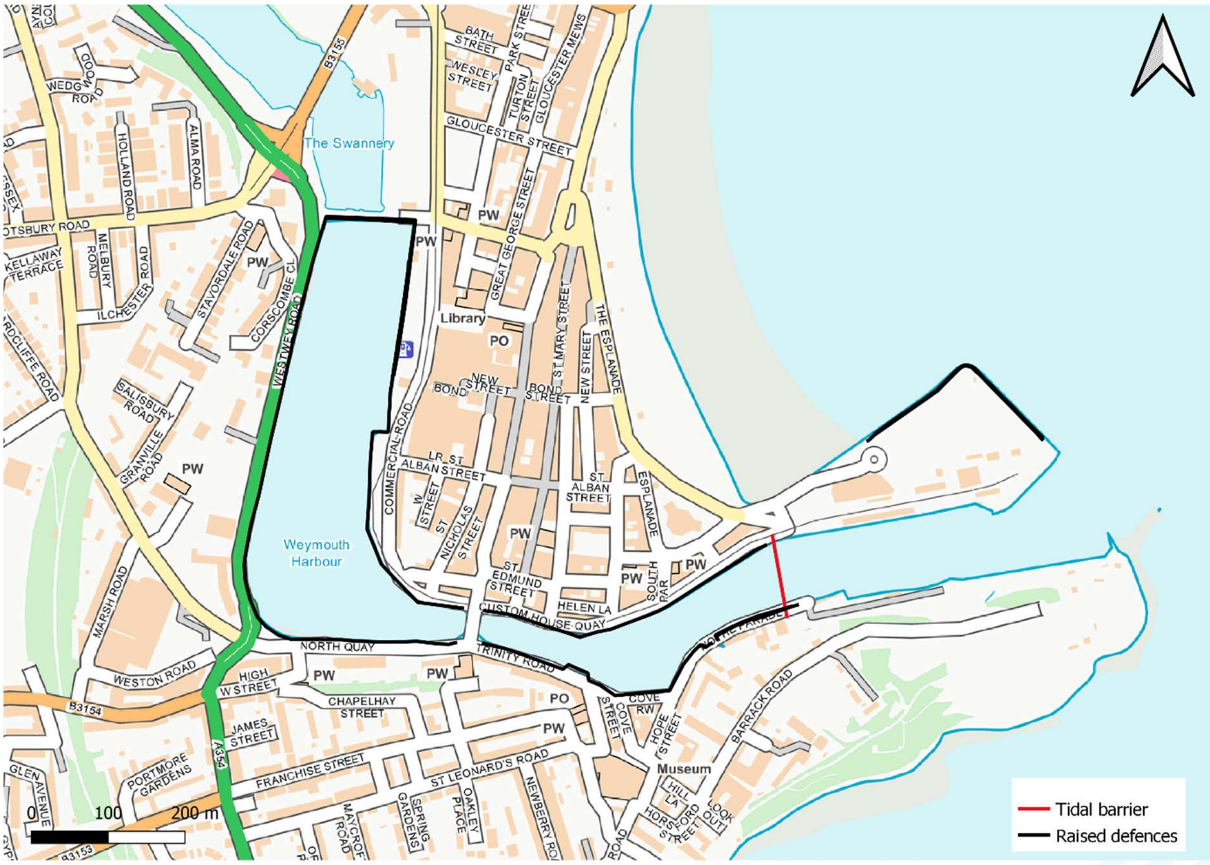

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Figure 7-9: Proposed Updates to Weymouth Harbour

The final update for the future defence scenario is the addition of the tidal barrier to the model. The aim of the tidal barrier is to protect the harbour from any storm surge tide events that may cause significant damage to infrastructure and private property within Weymouth. The tidal barrier was positioned across the Harbour entrance seaward of Weymouth Sailing Club buildings and Weymouth Lifeboat Station; during regular tidal conditions the tidal barrier will be opened to allow for access in and out of the Harbour. The barrier was re-used from the existing model. It was modelled with a TULFOW Operational Control (toc) file as TUFLOW has no current application that allows for an operational barrier to be implemented. Instead, a series of Sluice Gates were used to mimic the role of the Tidal Barrier. The 'toc' file contains a framework for the operation of the three sluice gates which controls the flow past the 'barrier' taking in to account the upstream storage capacity of Weymouth Harbour and Radipole Lake.

7.4.5 Defence breach scenario

In addition to the defended and undefended scenarios, a defence breach scenario was modelled. Breaches through the defences were added in five locations, that were classed as vulnerable locations and agreed with the client. These breaches occur when specific criteria were met and following the triggering of the breach, the defences are flattened down to the level of the surrounding ground level in the location of the breach. Environment Agency guidance was followed for the setup of the breach models. There are two distinct types of break condition within the Weymouth model.

Breach Type 1 are still water breaches within the harbour which occur when the water level reaches ¾ of the height of the defence, these breaches are 20m in width and last for a total time of 18 hours, after which the missing section of defence is plugged. The Breach Locations that use this Type are A, C, D & E, as shown in Figure 7-10.

Breach Type 2 (only being implemented at Breach Location B) triggers at the first instance of wave overtopping within the associated wave Overtopping Profile (WO_05). This breach type also lasts 18 hours, however, as this is located on the open coast the width of the breach is increased to 50m.

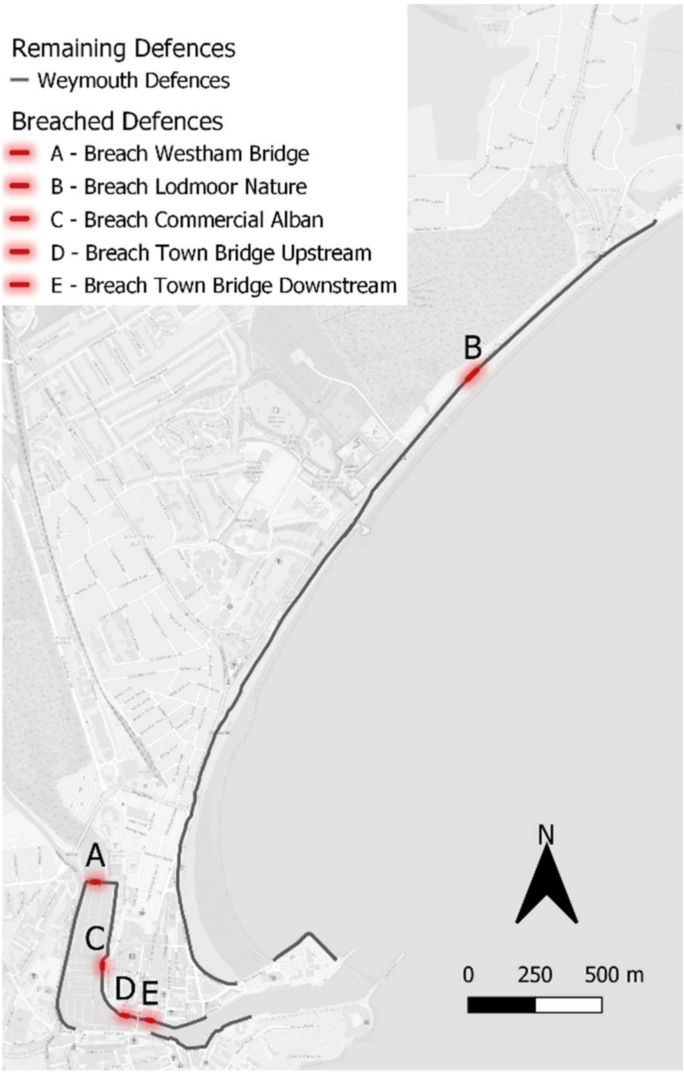

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Figure 7-10: Modelled breach locations.

7.4.6 Eroded beach scenario

The eroded beach describes an event where the beachfront was reduced to an eroded profile. This was achieved by retaining baseline defences, from the defended scenario, with updated wave overtopping rates, detailed in Section 4.1.1.

7.4.7 Silt scenario

The silt scenario represents a situation where there is siltation within Radipole Lake. Increased siltation would reduce the storage capacity of the Lake and could potentially lead to increased flood risk. To represent the siltation in the model, the base DTM levels within the areas of Radipole Lake, that are usually underwater, were raised by 0.5m. The areas of the lake with increased bed levels are shown in Figure 7-11.

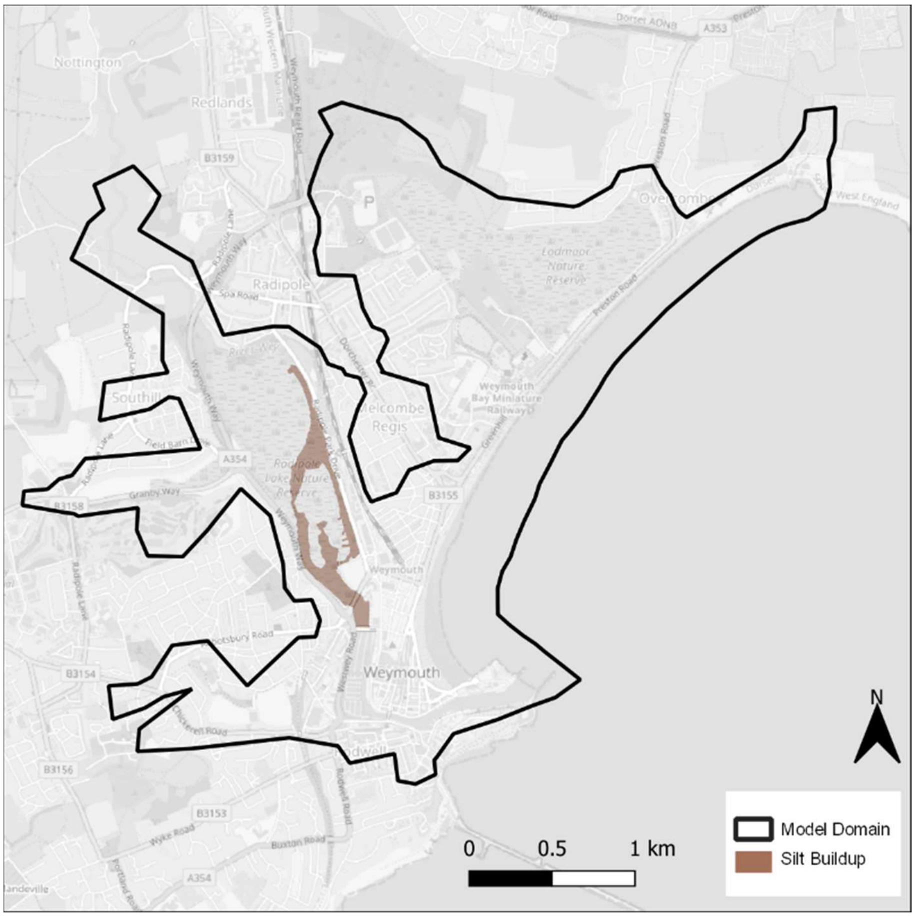

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Figure 7-11: Siltation in Radipole Lake, used within the silt scenario.

7.5 Event simulations

The models were setup to be run for a variety of present day and climate change events for different flood risk forcing. Data for the tidal boundaries was obtained from the CFB chainage point 4734 uplifted for present day events in 2022 and climate change events in 2098 and 2138. Details of the sea-levels applied to the model boundary are summarised in Table 7-1.

Table 7-1: CFB sea-levels used in the model boundary

Epoch\AEP (%)	50	10	5	2.5	2	1.33	1	0.5	0.1
2022	1.91	2.07	2.14	2.22	2.24	2.28	2.30	2.37	2.53
2098	2.89	3.05	3.12	3.19	3.22	3.26	3.28	3.35	3.51
2122	3.33	3.49	3.56	3.63	3.66	3.70	3.72	3.79	3.95
2138	3.62	3.78	3.85	3.92	3.95	3.99	4.01	4.08	4.24

For the fluvial inflows, the flows for the River Wey were taken from the previous study[7]i and the flows for the two tributaries that drain into the Lodmoor Nature Reserve were calculated using FEH catchment descriptors. Details of the fluvial flows used for each watercourse are summaries in Table 7-2. Uplifts to the fluvial flows for the climate change scenario used the Central uplifts for Dorset in the 2080s, which applied a 47% increase to the flows.

7 Environment Agency (2016). Weymouth model updates. JBA Consulting.

Table 7-2: Fluvial inflows used in the model boundaries for the present-day scenarios.

Epoch\AEP (%)	50	10	5	2.5	2	1.33	1	0.5	0.1
River Wey	15.91	26.44	32.24	38.65	40.17	44.89	49.26	61.19	102.91
Lodmoor Tributary 1	0.82	1.38	1.63	1.92	2.03	2.24	2.41	2.89	4.54
Lodmoor Tributary 2	0.40	0.67	0.79	0.94	0.99	1.09	1.17	1.41	2.23

The joint probability of flooding from still water and fluvial flooding was assessed using the Environment Agency joint probability best practice FD2308 guidance[8]. The simplified approach was used with a dependence factor (chi) of 0.125 to calculate joint probability curves, Figure 7-12.

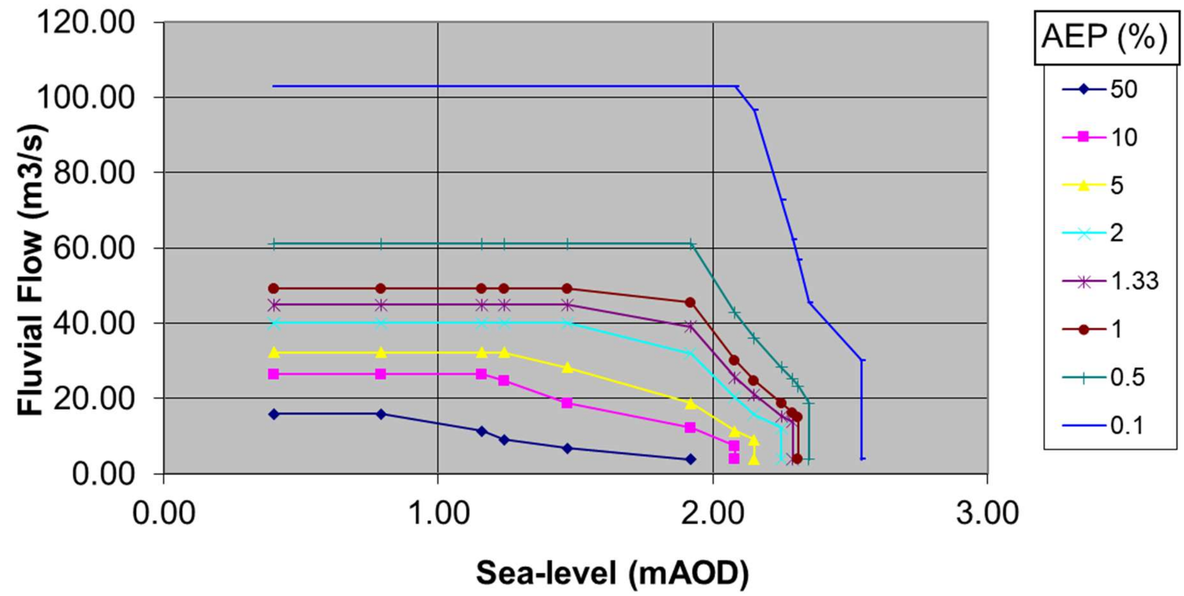

Figure 7-12: Joint probability curves for sea-level and fluvial flows at Weymouth.

Rather than running all combinations of conditions for each event, the models were run for a tidal dominated (TDT) event, a fluvial dominated event and a third joint probability event, selected from the mid-point of each joint probability curve. The tidal dominant events represent the event from the bottom of the joint probability curve where, for example, in a 0.5% AEP event, the tidal boundary has 0.5% AEP conditions, whereas the fluvial boundary

8 Environment Agency/Defra (2005) "Use of Joint Probability Methods in Flood Management: A Guide to Best Practice" R&D Technical Report FD2308/TR1. Defra.

contains the dependent flows (33%), as calculated through the joint probability. Table 7-3 details the event combinations that were simulated for the tidal dominated events.

Table 7-3: Joint probability tidal dominated events

Event AEP (%)	50	5	2.5	1.33	1	0.5	0.1
Tidal AEP (%)	50	5	2.5	1.33	1	0.5	0.1
Fluvial AEP (%)	1000	500	100	100	50	33	6

The event combinations for the fluvial dominated (FDT) runs represent events from the top of the joint probability curve, with the maximum flow for each event combined with the dependant tide, as detailed in Table 7-4.

Table 7-4: Joint probability fluvial dominated events

Event AEP (%)	50	10	5	2	1	0.5	0.1
Tidal AEP (%)	MHWS	MHWS	MHWS	100	50	33	6
Fluvial AEP (%)	50	10	5	2	1	0.5	0.1

For the third joint probability (FDP) event, these were selected from the mid-point of the joint probability curve and represent intermediate tide and flow conditions that combine to equate to the specific AEP event, as detailed in Table 7-5.

Table 7-5: Event combinations for the third Joint probability event

Event AEP (%)	50	10	5	2.5	2	1.33	1	0.5	0.1
Tidal AEP (%)	100	50	10	10	5	5	2	1.33	0.5
Fluvial AEP (%)	200	100	100	50	50	20	33	12.5	1.28

The model results are summarised in the Level 2 SFRA summary sheets.

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