JBA consulting

Woodspring Bay and Severn House Farm Flood Modelling and Mapping Report

Final Report

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Environment Agency

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Revision history

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Contract

This report describes work commissioned by JBA Consulting, on behalf of the Environment Agency, by a contract dated 20/06/2018. The Environment Agency's representative for the contract was Becca Simmins. Callum Rowett, Sarah Turner and Sophie Logan of JBA Consulting carried out this work.

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Purpose

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Carbon footprint

JBA is aiming to reduce its per capita carbon emissions.

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Abbreviations

1D	One-Dimensional
2D	Two-Dimensional
AEP	Annual Exceedance Probability
ANN	Artificial Neural Network
mAOD	Metres Above Ordnance Datum
BODC	British Oceanographic Data Centre
CCO	Channel Coast Observatory
CD	Chart Datum
CFB	Coastal Flood Boundary Dataset
DTM	Digital Terrain Model
EA	Environment Agency
EurOtop	European Overtopping Manual
GIS	Geographical Information System
HAT	Highest Astronomical Tide
HT	Water Level (Head) versus Time (hrs)
LIDAR	Light Detection and Ranging
NOC	National Oceanographic Centre
NPPF	National Planning Policy Framework
OD	Ordnance Datum
ODN	Ordnance Datum Newlyn
ST	Flow versus Time (m^3/s)
SWAN	Simulating Waves Nearshore
TIN	Triangular Irregular Network
TUFLOW	Two-Dimensional Unsteady Flow
UKCP09	UK Climate Projections 2009

1 Introduction

JBA Consulting was commissioned to complete a numerical modelling study, as part of the Programme Delivery Unit (PDU) Modelling and Mapping Lot 1, to assess coastal flood risk along the north coast in the Bristol Channel. The sites of interest include:

- *Woodspring Bay; and*
- *Severn House Farm.*

A suite of models was constructed as part of 2012 FIM Wessex North Coast modelling [(1)] to assess coastal flood risk at sites within the Bristol Channel. The model suite was made up of wave transformation, wave overtopping and flood inundation models. As part of this project, these models were reviewed, updated and reused to assess coastal flood risk at the two key locations of interest in Somerset and Gloucestershire; Woodspring Bay and Severn House Farm (Figure 1-1).

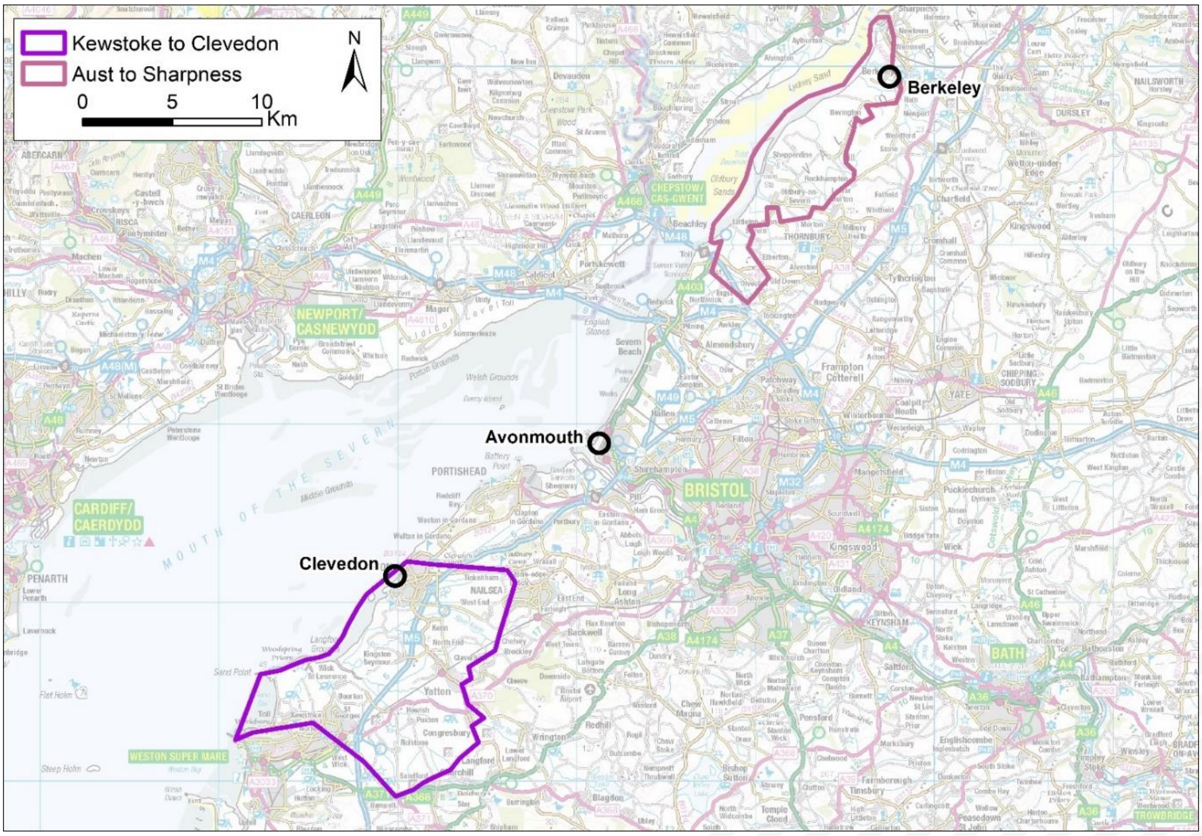

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Figure 1-1: Area of Interest

Several modelling packages and tools were used to understand the tidal and coastal flood risk as a single model is not capable of calculating wave transformation, overtopping and flood flow processes. This is important because coastal flood inundation is a result of extreme water levels (astronomical tide level plus tidal surge) and wave overtopping discharge rates (a function of water levels and wave action).

1 JBA Consulting. 2010s4401 - Environment Agency - South West Region - Somerset North Coast Flood Warning Improvements Study

Wave transformation modelling was used to transform offshore waves to the toe of the defence structure. The resulting wave conditions were then used in a wave overtopping model to calculate wave overtopping discharge rates at the defences. Wave overtopping boundaries were applied along the coastal frontage in detailed flood inundation models, running in parallel with a design tidal water level time series, to map coastal flood risk. The detailed flood inundation models were used to simulate a range of model scenarios and extreme events.

This model development report provides a technical overview of the modelling approaches, data and assumptions used in the study to assess the coastal and tidal flood risk in the Bristol Channel. The report should be read in conjunction with the project summary report. This report is separated into the following chapters:

- *Chapter 2 - Schematisation of defence structures*
- *Chapter 3 – Model boundary conditions*
- *Chapter 4 - Wave transformation model overview*
- *Chapter 5 - Wave overtopping model overview*
- *Chapter 6 - Flood inundation model overview*
- *Chapter 7 – Defence removal modelling*
- *Chapter 8 – Marsh loss modelling*

2 Schematisation of defence structures for Artificial Neural Network

Schematisation of defence profiles is principally required for calculating wave overtopping discharge rates during the wave overtopping calculations stage. It is also used in the wave transformation stage as it defines the location and elevation of where the nearshore wave conditions are extracted at each defence toe.

The locations of defence profiles and how they were schematised is discussed in the remainder of this chapter.

2.1 Defence locations

Coastal flood defences at Woodspring Bay are a combination of masonry sea walls, earth embankments and in places rock armour is present. At Sand Bay it is largely vegetated dunes that were formerly nourished. Severn House Farm is mainly protected by raised earth embankments.

Flood defences at each site were identified and separated into distinct defence sections for the wave overtopping calculations through site inspections, assessment of historical overtopping information and topographic data. These defences were split into 43 representative sections that were theorised as having a specific wave overtopping risk across the sites of interest. The number of defence sections at both sites are detailed in Table 2-1.

The location of the defence sections at each site is shown visually in Appendix A.

Table 2-1: Defence sections at each site

Location	Number of defence sections
Woodspring Bay	25
Severn House Farm	18

2.2 Software and requirements

This study has applied the Neural Network II methodology, known as the Artificial Neural Network (ANN), of which calculation details can be found in the second edition of the European Overtopping Manual (EurOtop)[2]. The ANN was developed by the European CLASH programme, to calculate the wave overtopping discharge rates at the defence sections. Briefly, EurOtop uses a large database of results from physical modelling tests to derive a solution based on complex defence profiles. The ANN uses an expanded database of field and model test results compared to that of the previous edition (Neural Network I) and was therefore preferentially used in this modelling study. The ANN in the EurOtop 2 manual requires a description of the defence geometric profile, and this is provided in the form of 22 input parameters. These include but are not limited to: crest height (R_c); armour height (A_c); armour width (G_c); berm elevation (h_b); berm width (B); upper slope (α_u); lower slope (α_d); and roughness (γ_f). A typical defence profile and the parameters required for the schematisation of ANN profiles are summarised in Figure 2-1.

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[2] EurOtop (2018). "Manual on wave overtopping of sea defences and related structures: An overtopping manual largely based on European research, but for worldwide application. " Van der Meer, J.W., Allsop,N.W.H., Bruce, T., De Rouck, J., Kortenhaus, A., Pullen, T., Schuttrumpf, H., Troch, P., and Zanuttigh, B., www.overtopping-manual.com

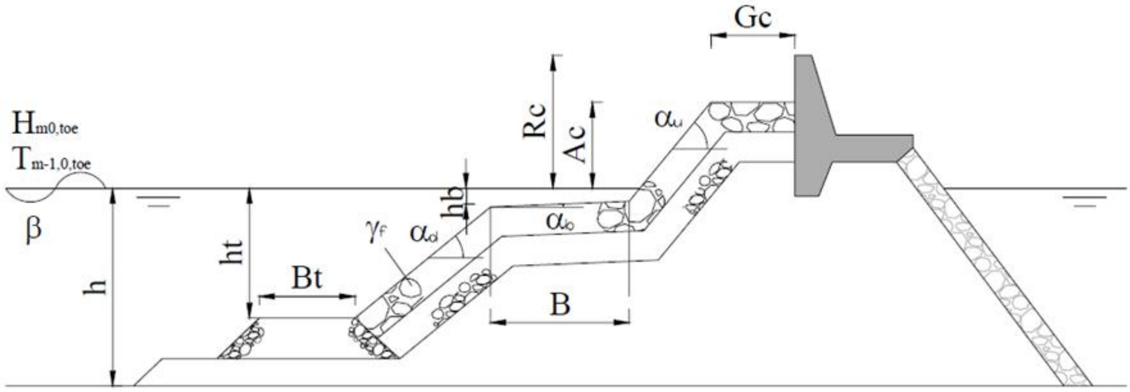

Figure 2-1: Schematisations of a typical defence profile for analysis using the Neural Network II overtopping tool

2.3 Determination of structure parameters for defended scenario

Defence schematisations were derived based on detailed cross-shore survey profiles including data from:

- *Cross-sectional beach surveys by the Channel Coast Observatory (CCO)*
- *Crest level survey (Royal Haskoning, 2007)*
- *Profiles extracted from 1m and 2m Digital Terrain Model (DTM) based on Light Detection and Ranging (LIDAR) supplied by the Environment Agency (EA)*
- *Data sources including site visit information and photographs.*

A representative profile characteristic of the defence section being modelled was generated using the cross-shore and crest level surveyed data where available. The changes in beach profile throughout the year, or throughout multiple years, was extracted where available and used to inform the defence schematisation; an average profile was used based on the data. Where CCO survey data was unavailable, the defence profile was extracted from high-resolution LIDAR, and the crest level survey used to inform defence crest levels. An example of this is shown in Figure 2-2 where the CCO survey data was unavailable and therefore the crest level survey data was used to determine the crest level and the high-resolution LIDAR was used to determine the geometry of the embankment and inform the defence profile slope. In this example the surveyed crest level is noticeably higher than the extracted LIDAR profile but used in preference to the LIDAR due to it being surveyed. For this project, the detailed cross-sectional data was supplemented with visual inspection data such as notes, sketches and photographs, to help inform the defence schematisation.

A defence schematisation QA sheet is supplied alongside this report which gives details on each defence schematisation and the data used to inform the resultant input parameters for the ANN.

A visual representation of the Neural Network defended schematisations used in this study are detailed in Appendix A.

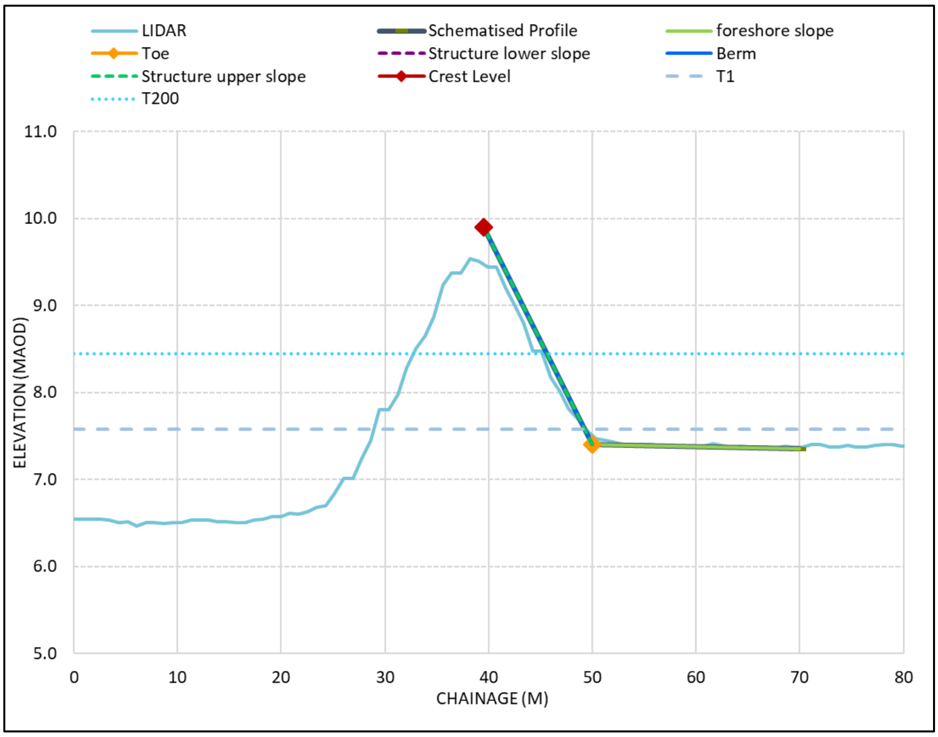

Figure 2-2: Example schematisation (Severn House Farm WO_10) where cross-sectional beach survey data was not available

2.3.1 Discussion on structure parameters

Most coastal structures can be relatively well schematised by means of the 22 structure parameters presented in Figure 2-1. Three parts can be distinguished in an average coastal structure, the toe, the centre or berm and the crest. The separation of these three parts is not always clear and depends on the hydraulic conditions and structure shape. In this way, the same structures could have a different schematisation for a different water level and wave attack. Figure 2-3 shows the three parts of a typical coastal structure, where the berm corresponds to the areas within the vertical distance 1.5*wave height (Hm0), toe above and below the water level.

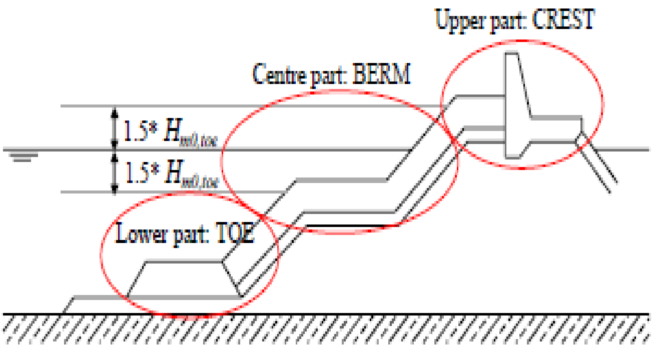

Figure 2-3: Three parts of typical coastal structure

Figure 2-4 shows a different schematisation of the same structure resulting from a higher water level. The schematisation reflects the fact that as water level increases, the influence of the 'toe' on wave conditions lessens. Consequently, the toe was located at the centre of the structure.

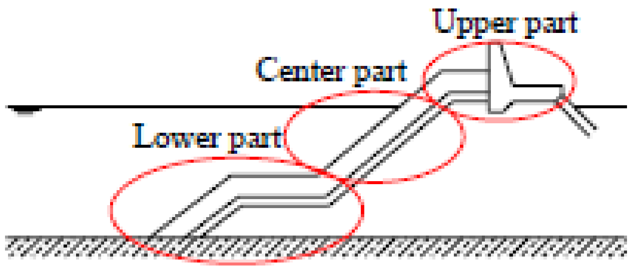

Figure 2-4: Difference in schematisation dependant on water level

For the purposes of this study, only one schematisation can be generated for all water level conditions. As such, it is more important to correctly schematise a structure for water levels where the associated wave overtopping risk is greatest. In some cases, this means that beaches and foreshores have not been schematised, they have instead been represented within the Two-Dimensional (2D) SWAN model. For continuity, the 2D output wave conditions must be representative of the Neural Network II input conditions. As nearshore wave conditions along the Wessex coast can be depth limited, it is crucial that the toe elevation within the 2D SWAN model matches the overtopping schematised profile. Before running the overtopping, the elevations of the SWAN 2D toe node and the toe of the schematised profile were compared. If the elevations were within 0.1m, the wave transformation outputs were considered appropriate for use within the ANN overtopping calculations.

2.4 Artificial Neural Network testing

The ANN tool was used to determine the most appropriate schematisation for all water levels. Testing was conducted on multiple schematisations at all defence sections. Each individual schematisation was tested using a range of water levels from low to extreme with estimated wave conditions that might be expected within the Bristol Channel from preliminary SWAN results.

In brief, the ANN tool compares input parameters to a database of tests on wave overtopping, reflection and transmission to predict average wave overtopping discharge.

The most accurate predictions are given when the input scenarios fall within the domain of validity of the ANN tool. The domain is defined by the structure types and wave conditions on which the ANN is trained. As such, the schematised parameters were adjusted that they best sit within the ANN training data to provide confidence in the modelled result.

2.5 Undefended scenario defence schematisation

Along the coastline of Woodspring Bay and Severn House Farm, the coastal flood defence network mitigates flooding to very low-lying topography on the landward side. Once the coastal flood defences are removed from the flood inundation model, almost the entire coastline would be at still water flood risk during extreme sea-level events. Therefore, for the undefended scenario only one area needed to be considered for the calculation of undefended overtopping discharges. The area where undefended schematisations were generated was within the Severn House Farm model at Berkeley Power Station where there were raised defences and the topography was above the extreme still water risk. The three defence profiles this impacted, and their locations is detailed in the undefended defence schematisation sheets provided in Appendix B.

For the undefended scenario, the formal raised coastal defences were effectively removed from the defence schematisation. The undefended schematised profile was used in the wave overtopping modelling process to determine undefended overtopping discharges.

An example of the defended and undefended schematisation for the raised grass embankment in at the northern end of Berkeley Power Station (Severn House Farm model) is shown in Figure 2-5 and Figure 2-6 respectively. In this example, the embankment was lowered to the ground level and a simple slope and floating berm was used to represent the undefended scenario.

The approach to how the undefended overtopping inflows were applied in the undefended scenario is detailed in Chapter 6.9.7 and 6.9.13 for Woodspring Bay and Severn House Farm respectively.

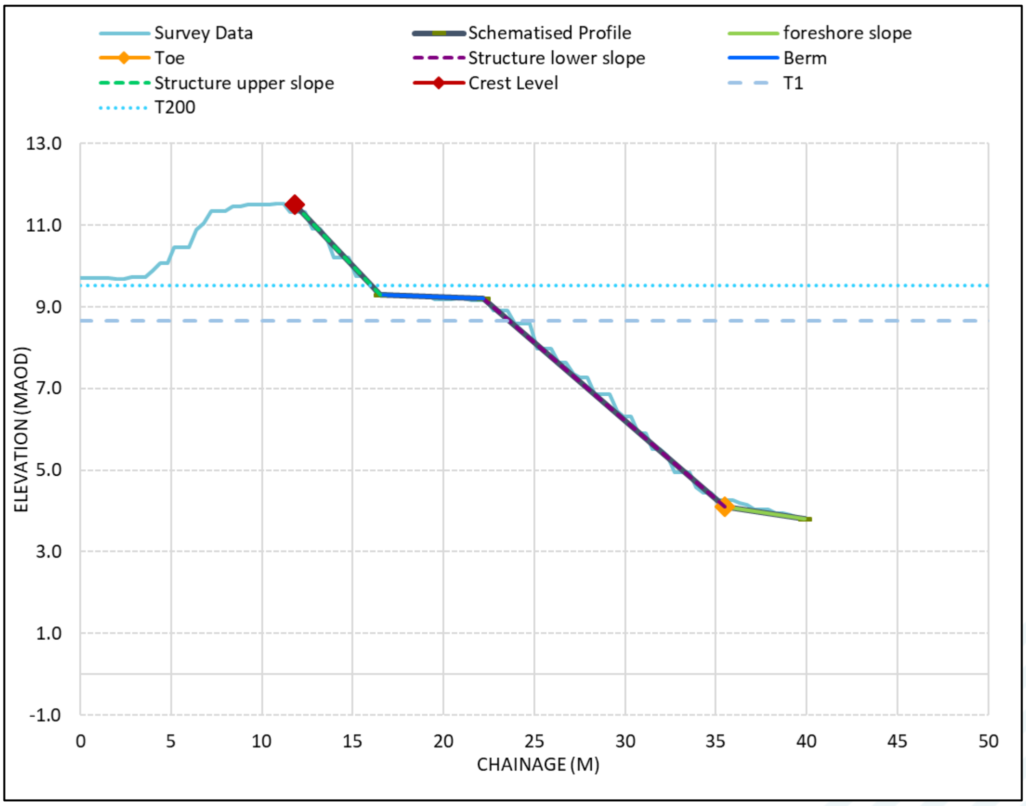

Figure 2-5: Severn House Farm (WO_6) defended schematisation

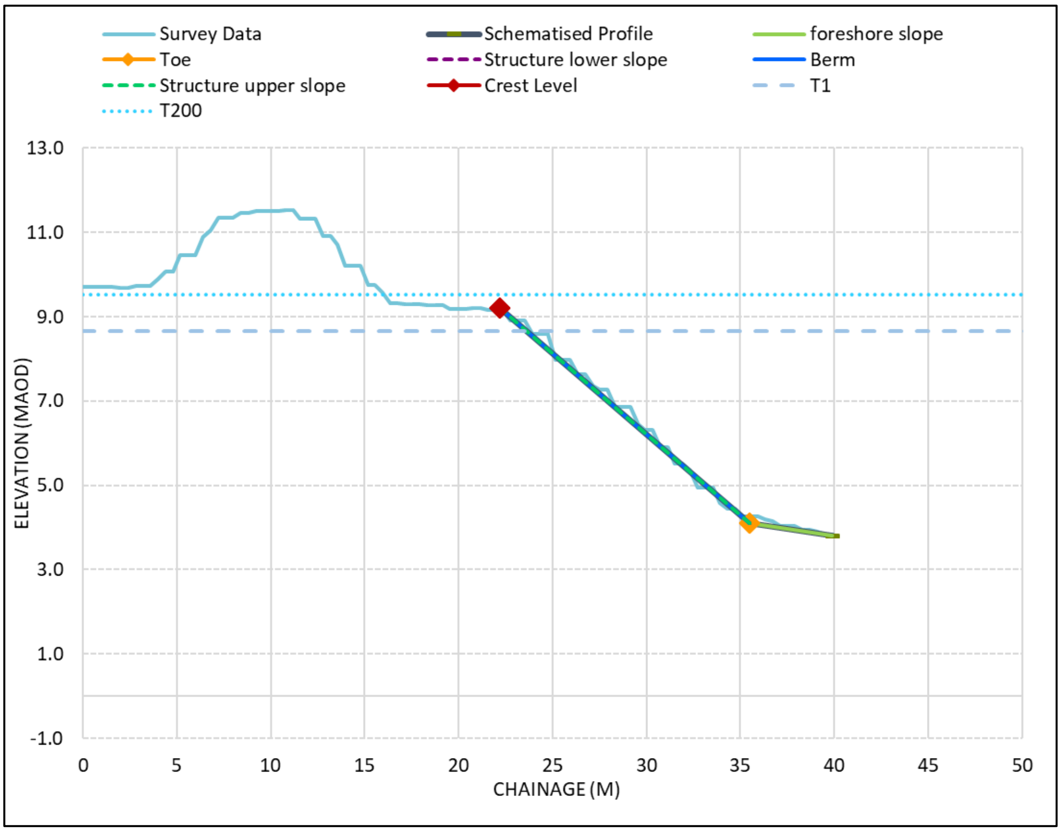

Figure 2-6: Severn House Farm (WO_6) undefended schematisation

3 Model Boundary Conditions

New boundary conditions were generated to drive the suite of coastal models for use in this modelling, mapping and forecasting project.

A stand-alone model boundary condition report is provided alongside this model development report[3] that details how the boundary conditions were generated.

The application of the boundary conditions in the wave transformation, overtopping and flood inundation models are described in Chapter 4, 5 and 6 respectively.

[3] JBA Consulting. 2018s0730 - Environment Agency - South West Region – Wessex Coastal Modelling Boundary Report

4 Wave transformation model overview

The existing Severn Estuary wave model was constructed using the SWAN (Simulating Waves Nearshore) modelling package as part of 2012 FIM Wessex North Coast modelling study. The model was updated with new boundary conditions and topographic data, and recalibrated using the following data sources:

- Wave data from the Wave Watch III point 573 (51.2969°N, -4.243°W)
- Water levels from the Class A Ilfracombe tide gauge.

A full description of the wave model updates and calibration is detailed in the standalone Severn Estuary wave model calibration report[4] that sits alongside this reporting.

4.1 Key Wave Modelling decisions

4.1.1 Woodspring Bay primary and secondary defences

At Woodspring Bay at Wick St Lawrence, there is a primary and secondary defence line. The secondary, more landward defence, is the larger main line defence with the primary defence along the coastal frontage being smaller, but acting to break waves before they reach the secondary defence (Figure 4-1).

Two versions of the wave mesh were set up to enable the extraction of wave conditions for both a with and without the primary defence scenario. The model scenarios simulated, and the mesh configuration was as follows:

- *Defended setup: Primary defence removed from mesh. Wave conditions extracted from infront of the primary and secondary defences for use in overtopping. The primary defence was removed from the mesh for this scenario as the ANN already accounts for reflection, so it was better to remove the defence.*
- *Primary defence removal scenario setup: Primary defence included in mesh as an obstacle with a transmissive boundary. Wave conditions extracted from infront of the secondary defence for use in overtopping. Due to the ground elevations behind no wave extraction and overtopping was required for the primary defence removal as this still water floods in all events.*

Removing the primary wall and replacing with an obstacle within the SWAN control file allowed the feature to be represented and makes for a numerically stable model that allows for the additional consideration of reflection and transmission (refer to Chapter 4.1.3 for further details).

Note; the wave extraction point for the secondary defence was intentionally placed away from the toe of the secondary defence on the flatter ground, as SWAN can give erroneous results when there are large changes in gradient. A similar approach was taken for the defences where there is a foreshore plateau at other locations in the estuary.

A secondary defence was also simulated at Kingston Seymour, but this did not require any variation to the wave or overtopping modelling. The defence removal modelling is discussed in detail in Chapter 7.

[4] Severn Estuary wave model calibration. May 2019. JBA Consulting.

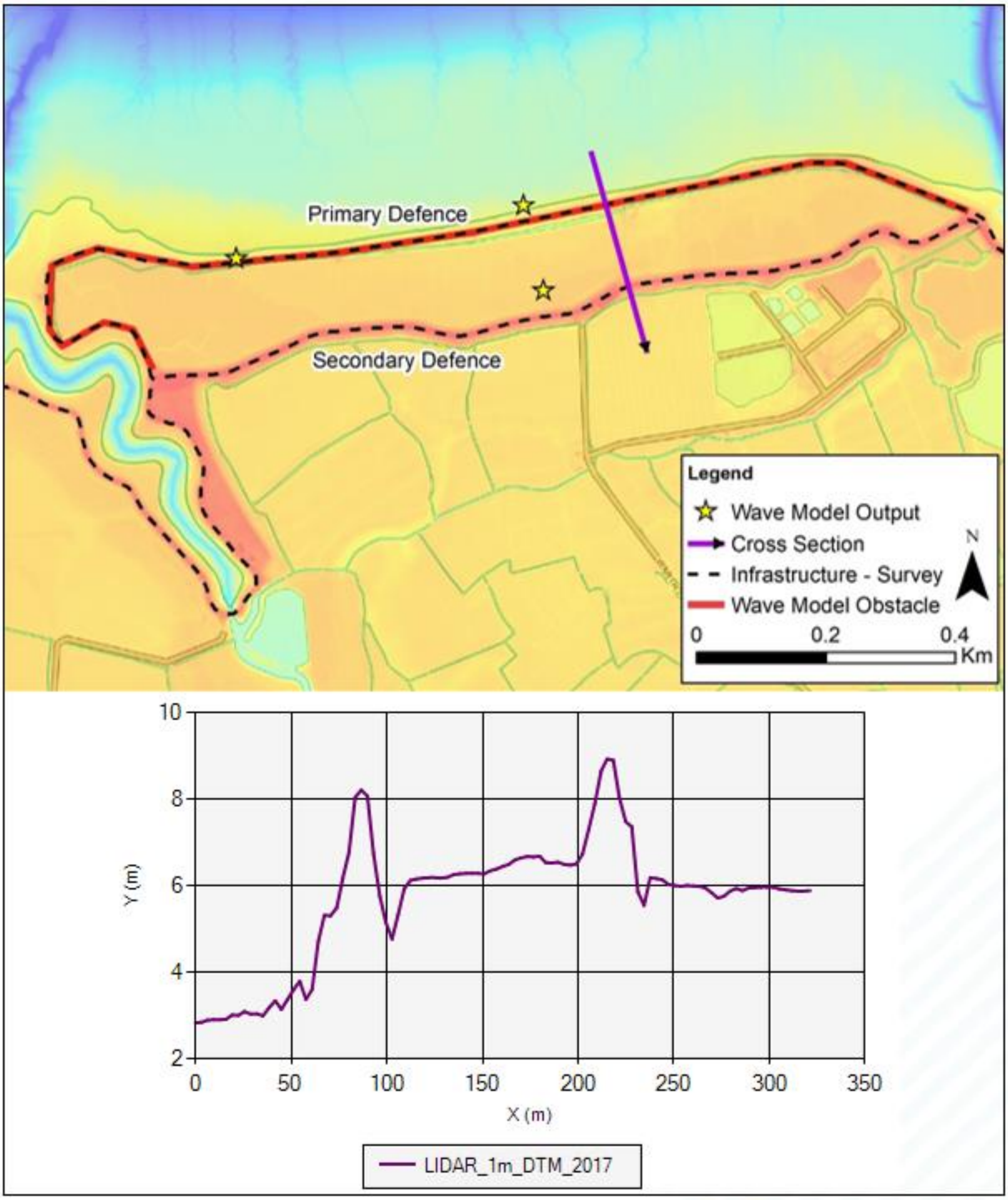

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Figure 4-1: Wick St Lawrence primary and secondary defence

4.1.2 Marine Lake

Marine Lake is a small lake to the south west of Clevedon designed as a safer and calmer environment for swimmers and lake users (Figure 4-2). During high tides and rougher seas, the outer seaward wall overtops which enables water and waves from the Bristol Channel to enter the lake.

Marine Lake was modelled as such that wave conditions were extracted from within the centre of Marine Lake to avoid any model convergence issues near steep gradients. These wave conditions were then extracted for use in calculating overtopping volumes of the raised concrete defence to the rear, fronting the promenade behind (Figure 4-2 – bottom image).

To model waves within the lake basin, the outer seawall and lake bathymetry needed to be included. The crest of the outer seaward wall is roughly 6.20mAOD; this is lower than the 50% AEP CFB water level. Therefore, the outer seawall was modelled using a varying transmissive obstacle within SWAN as shown visually on Figure 4-3 (refer to Chapter 4.1.3 for transmission coefficient calculation).

The lake basin was lowered within SWAN to an estimated bed depth. The lake was assumed to be 2.00m deep from that of the outer seaward wall elevation and the lake floor was lowered to 4.20mAOD.

Images taken from Google Maps

Figure 4-2: Marine Lake

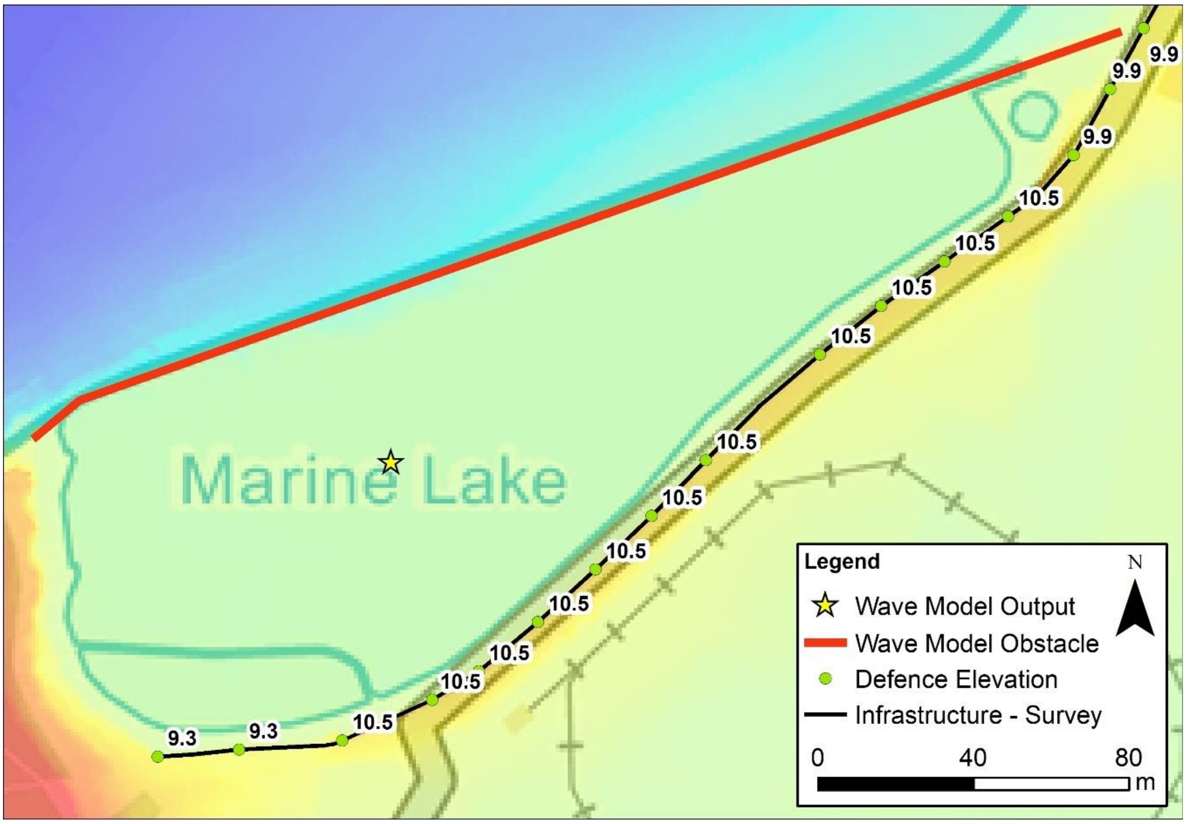

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Figure 4-3: Marine Lake wave model setup

4.1.3 Transmission and reflection coefficients

Reflection from near-vertical structures is almost 100%, however this reflection is restricted by the amount of transmission. Transmission coefficients were applied to the offshore seawall at Marine Lake and the Primary defence at Wick St Lawrence during design simulations to allow the propagation of wave energy through the defences so wave conditions can be extracted behind the seaward line. The transmission coefficients (Ct) were calculated mathematically using equations (see Table 4-1) provided in the Rock Manual. A transmission coefficient was calculated for each SWAN simulation using the water level and wave height at the outer seawall and primary defence.

To determine the water level and wave conditions at Wick St Lawrence and Marine Lake, a spatial adjustment was used based on the CFB extreme water level variation from the wave model boundary to each site. The offshore wave conditions were then depth limited based on the defence toe levels at each site for use in the coefficient calculations.

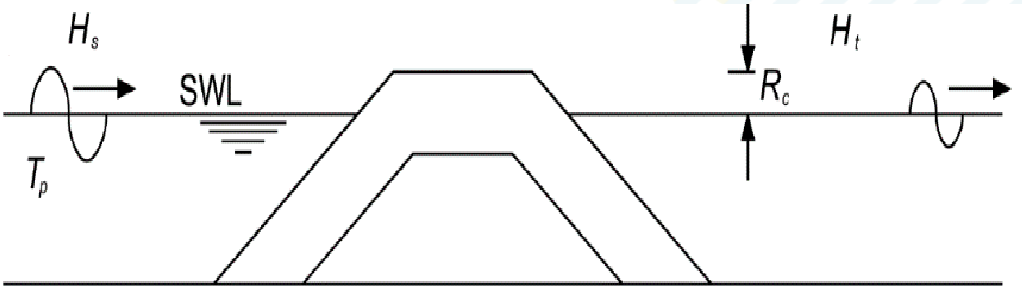

Figure 4-4 Schematised transmission through a breakwater

Table 4-1 Transmission coefficient equations from the Rock Manual

$R_c/H_s \ll -2.00$	$C_t = 1.00$
$-2.00 < R_c/H_s < -1.13$	$C_t = 0.80$
$-1.13 < R_c/H_s < 1.20$	$C_t = 0.46 - 0.30\ R_c/H_s$
$1.20 < R_c/H_s < 2.00$	$C_t = 0.10$
$R_c/H_s \gg 2.00$	$C_t = 0.00$

*Hs is the wave height at the toe of the breakwater and Rc is the crest freeboard (water level – crest level)

4.2 SWAN 1D

As discussed above, SWAN 2D was the primary methodology for the design event wave modelling. However, at Woodspring Bay, the impact of marsh loss was modelled using SWAN 1D. Wave conditions were taken from the 2D SWAN model and transposed across the foreshore and beach using SWAN 1D. The location and methodology for the marsh loss modelling using SWAN 1D is discussed in Chapter 8.

5 Wave overtopping model overview

5.1 Method

The methods outlined in the second edition of the EurOtop manual were used to calculate wave overtopping discharges for this project. The manual includes methods and guidelines on prediction of wave overtopping at seawalls, flood embankments, breakwaters and other shoreline structures. For this study the ANN methodology was used. This method allows for the rapid assessment of complex multi-component defence structures, which are characteristic of many of the defences around the Wessex coastline. The ANN tool requires the following input data at the toe of a defence structure to derive mean wave overtopping discharge rates:

- Nearshore incident wave conditions
- Defence profile schematisation
- Still water level (including wave set-up)

The nearshore wave and water level conditions generated by the wave transformation modelling for every joint probability event, were simulated using the ANN tool. The resulting wave overtopping discharges were analysed and ranked to determine the worst-case overtopping volume for each AEP.

The overtopping rate is expressed in terms of cubic metres per second, per metre of defence length ($m^3/s/m$). Wave overtopping discharges were calculated for the 43 defence profiles as discussed in Chapter 2. A time series of wave overtopping volumes was generated using the worst-case nearshore wave conditions. 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; 10%, 5%, 3.3%, 2%, 1.33%, 1%, 0.5% and 0.1%. Note that the 20% AEP event was simulated in addition to those listed for the Severn House Farm model only. Wave overtopping discharges were also calculated for climate change conditions, as discussed in chapter 5.2.

In cases where the still water level is at or above the defence crest, resulting in a zero or negative freeboard, the wave overtopping volumes were adjusted to avoid double counting the volume of water overtopping the defence from wave action, with that from still water flooding. This is achieved by calculating the volume of water overtopping the defence from still water flooding, that would occur within TUFLOW, and subtracting this from the wave overtopping volumes. This way, the still water flooding is controlled by the 2D model shallow water equations and not by the separate weir equation that is applied in the negative freeboard calculations within the overtopping calculations.

Figure 5-1 shows the wave overtopping discharge profile for six present day events at WO_10 (toe 51) in the Woodspring Bay model after it was adjusted for negative freeboard. Note that the 0.1% AEP event (1 in 1,000-year event) discharge drops off to zero between 79.5 hours and 80.3 hours as still water flooding is expected to occur in the flood inundation model. The volume associated with still water flooding was calculated and removed from the overtopping discharge to avoid double counting this volume within the flood inundation model.

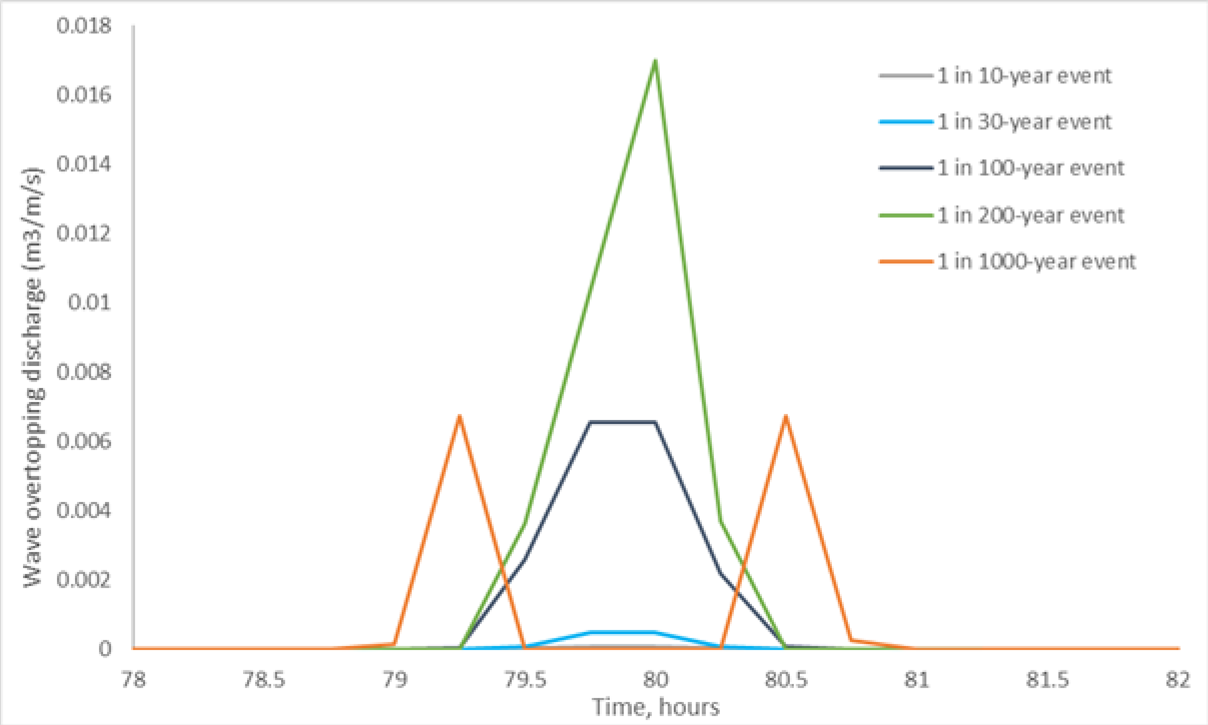

Figure 5-1: Wave overtopping at WO_02 in Severn House Farm

5.2 Climate change wave overtopping calculations

Wave overtopping discharges were generated for climate change events for the 0.5% AEP events for the 2068 and 2118 epochs. The modelling method used to calculate wave overtopping for climate change is the same as for present day, running the joint probability conditions through the wave transformation and overtopping models; but the joint probability offshore combinations were uplifted to represent the potential future situation at the start of the modelling process.

Climate change uplifts were applied to the offshore joint probability conditions as follows:

- Sea levels were uplifted based on The UK Climate Projections 2009 (UKCP09) medium emissions 95th percentile projection pathway and the National Planning and Policy Framework (NPPF) guidance.

- Increases in offshore wind speed and wave height allowances were uplifted by 10% based on UKCP09 guidance for the epoch range of 2056 to 2115.

A summary of the climate change values applied in the modelling and guidance used is shown in Table 5-1.

The uplifted joint probability combinations were then used to calculate overtopping rates under climate change conditions as follows:

- The revised joint probability combinations dataset for climate change was simulated in the wave transformation model to transform all the conditions from the offshore to the nearshore.

- The revised dataset of wave conditions at the toe of the defence structures, which now incorporate the impact of climate change, were run through the Neural Network tool.

- The overtopping discharge rates were ranked, and the worst-case conditions identified. A time-series of overtopping for each AEP was generated using the worst-case wave conditions and a varying water-level time-series to produce overtopping discharges adjusted for the future epochs.

Table 5-1: Climate change uplifts as per the UKCP09 and NPPF guidance

Guidance	Epoch	Sea level rise uplift (m)	Wind speed and wave height (%)
UKCP09	2068	Woodspring Bay: 0.327 and 0.326 Severn House Farm: 0.325 and 0.324	10%
	2118	Woodspring Bay: 0.756 and 0.754 Severn House Farm: 0.752 and 0.75	10%
NPPF	2068	All models: 0.432	10%
	2118	All models: 1.121	10%

5.3 Overtopping results

The worst-case mean overtopping rate for each AEP at each defence toe, alongside the associated nearshore wave and water level conditions, is provided in Appendix C.

5.4 Assumptions

The behaviour of waves in the nearshore and surf zone is highly complex and the subject of detailed research. For this reason, several assumptions were made to represent wave overtopping at the 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 worst-case conditions. For this study, worst-case conditions were identified from the joint probability combinations by calculating the largest overtopping volume associated with each AEP combination of conditions. The full time-series of worst-case overtopping discharges were applied in the model as a mean overtopping discharge.

The most important assumption is that wave conditions, remain consistent throughout the progression of the tidal curve. 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 simultaneously. Changes in overtopping rates are therefore a result of the changing water level conditions rather than any changes in the incident wave conditions. Environment Agency (EA) Flood and Coastal Risk Management Modelling

Guidance[5] recommends modelling wave action over a 12-24-hour period, after which the waves diminish as the storm moves and the wind changes direction. The wave overtopping discharges were calculated over the peak tidal cycle and included for this single peak only.

Offshore winds are accounted for in the offshore 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 modelling.

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.

5.5 Overtopping model validation and sensitivity testing

Evaluating the performance of wave overtopping models is complicated due to the scarcity of observed wave overtopping data available for model verification. This stems from the fact that recorded overtopping data does not exist in the way that recorded wave data does. 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 along the Woodspring Bay and Severn House Farm coastline, comprehensive sensitivity testing was undertaken to examine how variations in the initial parameterisations developed for each defence section affect overtopping volumes.

5.5.1 Frequency analysis

The modelled historical frequency with which overtopping exceeded tolerable discharges was calculated at each defence. The tolerable discharge limits were taken from the EurOtop manual[6]. The thresholds for unaware pedestrian (0.03 l/s/m), aware pedestrian (0.1 l/s/m) and trained staff (l l/s/m) were used. The annual frequencies of exceedance are shown in Table 5-2. The frequency was calculated as the average number of events with an overtopping rate equal or above the threshold per year. Events where selected from a three hourly interval and hence a single storm may be represented by more than one 'event'.

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

6 EurOtop (2007) Wave overtopping of Sea Defences and Related Structures: Assessment Manual. Die Kuste.

Table 5-2: Annual overtopping frequency above safety thresholds for each defence

Model	Defence	Annual overtopping frequency		
		> 0.03 l/s/m	> 0.10 l/s/m	> 1.00 l/s/m
Woodspring Bay	WO_01	0.36	0.12	0.00
Woodspring Bay	WO_02	0.00	0.00	0.00
Woodspring Bay	WO_03	0.08	0.00	0.00
Woodspring Bay	WO_04	0.08	0.00	0.00
Woodspring Bay	WO_05	0.00	0.00	0.00
Woodspring Bay	WO_06	1.44	0.88	0.20
Woodspring Bay	WO_07	0.24	0.20	0.00
Woodspring Bay	WO_08	0.64	0.56	0.12
Woodspring Bay	WO_09	1.48	1.16	0.56
Woodspring Bay	WO_10	12.40	7.52	2.80
Woodspring Bay	WO_11	10.88	7.00	2.24
Woodspring Bay	WO_12	0.28	0.16	0.00
Woodspring Bay	WO_13	0.40	0.24	0.08
Woodspring Bay	WO_14	4.96	2.40	0.68
Woodspring Bay	WO_15	1.00	0.60	0.20
Woodspring Bay	WO_16	0.04	0.04	0.00
Woodspring Bay	WO_17	0.00	0.00	0.00
Woodspring Bay	WO_18	0.28	0.12	0.04
Woodspring Bay	WO_19	0.56	0.40	0.20
Woodspring Bay	WO_20	0.04	0.00	0.00
Woodspring Bay	WO_21	0.00	0.00	0.00
Woodspring Bay	WO_22	0.12	0.00	0.00
Woodspring Bay	WO_23	0.32	0.08	0.00
Woodspring Bay	WO_24	1.04	0.16	0.00
Woodspring Bay	WO_25	3.28	1.68	0.68
Severn House Farm	WO_01	0.56	0.32	0.12
Severn House Farm	WO_02	1.24	0.84	0.24
Severn House Farm	WO_03	4.80	2.60	0.44
Severn House Farm	WO_04	0.00	0.00	0.00
Severn House Farm	WO_05	0.00	0.00	0.00
Severn House Farm	WO_06	0.00	0.00	0.00
Severn House Farm	WO_07	0.88	0.44	0.12
Severn House Farm	WO_08	0.24	0.16	0.04
Severn House Farm	WO_09	0.92	0.76	0.52
Severn House Farm	WO_10	0.24	0.24	0.04
Severn House Farm	WO_11	0.92	0.48	0.16
Severn House Farm	WO_12	0.36	0.24	0.04
Severn House Farm	WO_13	0.16	0.00	0.00
Severn House Farm	WO_14	0.76	0.64	0.28
Severn House Farm	WO_15	0.20	0.12	0.04
Severn House Farm	WO_16	0.64	0.56	0.08
Severn House Farm	WO_17	0.84	0.48	0.20
Severn House Farm	WO_18	0.00	0.00	0.00

Profiles identified with the most frequent overtopping were profiles 10 and 11 in the Woodspring Bay Model. Each of these profiles had annual frequencies of over 10 events per year that were exceeding the 'unaware pedestrian' threshold of 0.03l/s/m. Both these profiles are part of the primary defence line at Wick St Lawrence which has a significantly lower crest than the other profiles in the model and a high rate is therefore expected at these two locations. Another profile in the Woodspring Bay model with a high frequency is profile 14. Profile 14 has a slightly lower crest than the adjacent profiles and is on a slightly more exposed section of coast. This position leads to frequent overtopping. In the Severn House Farm model, profile 3 has the highest frequency due to the low defence toe at this location allowing for large waves to reach the foot of the defence and resultant more frequent overtopping events. The crest of profile 3 is also lower than the surrounding profiles.

5.5.2 Historical flood events

Historically the two study sites of Woodspring Bay and Severn House Farm have only experienced one known extreme flood event. This event was that of the January 30th 1607 Bristol Channel floods which caused the largest loss of life from any sudden onset natural catastrophe in the United Kingdom during the past 500 years. Between 500 and 2,000 people drowned in villages and isolated farms on low-lying coastlines around the Bristol Channel and Severn Estuary (Figure 5-2). The cause of the flood is still debated, stemming from tsunami claims in 2002, but evidence of the timing of the floods relative to the tides, other weather observations, and the absence of any reports of an earthquake, support the theory that the event was a wind driven storm surge superimposed on an extreme spring tide[7]. Defences throughout the Bristol Channel since this event have markedly improved, including those at Woodspring Bay and Severn House Farm. However, it is a useful reminder of the dangers posed by an extreme surge event and the importance of flood defence management.

Figure 5-2: Depiction of flooding as shown in the pamphlet Lamentable Newes out of Monmouthshire (from Great Flood of 1607 website: http://website.lineone.net/~mike.kohnstamm/flood/)

7 1607 Bristol Channel Floods: 400-Year Retrospective. Risk Management Solutions Special Report. 2007.

In more recent times the study sites have not experienced any major flood events. Flooding was small scale and short-lived with no threat to human life. Each of the sites is, however, susceptible to flooding from both wave overtopping and still water flooding and the risk is likely to increase in the future under climate change conditions. This chapter discusses the historic coastal flood events for each of the five sites.

Information was collated using the following sources:

- *Evidence from the Flood Reconnaissance Information System (FRIS) database.*
- *Flood warning performance data.*
- *YouTube videos of overtopping events.*
- *Class A gauge readings at Avonmouth/Portbury, Hinkley and Severn Bridge from The British Oceanic Data Centre (BODC) and the National Oceanic Centre (NOC).*

The Class A tidal water level gauges closest to the study areas are Avonmouth and Hinkley. Time series data from these gauges were used to inform the historic flood record. Analysis of the spatial variation in extreme sea levels using the 2018 CFB dataset shows sea levels increase up the Bristol Channel as tidal waters are funnelled into the estuary. This trend is reflected in gauge readings between Minehead and Portishead.

Areas at risk of coastal flooding are predominantly confined to the low-lying coastal areas and properties that front the coastal defences, as follows:

- *Sand Bay is at risk from overtopping of the dune system that borders the road. If water depth is great enough this area and the area behind is susceptible to still water flooding.*
- *The section of coast between Wick St Lawrence and the Blind Yeo culvert is at risk from overtopping and still water. However due to the remoteness of these defences and water passing over the defences will not impact any residential sites.*
- *The parkland and shorefront businesses around Marine Lake are at risk from overtopping from waves inside Marine Lake. This is especially a risk along the section of defence that borders Poet's walk as this is considerably lower than the adjacent defence.*
- *The Severn House Farm model covers a remote section of coastline mainly fronted by grass embankments where the risk from overtopping is minimal. However once water levels are great enough to pass over the defences much of the model domain is at risk from still water flooding due to the low-lying nature of the area.*

5.5.3 Sensitivity testing

Sensitivity testing was undertaken for all overtopping defence profiles[8]. The sea-state conditions for the 0.5% AEP event at each defence were used to perform the sensitivity testing. Any profiles that did not overtop during the present day 0.5% AEP have zero overtopping during the sensitivity tests.

At each defence the sensitivity of overtopping rates to the following six parameters was tested: wave height at the toe, wave period, angle of down slope, angle of upper slope, crest freeboard and armour freeboard. Two alternative values from that of the baseline defended profile schematisation were tested for each parameter as detailed in Table 5-3. Note that many of the profiles already have a shallow beach slope which approaches the

8 Note; A few additional overtopping defence profiles were added post draft modelling review and these have not been included in the sensitivity analysis

limits of Neural Network. As such lowering the slope would have taken them out of range of the model limits, instead only increases in slope were calculated.

Table 5-3: Sensitivity testing model parameters

Overtopping parameter	Minimum	Maximum
Wave height at toe	-10%	+10%
Wave period (s)	-1	+1
Angle of down slope (°)	+5%	+10%
Angle of upper slope (°)	+5%	+10%
Crest freeboard (m)	-0.1	+0.1
Armour freeboard (m)	-0.1	+0.1

The results of the sensitivity testing can be found in Appendix E. As expected, the analysis demonstrated that the wave overtopping models are sensitive to the schematisation, and even small variations in each parameter will result in changes to the overtopping rate. Across the sensitivity simulations the overtopping was observed to vary significantly.

Adjusting the schematisation parameters resulted in some inputs that were either 'out of range' of the ANN model tool or had an unrealistically high Rc/Hs (armour crest over significant wave height) value. These results skew the data making it difficult to make inferences from the sensitivity tests and as such, the results shown in Appendix E should be considered with a degree of caution. The ANN works by selecting the nearest field or lab test result that closest matches the input parameters. Increasing the wave height value by 10% from the baseline simulation can return a result based on an entirely different field test. This can lead to a comparison between the baseline and sensitivity test showing inconsistent results; in the case of some sensitivity tests for this project giving lower overtopping rates than the baseline case that has a lower wave height. Further investigation into the ANN is recommended for future use in model sensitivity.

Overall, the sensitivity analysis has shown that the profiles are sensitive to defence schematisation, however, the defence schematisations were based on the best available data available at the time. Estimation of overtopping is inherently uncertain. Rates of overtopping calculated in the EurOtop and Neural Network manual are based on a dataset of small-scale physical model tests. More generally, overtopping is considered accurate/reliable only within an order of magnitude. The results of the sensitivity testing suggest that the sensitivity of the defence profiles are within the accuracy of overtopping estimation and so the results may be considered "reliable", while understanding that all wave overtopping is "indicative" to an order of magnitude only.

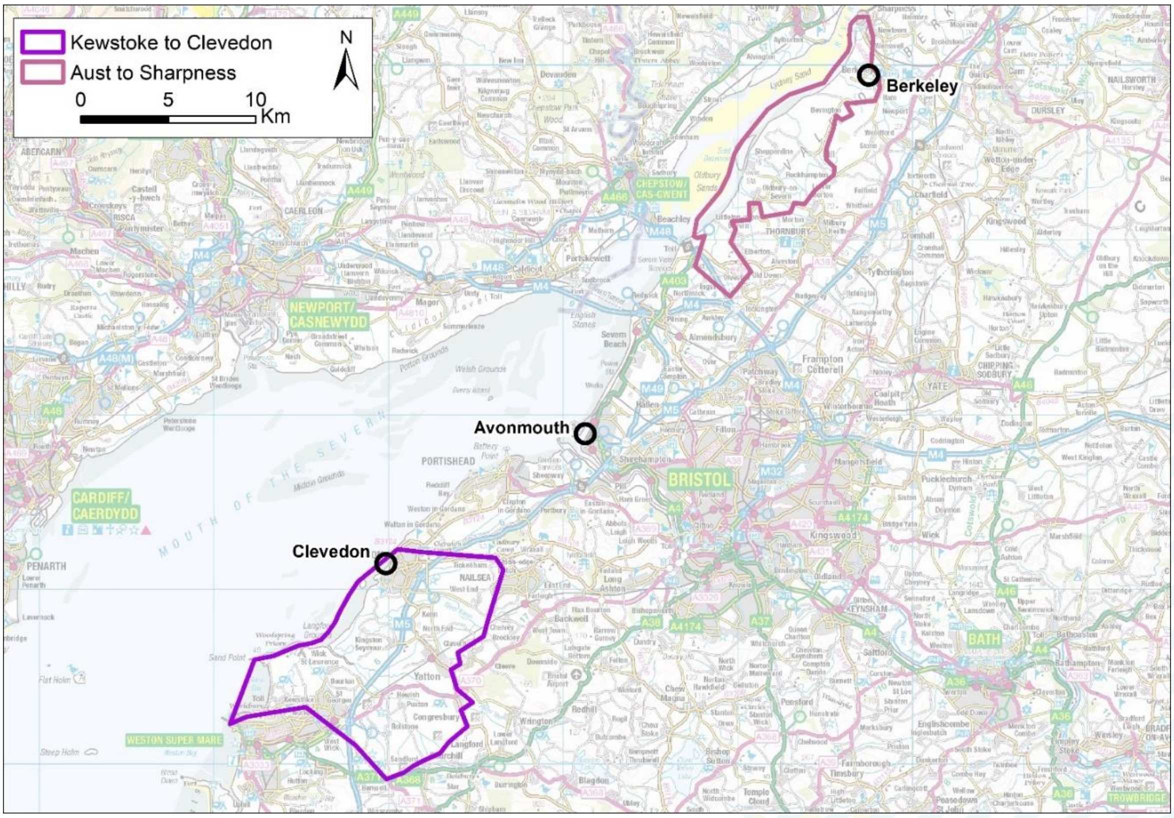
It must be noted that within Appendix D and E, there are two profiles within the Severn House Farm domain and one profile within Woodspring Bay domain that have not been included in the sensitivity testing, due to these profiles being added onto the modelling at a later date within the project.

6 Flood inundation model overview

The coastal and tidal flood risk at Woodspring Bay and Severn House Farm along the Wessex coastline was assessed using two 2D TUFLOW models. As part of the 2012 FIM Wessex North Coast modelling a series of TUFLOW flood inundation models were generated (Figure 6-1). The sites of interest for this study were covered by the existing models, however the following adjustments have been made:

- *Kewstoke to Clevedon (named Som3) covers the Woodspring Bay area. This model was extended to the south-west to include Weston-super-Mare, so that the flood flows can pass between Woodspring Bay and Weston-super-Mare across the low lying topography. However, the direct flood risk to Weston-super-Mare is not being considered as part of this project, therefore, there are no overtopping defence schematisations in this section of coastline, only considering still water flood risk from Weston-super-Mare.*
- *Aust to Sharpness (named Som5) covers the area surrounding Severn House Farm. This model domain was extended eastwards to prevent glass walling from occurring and allow water to flow to its maximum extent.*

This chapter provides a technical overview of the TUFLOW modelling undertaken for this project.



Contains OS data © Crown copyright and database right 2020

Figure 6-1: Existing TUFLOW model domains

6.1 Available data

The Environment Agency 2017 filtered composite LIDAR dataset 1m resolution formed the basis of the topographic data included in the new hydraulic models. A low-tide flown Surfzone LIDAR dataset generated in 2014 was provided and used to extend the foreshore topography out to low water. Other key datasets used in the model building process include:

- *Crest level survey (Royal Haskoning, 2007) - used to inform defence levels and locations following some spot checks against the latest LIDAR.*
- *Congresbury Yeo Tidal Bank Scheme crest level and section information.*
- *One-Dimensional (1D) structure data extracted from existing 2012 FIM Wessex North Coast modelling.*
- *CFB dataset of extreme sea levels (2017 base year).*
- *UKCP09 and NPPF sea level rise guidance.*
- *OS MasterMap data to delineate land use types.*
- *Ordnance Survey (OS) 1:10,000, 1:25,000 and 1:50,000 mapping.*

No bathymetric data was required as the TUFLOW model boundaries where located where low-tide flown LIDAR data was available in the low tide area.

6.2 Modelling software

The latest build of TUFLOW available when this project was commissioned, 2018-03-AE-iSP-w64, was used in this study. No additional modelling software was used in this project.

As of TUFLOW build 2017-09-AA, TUFLOW offers HPC (Heavily Parallelised Compute) as an alternate 2D Shallow Water Equation (SWE) solver to TUFLOW Classic. Whereas TUFLOW Classic is limited to running a simulation on a single CPU core, HPC provides parallelisation of the TUFLOW model, allowing a single TUFLOW model to be run across multiple CPU cores or GPU graphics cards. Simulations using GPU hardware provides significantly quicker model run times for TUFLOW users.

For this study we used TUFLOW HPC to benefit from the improved model run times and enable the model cell size to be increased to 2m resolution to better represent key flow paths. To simulate TUFLOW using HPC, the model setup and functionality is essentially the same as Classic, with only the following additional information required in the TUFLOW control file:

- Solution Scheme == HPC
- Hardware == GPU

Unlike TUFLOW Classic, HPC uses an explicit finite volume solution. This means that the model is mass/volume conserving by construction (0% mass error). Therefore, the cumulative mass error often used to determine if a TUFLOW Classic model is numerically converging, is not of much use for a HPC model. The stability of the HPC explicit finite volume scheme is linked to the timestep, flow velocities, water depth, and eddy viscosity. The maximum timestep that can be used while maintaining model stability changes as the model evolves. While it is possible to choose a fixed timestep ahead of time (similarly to TUFLOW Classic), in this study we used the adaptive time stepping (where the solver continually modifies the timestep based on various stability criteria) to shorten run times and guarantee model stability from start to finish. The unconditional stability of the HPC solver means it remains stable by reducing its timestep and does not alert the modeller to stability issues. Therefore, a more rigorous assessment of model stability was undertaken, as follows:

- All models were simulated using TUFLOW Classic in the first instance to help identify model issues based on warning messages or poor mass error.
- Checks were made on model timesteps; excessively small timesteps would be a strong indicator of poor model health. A high occurrence of repeated timesteps would indicate an issue in the model data or set up.

- A thorough assessment of the model results was undertaken. Water level fluctuations, flow patterns, performance at boundaries and links were thoroughly assessed.

Model stability checks for each model are detailed in Appendix F.

6.3 Modelled Scenarios and events

The modelled scenarios and events undertaken as part of this study are detailed in Table 6-1. Two basic model configurations were developed; a defended modelling configuration and an undefended modelling configuration. It was necessary to build these two different configurations because of the requirement to model the current presence of defences and absence of the defences to produce the Flood Zone and Areas Benefitting from Defence (ABD) outlines. The defended scenario included:

- *Formal defences*
- *Defacto defences and infrastructure*
- *Wave overtopping inflows along the coastline.*

The undefended scenario included:

- *Removal of all formal raised defences, including raised defences set back from the coastal frontage*
- *Defacto defences and infrastructure*
- *Undefended wave overtopping inflows along the coastline where topography was above extreme water levels and there was a raised defence*
- *No overtopping included where topography behind removed coastal defence was very low and would be still water dominated.*

The locations of the defended and undefended overtopping, along with the defence profile schematisations is provided separately in Appendix A and B respectively.

Due to the low-lying topography in both the Woodspring Bay and Severn House Farm models, the dominant flood risk is still water in the undefended scenario. As such, no undefended overtopping was included in the Woodspring Bay model for the undefended scenario. In the Severn House Farm model, three defences were schematised for the undefended model (WO_4 through WO_6) as they are raised defences that will have an associated overtopping risk.

In addition to these key configurations, defence removal and marsh loss scenarios at Woodspring Bay were modelled as discussed further in Chapter 7 and 8 respectively.

The following TUFLOW control files were used to run the models (*'Model_name'* denotes the name of the two models run):

- *'Model_name'_~s~_~e~_001.tcf – TUFLOW control file*
- *'Model_name'_General_Commands_001.trd – TUFLOW results directory*
- *'Model_name'_Events_001.tef – TUFLOW events file*
- *'Model_name'_001.tgc – TUFLOW geometry control file*
- *'Model_name'_Boundary_Control_001.tbc - – TUFLOW boundary control file*
- *Topo_Roughness.tmf – TUFLOW material file*

Table 6-1: Modelled scenarios and simulation list

Event (% AEP)	Event (Return Period, years)	Scenario	
		Defended	Undefended
10	10	✓	✓
5	20	✓	✓
3.3	30	✓	✓
2	50	✓	✓
1.3	75	✓	✓
1	100	✓	✓
0.5	200	✓	✓
0.1	1,000	✓	✓
0.5 + UKCP09 (2068 and 2118)	200 + UKCP09 (2068 and 2118)	✓	✓
0.5 + NPPF (2068 and 2118)	200 + NPPF (2068 and 2118)	✓	✓

6.4 Raw Model outputs

A list of model grid outputs generated as part of this study are detailed in Table 6-2. These grids are an output of the maximum modelled value across the grid during the model simulation. Key model parameters can be found in Table 6-3.

Table 6-2: Raw model outputs and parameters

Grid Name	Description
d	Water depth - maximum
h	Water level - maximum
V	Velocity - maximum
ZUK0	Hazard rating grid

Table 6-3: Key model parameters

Parameter	Time
Woodspring Bay model start time (hrs)	48.00
Woodspring Bay defended model end time (hrs)	97.25
Woodspring Bay undefended model end time (hrs)	110.75
Woodspring Bay defended model run time (hrs)	6.50
Woodspring Bay undefended model run time (hrs)	11.50
Woodspring Bay map output interval (sec)	900.00
Severn House Farm model start time (hrs)	50.00
Severn House Farm defended model end time (hrs)	101.75
Severn House Farm undefended model end time (hrs)	101.75
Severn House Farm model defended run time (hrs)	4.50
Severn House Farm model undefended run time (hrs)	5.50
Severn House Farm map output interval (sec)	900.00

Flood hazard rating grids, a function of risk associated with flood depth and velocity, were produced for each simulation. The grids required for these maps were generated according to the following equation:

Hazard Rating (HR) = Depth x (Velocity + 0.5) + (Debris Factor)

This equation is in line with the Department for the Environment, Food and Rural Affairs' (Defra's) "Supplementary Note on Flood Hazard Ratings and Thresholds" that was issued in May 2008. A debris factor of 0.5 was used for depths < 0.25m and a debris factor of 1.0 was used for depths > 0.25m. These Debris Factors correspond to a conservative approach and corresponding values in Defra’s Supplementary Note. We adopted this approach, despite large areas of the modelled floodplain being rural, to fit in with the more general conservative approach adopted for flood mapping projects.

This Supplementary Note also provides guidance on classifying Flood Hazard Ratings, as detailed below. These were adopted for the study.

- *HR < 0.75: Very low hazard.*
- *0.75 < HR < 1.25: Danger for some.*
- *1.25 < HR < 2.0: Danger for most.*
- *HR > 2.0: Danger for all.*

6.5 Model cell size

For the two model domains the 10m existing model resolution was increased to 5m resolution. This was a suitably high resolution when considering the following:

- *Detailed enough to represent important flow paths, for example roads are typically 15m wide*
- *Acceptable model simulation times*
- *Manageable results file sizes*

As is often the case with 2D modelling, greater detail could be achieved using a finer grid resolution, but this would make model simulations times and results file sizes unmanageable.

6.6 Buildings approach

To represent buildings within both models the following options are available to TUFLOW modellers, as discussed in Engineers Australia, 2012[9]:

- *Increased model roughness for building footprints*
- *Blocking out of model elements*
- *Modelling of exterior walls partially or in full*
- *Using energy loss coefficients within the footprint*
- *Modelling buildings as porous elements*

The 2012 paper by Engineers Australia states that whichever method of building representation is used, the impact of flow volume stored within buildings is insignificant. It concludes any of the methods above, if applied correctly, are adequate for estimating peak water levels when the grid resolution is below 10m. A grid size of 2m or less is required for accurately representing flow vectors around each building. For the current project the model is required to estimate peak design levels across the two models and as such the use of a 5m grid is appropriate.

Buildings were represented using the stubby building approach as per EA document '379_05 Computational Modelling to Assess Flood and Coastal Risk'.

Buildings were stamped into the underlying topography, raising up the elevation of the building footprint using a threshold value of 0.30m. Building footprints were extracted from OS MasterMap. The building footprints were also assigned a Manning's n topography roughness of 0.30 to account for the tortuous flow path taken by water through these features.

6.7 Topographic roughness

Areas of spatially varying topographic roughness across the two models were defined based on OS MasterMap data. The land use polygons were split into several categories and associated with a roughness coefficient in the form of a Manning's n value, these are shown in Table 6-4. Both models used these land use categories and roughness coefficients.

There are no specific guidelines for setting floodplain roughness values, therefore, modelling judgement is required to select appropriate values.

[9] Engineers Australia (2012). Revision Project 15: Two dimensional simulations in urban areas; representation of buildings in 2D numerical flood models. Australian Rainfall and Runoff.

Table 6-4: Manning's m roughness values based on OS MasterMap data

Manning's n	Topography Category	Land Type
0.030	10	Default Floodplain Value (Water)
0.300	1	Buildings
0.100	2	Structures
0.030	3	Inland and Coastal Water
0.070	4	Natural Surface and Gardens
0.025	5	Manmade Surface Roads and Paths
0.100	6	Trees, Roughland and Scrub
0.046	7	Marsh, reeds or saltmarsh

6.8 Inundation model boundary approach

Two types of hydraulic boundaries were required:

- *A still water boundary was located offshore, allowing the propagation of the tide and surge into the model domain*
- *Wave overtopping boundaries, along the coastal frontage to inject wave water into the model at the location of the flood defences*

6.8.1 Water level boundaries

Design tide curves were generated for eight AEP present day 2018 events and two climate change epochs (2068 and 2118) for the 0.5% AEP event based on UKCP09 and NPPF estimates for change (Table 6-1). This process used information from three principle sources of data:

- *Extreme still water sea level estimates*
- *A design astronomical tide*
- *A design surge shape*

Extreme still water sea level estimates were obtained from the Coastal Flood Boundary Dataset (CFBD) produced in 2018 using the most suitable chainage point for each of the models. These estimates are provided for a baseline year of 2017 and were updated to account for sea level rise to the year of project start (2018) using UKCP09 and NPPF guidance. Surge profiles for each of the study areas were chosen based on their proximity to the nearest surge estimate; for both sites this was at Avonmouth.

Environment Agency guidance[10] was followed to develop tidal curves for the two models. A Highest Astronomical Tide (HAT) tide was extracted from a local gauge, smoothed and resampled to 15-minute intervals. The peak of the design surge profile was aligned to coincide with the low tide level before the maximum peak of the HAT to generate the tidal profile. The design surge profile was multiplied by a growth factor and added to the HAT to form a new tide curve for each event. An example of the tide curve generated for the 0.5% AEP 2018 event at Woodspring Bay is shown in Figure 6-2.

The CFB guidance allows the placement of the surge peak at any point from low to high tide. In this instance, aligning the peak surge with the trough increases the overall volume of the tidal graph by a greater volume than aligning the peak surge with the peak

[10] Environment Agency (2011) 'Using the national coastal flood boundary data for the coasts of England and Wales: Operational Instruction 490_11

tide. Alignment with the trough was therefore chosen as it was the more conservative option.

The best practice guidance recommends that three tidal cycles are modelled for tidal inundation studies, with the tidal peak occurring during the second cycle. For this study the surge profile affected four tidal cycles and as such it was decided to run these four cycles through the model.

Due to the spatial variation in extreme water levels along the coastline of each model, a HX (Water Level (Head) from and eXternal source) boundary was used. This boundary interpolates the water level between multiple tide curve locations situated along the model boundary.

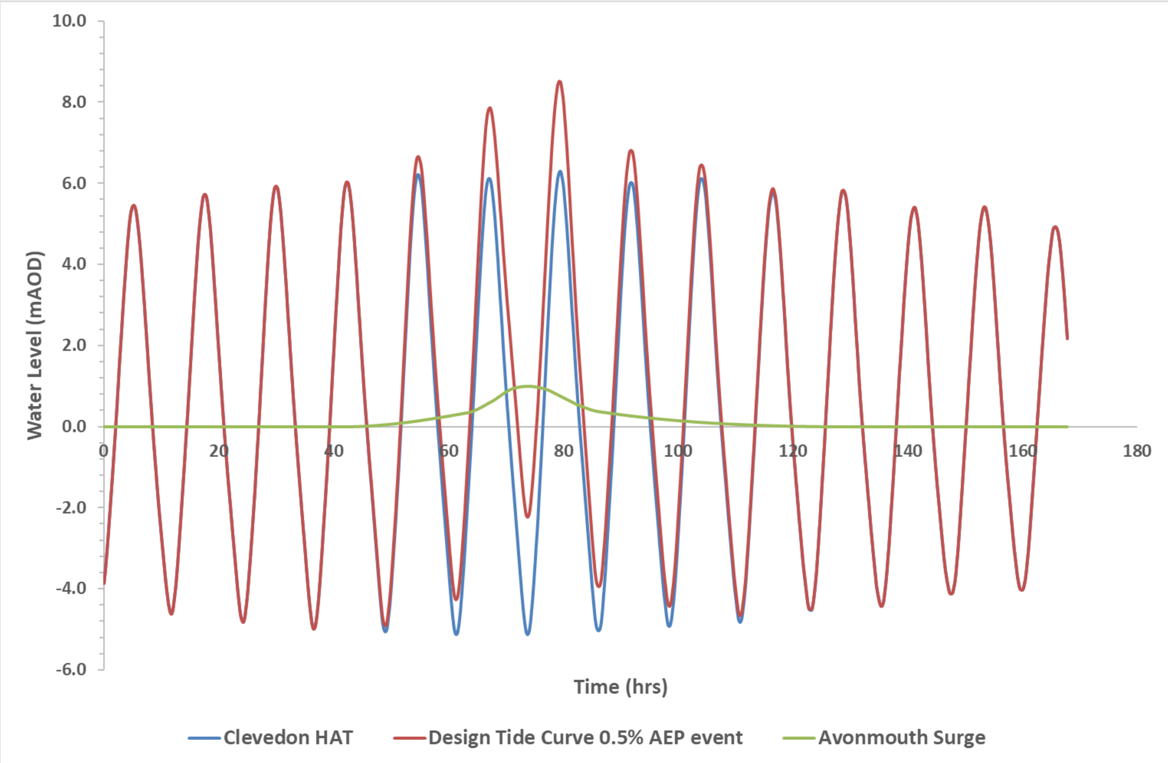


Figure 6-2: Tide curve example for Woodspring Bay, 0.5% AEP (present day)

A HT boundary was also included at Weston-super-Mare in the Woodspring Bay model (chainage 350) tying into high ground in the north at Claremont Crescent and high ground at the mouth of the River Axe. This was included as during the undefended and more extreme defended scenarios, still water flooding is dominant as the floodwaters pass between Weston-super-Mare, Sand Bay and Woodspring Bay due to the low topography. The chainage used in the HT boundary was chosen based on the higher of the two CFB extreme sea level points in the Weston-super-Mare area (chainage 350 is 0.9m greater than chainage 348). This conservative, higher water level, was chosen because the model boundary at the southern end partly travels up the River Axe estuary before tying into high ground. Levels would be expected to rise as they begin to propagate up the Axe estuary as they are constricted, and therefore use of the higher CFB level should suitably cover any increase in estuary water level..

The models were run using design tide curves. The Woodspring Bay model uses six tide curves with interpolation applied between each point, and in Severn House Farm there were five.

6.8.2 Wave overtopping boundaries

The wave overtopping discharges were injected into the inundation models using ST boundaries (Flow versus time (m3/s)) applied directly behind the coastal defence line.

ST boundaries apply flow evenly along the inflow line and activate single model boundary cells that the inflow line sits on top of. This approach was applied where there was a raised coastal defence. Overtopping was calculated in $m^3/s/m$ and to represent wave overtopping within the inundation model, the volumes were adjusted using a multiplication factor. The factor was calculated by dividing the defence length by the number of model cells that the ST inflow line activated. If for example, a 100m length of defence in a 10m resolution model activated 10 model cells along the boundary, then the multiplication factor would be 10. If, however, only 9 model cells were activated by the ST boundary, then the multiplication factor would be increased to 11.11, to provide the correct overtopping volume into the model for the length of defence being modelled.

An ST line does not account for the momentum of waves crashing over the coastal defence, and therefore in locations where there was no raised coastal defence, or the topography was sloping seaward, overtopping volumes from an ST line may simply run seaward and show very little overtopping flood risk. In both Woodspring Bay and Severn House Farm model the raised coastal defences and low-lying topography behind meant that use of an ST line was appropriate as overtopping inflows were forced landward.

6.9 TUFLOW model build

Two TUFLOW models were generated as part of this study as described in Table 6-5. Each model and the key model build details and decisions is discussed in the remainder of this chapter.

Table 6-5: Model names and area of interest

Model name	Area of interest
Woodspring Bay	Woodspring Bay, Marine Lake and Kewstoke
Severn House Farm	Oldbury-on Severn, Shepperdine, Berkeley, Sharpness

6.9.1 Woodspring Bay TUFLOW model domain

The TUFLOW domain follows the shoreline, extending from Weston Bay in the south to Clevedon in the north where the model boundary ties into high ground (Figure 6-3). The grid orientation is aligned along the dominant flow path along Woodspring Bay, running from south west to north east roughly at a 40° angle.

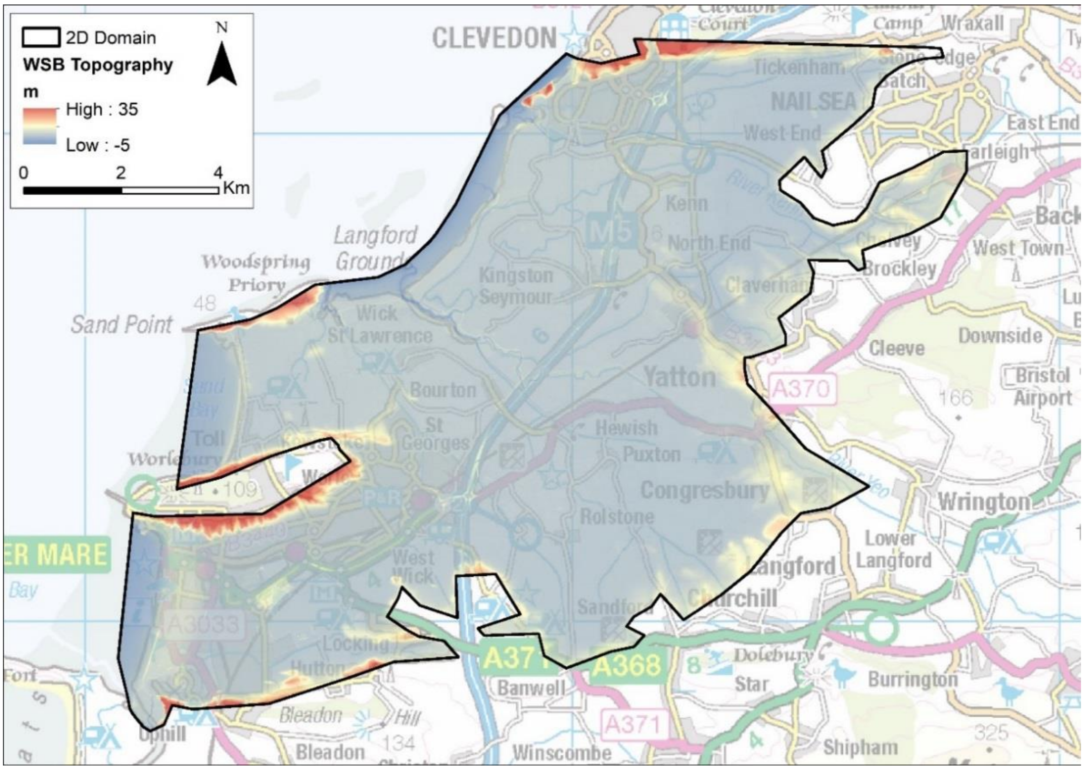


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Figure 6-3: Woodspring Bay model domain

6.9.2 Woodspring Bay TUFLOW model geometry

Coastal defences included in the Woodspring Bay model are shown in Figure 6-4. The coastal defences consist of a mixture of grassed earth embankments, masonry sea walls, sand dunes and in places rock armour is present. These defences were removed from the model topography in the undefended simulations.

The topographic modifications included in the model are detailed in Table 6-6. The CYTB defences shown in Figure 6-4 represent defences from the Congresbury Yeo Tidal Banks scheme.

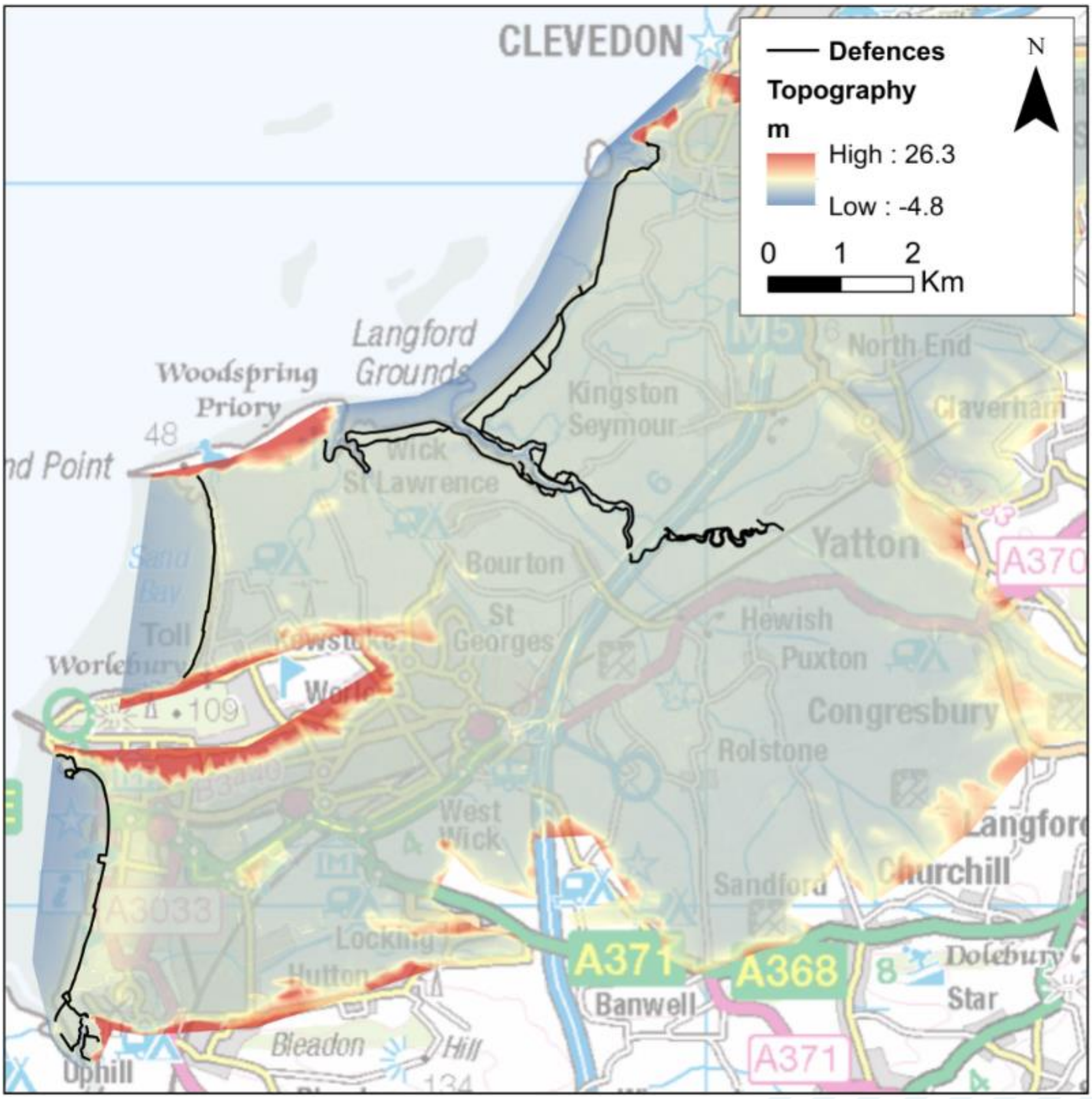

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Figure 6-4: Woodspring Bay location of coastal defences included in model

Table 6-6: Modifications to ground model

ID Layer Name	Command	Description
Base model DTM data		
lt_lidar.txt	Read Grid Zpts	TUFLOW reads in a text file of grid points attributed with elevations derived from 2m low tide LIDAR data flown 2013 provided by the Environment Agency.
lidar_dtm.txt	Read Grid Zpts	TUFLOW reads in text file of grid points attributed with elevations derived from 1m composite LIDAR data provided by the Environment Agency.
Additional Modifications		
2d_zsh_WSB_005_Defences_L.shp 2d_zsh_WSB_005_Defences_P.shp	Read GIS Z Shape	Defence crest levels stamped into the 2D domain based on Royal Haskoning 2007 survey data.
2d_zsh_WSB_Previous_Defences_001_L.shp 2d_zsh_WSB_Previous_Defences_001_P.shp	Read GIS Z Shape	Defence crest levels stamped into the 2D domain taken from previous modelling, source unknown.
2d_zsh_CYTB_scheme_003_L.shp	Read GIS Z Shape	Defence crest levels stamped into the 2D domain taken from CYTB Construction Design Manual (CDM) Regulations Health & Safety File – refer to Chapter 6.2.4.
2d_zsh_CYTB_scheme_settlement_002_L.shp	Read GIS Z Shape	Defence crest levels stamped into the 2D domain for the climate change setup. Levels taken from the CYTB H&S file which provided crest levels for each defence section in 25-years' time – refer to Chapter 6.2.4.
2d_zsh_WSB_oldbridge_Gully_001_L.shp 2d_zsh_WSB_oldbridge_Gully_002_P.shp	Read GIS Z Shape GULLY	Gully line taken from previous model – levels have been taken from 2017 1m composite LIDAR data provided by the Environment Agency.
2d_zsh_WSB_Banwell_Gully_002_L.shp 2d_zsh_WSB_Banwell_Gully_002_P.shp	Read GIS Z Shape GULLY	Gully line taken from previous model – levels have been taken from 2017 1m composite LIDAR data provided by the Environment Agency.
2d_zsh_WSB_blindyeo_Gully_003_L.shp 2d_zsh_WSB_blindyeo_Gully_003_P.shp	Read GIS Z Shape GULLY	Gully line taken from previous model – levels have been taken from 2017 1m composite LIDAR data provided by the Environment Agency.

2d_zsh_WSB_landyeo_Gully_002_L.shp 2d_zsh_WSB_landyeo_Gully_003_P.shp	Read GIS Z Shape GULLY	Gully line taken from previous model – levels have been taken from 2017 1m composite LIDAR data provided by the Environment Agency.
2d_zsh_WSB_ken_Gully_002_L.shp 2d_zsh_WSB_ken_Gully_003_P.shp	Read GIS Z Shape GULLY	Gully line taken from previous model – levels have been taken from 2017 1m composite LIDAR data provided by the Environment Agency.
2d_zsh_WSB_Uphill_Gully_002_L.shp 2d_zsh_WSB_Uphill_Gully_002_P.shp	Read GIS Z Shape GULLY	Gully line taken from previous model – levels have been taken from 2017 1m composite LIDAR data provided by the Environment Agency.
2d_zsh_WSB_CongresYeo_Gully_001_L.shp 2d_zsh_WSB_CongresYeo_Gully_001_P.shp	Read GIS Z Shape GULLY	Gully line along watercourse, additional in new model setup. Levels have been taken from 2017 1m composite LIDAR data provided by the Environment Agency.
2d_zsh_WSB_Additional_Gullies_001_L.shp 2d_zsh_WSB_Additional_Gullies_001_P.shp	Read GIS Z Shape GULLY	Gully lines to allow creation of flow pathways, additional in new model setup. Levels have been taken from 2017 1m composite LIDAR data provided by the Environment Agency.
2d_zsh_WSB_Buildings_001_R.shp	Read GIS Z Shape	Stubby Buildings. Building footprints raised by 0.3m.
2d_zsh_WSB_defence_removal_003.shp 2d_zsh_WSB_defence_removal_004.shp 2d_zsh_WSB_defence_removal_005.shp	Read GIS Z Shape	Flattens the defence topography for the undefended run.
2d_zsh_WSB_DTM_patches_001_R.shp	Read GIS Z Shape	Infills the LIDAR to account for areas where embankments have culverts intercepting.

6.9.3 Congresbury Yeo Tidal Bank Scheme

The Congresbury Yeo Tidal Banks (CYTB) scheme is a tidal defence scheme that was completed to provide improved flood protection for more than 4,100 homes and businesses in North Somerset. The CYTB scheme involved upgrading the existing tidal banks (widening and raising) along the Congresbury Yeo estuary, and the construction of three new sections of bank. The CYTB scheme is located between the towns of Clevedon (to the north) and Weston-super-Mare (to the south) and detailed on Figure 6-5.

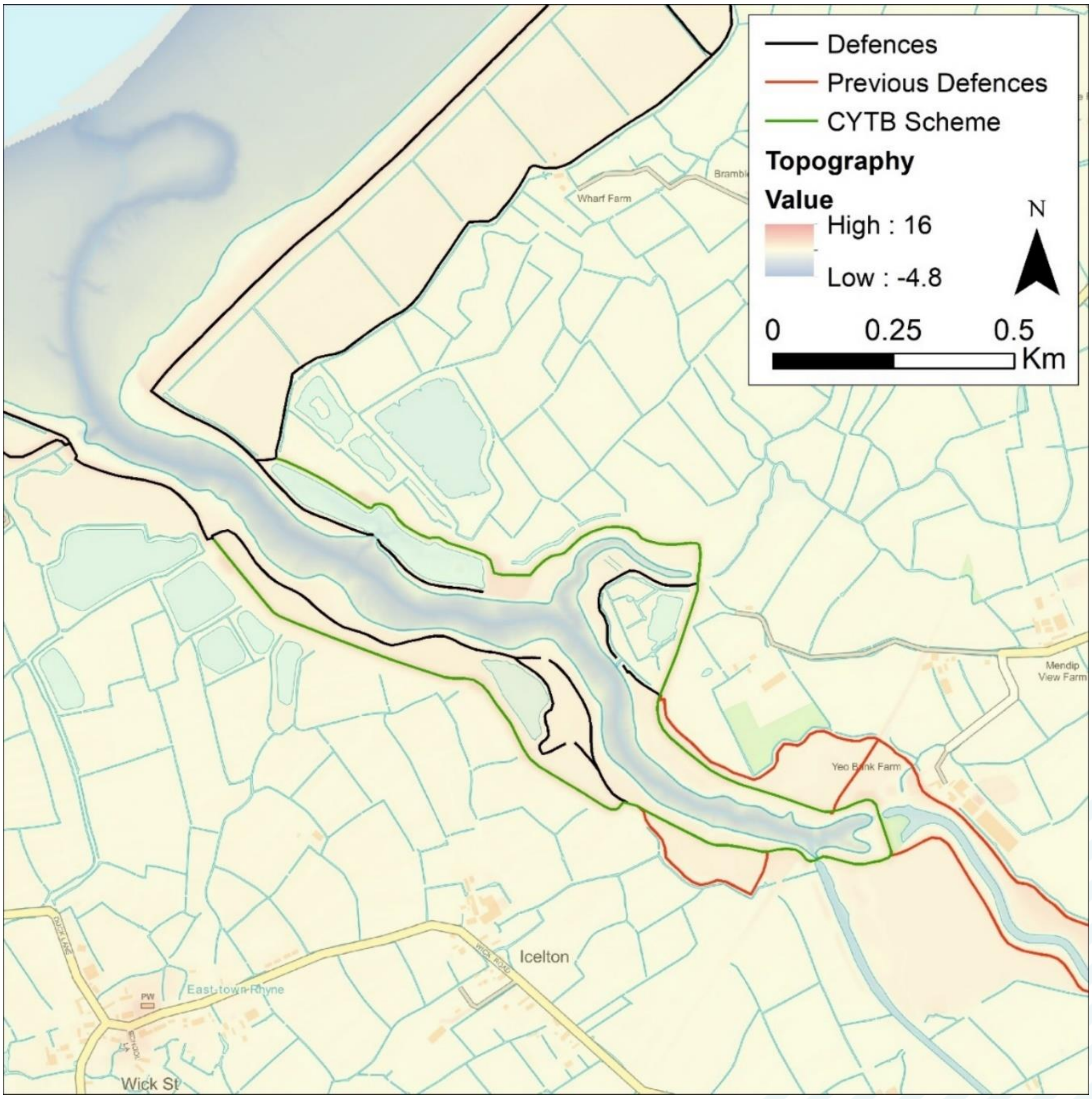

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Figure 6-5: Congresbury Yeo Tidal Bank Scheme Defences

The CYTB scheme (site layout shown on Figure 6-6) was included in the Woodspring Bay model in two forms:

- *Present day design setup: Defence levels included based on design crest level criteria as detailed in the CYTB Construction Design Manual (CDM) Regulations Health & Safety File[11], described as CYTB H&S file from this point, that provided a description of crest levels and overview map of the scheme.*
- *Climate change setup: The CYTB H&S file detailed the CYTB scheme to have a design life of 25-years and provided crest levels for each defence section in 25-years. These levels have been used in the climate change model setup.*

A brief description of the present-day design sections and levels are as follows:

- *North bank:*
- *Raising of existing north embankment to 8.62mAOD from chainage 0-400m;*
- *Raising of existing north embankment to 9.49m AOD from chainage 400-630m;*
- *Raising of existing north embankment to 8.62mAOD from chainage 630-850m;*
- *Raising of existing north embankment to (8.60 on site location) 8.49mAOD from chainage 850-1250m;*
- *Raising of existing north embankment to 9.11mAOD from chainage 1250-1400m;*
- *Raising of existing north embankment to 8.49mAOD from chainage 1400-1950m.*
- *South bank:*
- *Raising of existing south embankment to 8.62mAOD from chainage 0-650m;*
- *A new south embankment to 9.49mAOD from chainage 650-1719m.*

A brief description of the climate change sections (25-years' time) and levels are as follows:

- *North bank:*
- *Raised embankments were built to have a crest height of 8.44mAOD (north bank) after 25 years.*
- *South bank:*
- *Raised embankments were built to have a crest height of 8.57mAOD (south bank) after 25 years;*
- *New embankments were built to have a crest height of 8.82mAOD after 25 years (chainage 650-1719m south bank).*

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[11] Congresbury Yeo Tidal Banks. CDM Regulations Health & Safety File. Environment Agency. February 2019.

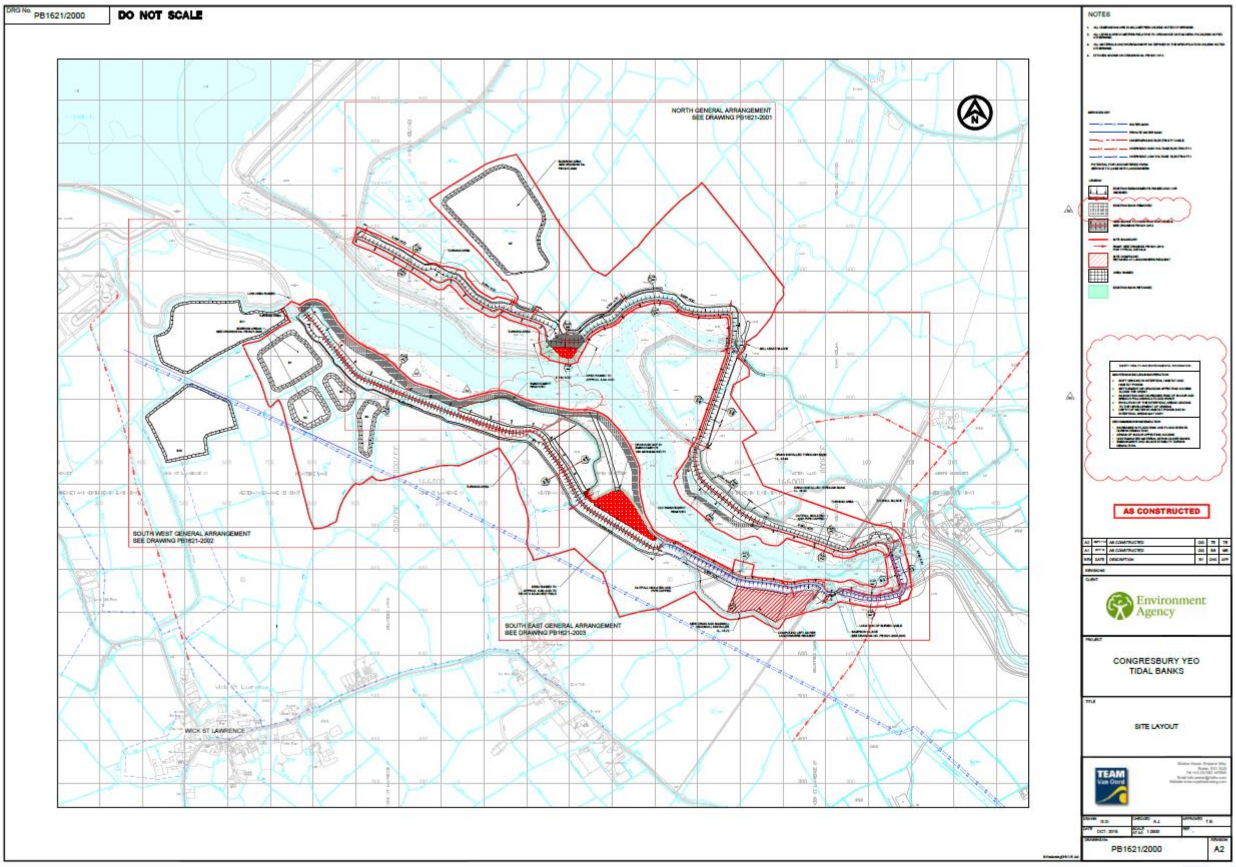

Figure 6-6: Congresbury Yeo Tidal Banks site boundary (PB1621/2000)

6.9.4 Woodspring Bay 1D Network

There are 30 culverts within the Woodspring Bay model domain as located on Figure 6-7 for the defended scenario. Table 6-7 details the key culvert attributes included in the model for the defended scenario while the source of the data is detailed in Table 6-7. Most data came from the previous model files or NAFRA. In areas where there was no data available, assumptions were made. The location of many culverts within the model are associated with the two main infrastructure mechanisms, the railway line and the M5 motorway which intersect the model domain.

For the undefended model configuration, in areas where defences were removed from the model DTM, culverts were also removed from the model to allow an undefended state to be represented as accurately as possible. Figure 6-8 shows the undefended scenario culvert locations. All remaining culverts in the undefended scenario were open (no defence structures included).

The culvert data was added to the model as a 1D network line connected to a 2D boundary at the end of each culvert using the following files:

Defended:

- *1d_nwk_WSB_culverts_004_L.shp*
- *2d_bc_WSB_culverts_002_P.shp*

Undefended:

- *1d_nwk_WSB_Udef_culverts_004_L.shp*
- *2d_bc_WSB_Udef_culverts_003_P.shp*

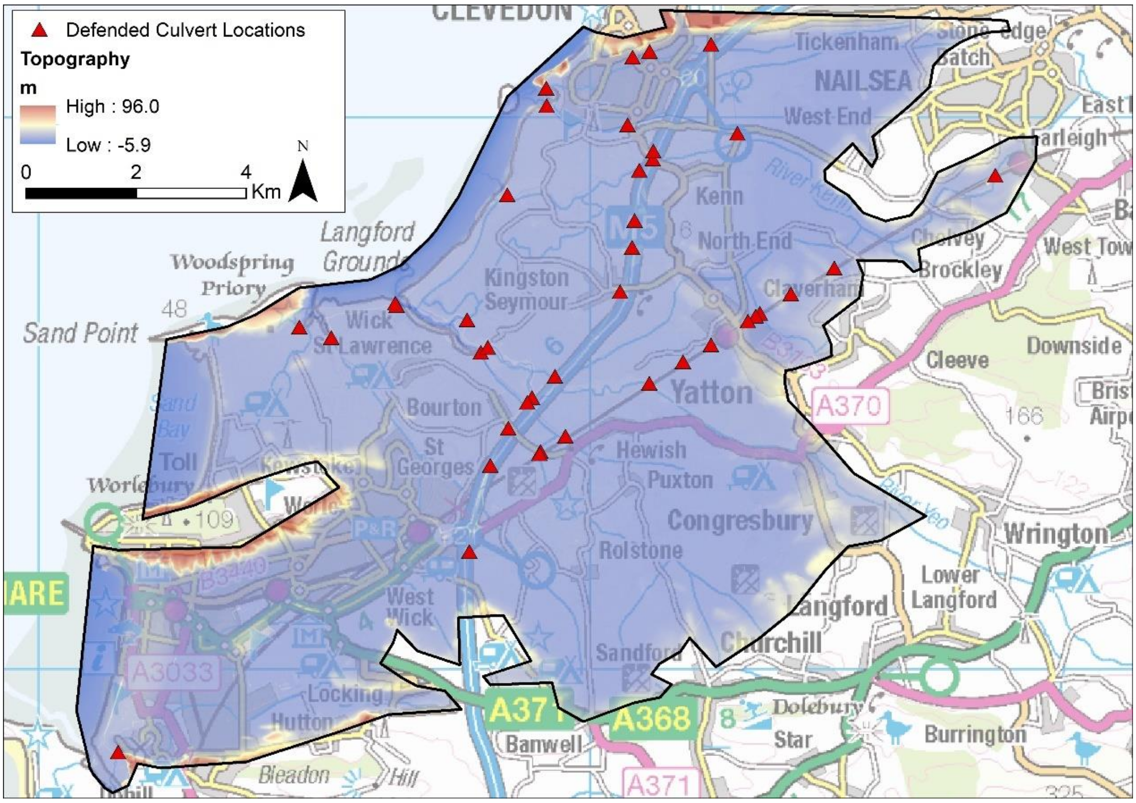

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Figure 6-7: Location of culverts for the defended model setup

Table 6-7: Details of culvert attributes included in WSB defended modelling

Culvert ID	Type	Len_or_ANA	n_nF_Cd	US_Invert	DS_Invert	Width_or_D	Height_or_
LYeo2	R	248	0.020	4.45	4.44	1.00	1.00
LYeo3	R	216	0.020	4.51	4.49	1.00	1.00
LYeo4	C	37	0.025	4.65	4.50	0.30	0.00
RiverKenn3	R	70	0.020	3.75	3.66	1.00	1.00
RiverKenn2	R	17	0.020	3.46	3.44	1.00	1.00
Old_BR	RU	10	0.020	1.40	1.30	1.00	1.00
NewBow	RU	21	0.016	3.04	2.00	1.90	4.02
UphillSluice	RU	52	0.016	3.20	3.00	5.00	3.00
Tutshill Slu	RU	26	0.016	2.26	2.00	2.80	3.40
BYeo1	RU	30	0.020	0.46	0.46	1.00	1.00
M5_4	R	100	0.024	4.90	4.45	1.20	4.90
M5_5	R	138	0.024	4.10	5.20	1.20	4.90
M5_6	R	53	0.025	3.70	4.10	3.00	3.06
M5_1	R	60	0.016	3.10	3.40	3.35	2.41
M5_2	R	72	0.016	2.53	2.44	6.10	2.44
M5_3	R	58	0.016	2.09	2.33	3.60	1.44
M5_7	R	67	0.024	4.90	4.80	1.20	4.90
M5_8	R	56	0.016	4.66	4.60	2.40	3.61
M5_9	R	90	0.024	5.40	5.10	1.20	4.90
M5_10	R	73	0.016	4.70	4.50	2.40	3.35
Hew_1	R	21	0.025	4.70	3.80	1.68	4.20
Hew_2	R	25	0.025	4.20	4.70	1.68	4.20
oldbridge	R	60	0.025	2.39	2.41	3.60	1.44
129_72	R	27	0.025	3.70	3.29	3.00	3.40
128_74	R	39	0.025	5.70	4.60	1.25	4.70
129_39	R	29	0.025	4.58	4.25	1.40	4.90
129_67	R	21	0.025	3.40	3.90	1.25	3.40
130_6	R	38	0.025	4.30	4.00	1.25	4.60

Culvert ID	Type	Len_or_ANA	n_nF_Cd	US_Invert	DS_Invert	Width_or_D	Height_or_
130_44	R	67	0.025	5.10	4.55	1.70	5.10
130_73	R	45	0.025	4.30	4.45	1.70	5.10
131_30	R	45	0.025	4.70	4.60	6.10	5.10
LYeo1	RU	15.8	0.020	4.30	3.10	1.20	0.50
RiverKenn1	CU	307	0.024	3.30	1.12	1.50	0.00
BYeo2	R	23	0.020	1.00	1.00	1.00	1.00
Kenn_Blind	R	21	0.020	2.20	0.90	1.00	1.00
Brookside	R	37	0.020	6.94	6.79	1.00	1.00
WSB2	RU	66	0.020	4.89	-0.20	1.00	1.00
WSB1	RU	90	0.020	5.00	0.30	1.00	1.00
WSB3	RU	33	0.020	3.58	3.30	1.00	1.00
Sand_Rhyne	RU	15	0.020	3.68	3.00	1.00	1.00

Type: C (circular), R (rectangular), RU (rectangular unidirectional flow), CU (circular unidirectional flow)

Table 6-8: Details of culvert data sources

Culvert ID	Data Source						
	Type	Len_or_ANA	n_nF_Cd	US_Invert	DS_Invert	Width_or_D	Height_or_
LYeo2	Assumed	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
LYeo3	Assumed	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
LYeo4	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
RiverKenn3	Assumed	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
RiverKenn2	Assumed	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
Old_BR	NAFRA	Inferred from	Manning's N	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m

Culvert ID	Data Source						
	Type	Len_or_ANA	n_nF_Cd	US_Invert	DS_Invert	Width_or_D	Height_or_
		LIDAR and mapping	Value				
NewBow	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
UphillSluice	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
Tutshill Slu	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Previous Model		Previous Model	Previous Model
BYeo1	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
M5_4	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
M5_5	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
M5_6	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
M5_1	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
M5_2	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
M5_3	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Previous Model		Previous Model	Previous Model
M5_7	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Previous Model		Previous Model	Previous Model
M5_8	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
M5_9	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model

Culvert ID	Data Source						
	Type	Len_or_ANA	n_nF_Cd	US_Invert	DS_Invert	Width_or_D	Height_or_
M5_10	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
131_74	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
Hew_1	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
Hew_2	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Previous Model		Previous Model	Previous Model
Oldbridge	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
129_72	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
128_74	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
129_39	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
129_67	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Previous Model		Previous Model	Previous Model
130_6	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
130_44	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
130_73	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
131_30	Previous Model	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
LYeo1	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA

Culvert ID	Data Source						
	Type	Len_or_ANA	n_nF_Cd	US_Invert	DS_Invert	Width_or_D	Height_or_
RiverKenn1	NAFRA/Previous Model	NAFRA	Manning's N Value	Inferred from LIDAR		Previous Model	Previous Model
BYeo2	Assumed	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
Kenn_Blind	Assumed	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
Brookside	Assumed	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
WSB2	NAFRA/Assumed	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
WSB1	NARFA/Assumed	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
WSB3	NAFRA/Assumed	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
Sand_Rhyne	NAFRA/Assumed	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m

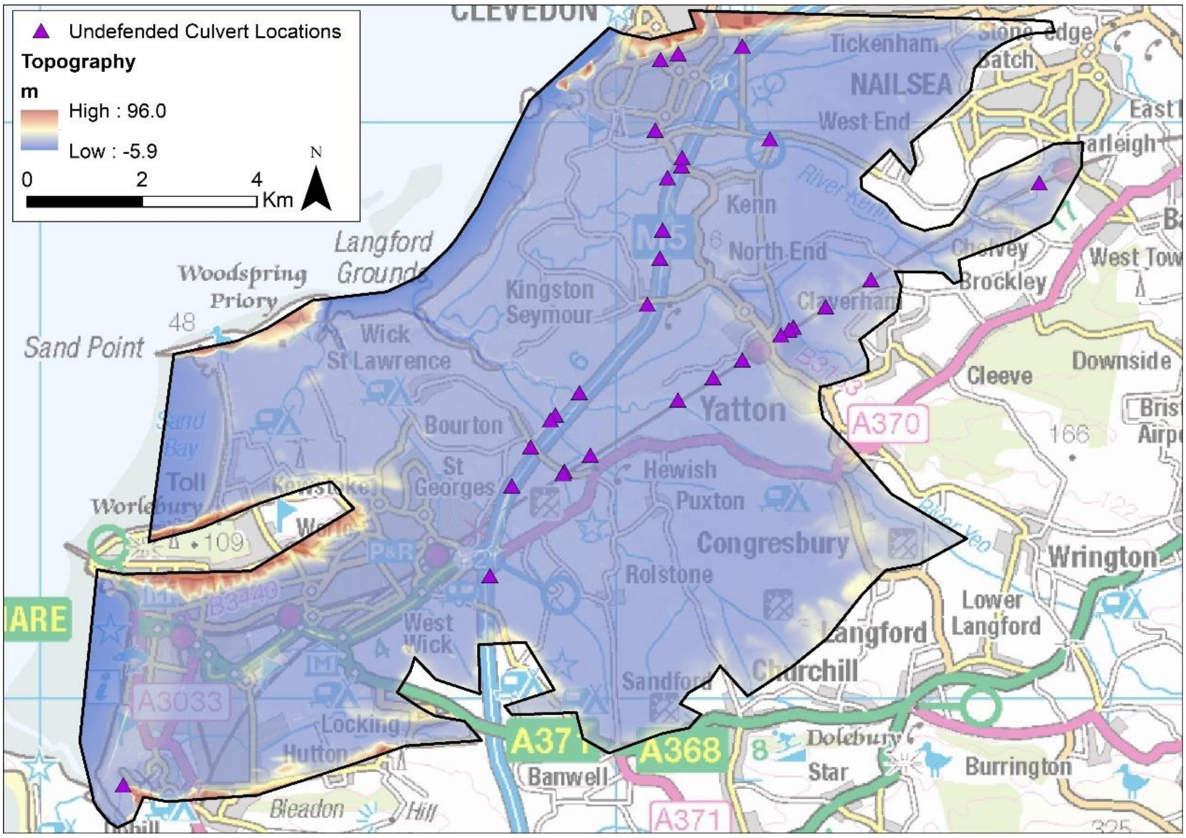

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Figure 6-8: Location of culverts for the undefended model setup

6.9.5 Woodspring Bay model boundaries

A full description of the data used to generate the model boundary conditions can be found in the Woodspring and Severn House model boundary report[12] that sits alongside this reporting. A summary of flood inundation model boundaries is provided below.

A HX and a HT offshore water level boundary was used to drive the Woodspring Bay model as located on Figure 6-9 and the CFB extreme sea level point used in the tide curve generation are identified. The HX boundary was used to define the spatial variation in extreme sea levels along the coastline while the HT applied a single head-time boundary for application at Weston Bay. The data used to generate the tide curves for the Woodspring Bay model is summarised in Table 6-9.

ST boundaries were used to apply wave overtopping discharges on the landward side of the coastal defence network as shown on Figure 6-10.

A HQ boundary was used to allow water to flow out of the model, located on the railway immediately west of Oldmixon (Figure 6-9).

[12] Woodspring and Severn House Model Boundary Report. March 2020. JBA Consulting.

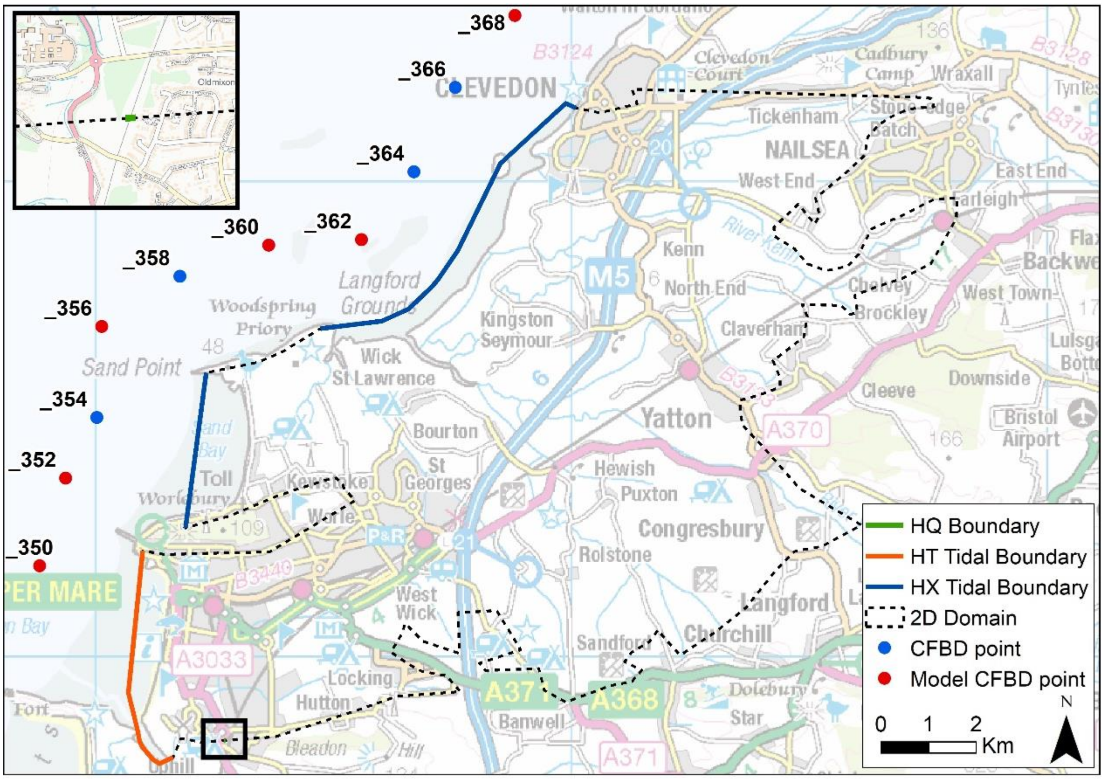

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Figure 6-9: Woodspring Bay water level boundary and CFB point location

Table 6-9: CFB points used in modelling

Model	Extreme Sea Level	Astronomical Tide	Surge Shape Location	UKCP09 Grid
Woodspring Bay	CFB Chainage Point _350	Weston-super-Mare	Avonmouth	23888
	CFB Chainage Point _352			23888
	CFB Chainage Point _356			23683
	CFB Chainage Point _360	Clevedon		23684
	CFB Chainage Point _362			23684
	CFB Chainage Point _368			23684

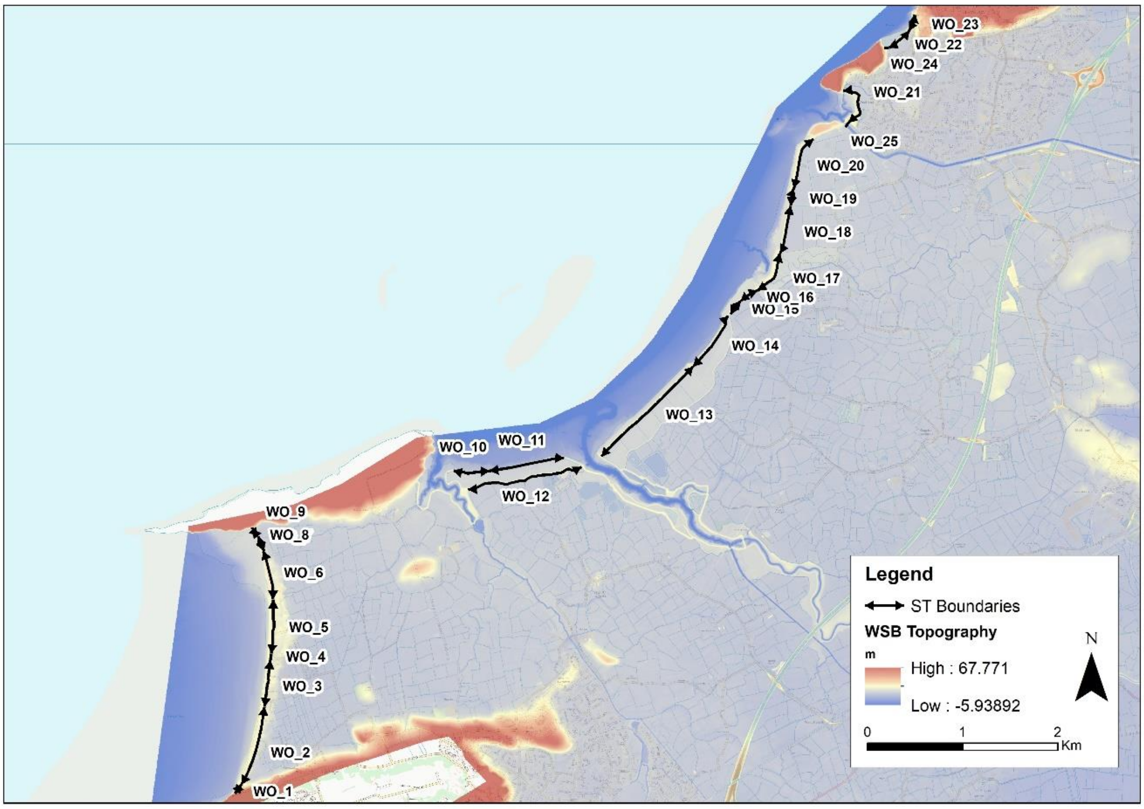

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Figure 6-10: Location of wave overtopping ST boundaries for WSB

6.9.6 Woodspring Bay TUFLOW stability fixes

There were no stability patches applied within the WSB model setup.

6.9.7 Undefended Modelling

Due to the low-lying topography within the Woodspring Bay model, the dominant flood risk is still water in the undefended scenario. As such, no defences were schematised for wave overtopping inflows in the undefended scenario for Woodspring Bay. Formal raised defences were removed from the model topography and defacto defences such as the M5 and promenade at Weston-super-Mare are left in the model topography. The defence removal included the following decisions:

- *the raised wall and path behind Marine Lake was lowered down to the lower levels on the landward side*
- *the dunes at Sand Bay were flattened down to road level as per previous modelling*
- *the dunes at the southern end of Weston Bay were removed as per previous modelling*
- *raised embankments at Uphill were removed*
- *primary and secondary wall defences on Royal Parade and the north of Marine Parade at Weston-super-Mare were removed*

Culverts were also removed from the model where defences were removed. Defence removal was applied within the model using the following files:

- *2d_zsh_WSB_defence_removal_003_R.shp*
- *2d_zsh_WSB_defence_removal_004_R.shp*

- *2d_zsh_WSB_defence_removal_005_R.shp*

Defended overtopping inflows were included along Sand Bay coastal frontage (WO_1 to WO_9) to limit the difference between the defended and undefended flood extents as still water flooding does not occur at Sand Bay until the larger events. The undefended overtopping at Sand Bay was applied using the following file:

- *2d_bc_WSB_WO_UDef_001_L.shp*

6.9.8 Severn House Farm TUFLOW model domain

The TUFLOW domain follows the Bristol Channel south bank, extending from Aust in the South to Sharpness in the North where the model water level boundary ties into high ground. The grid orientation is aligned south west to north east roughly at a 40° angle (Figure 6-11).

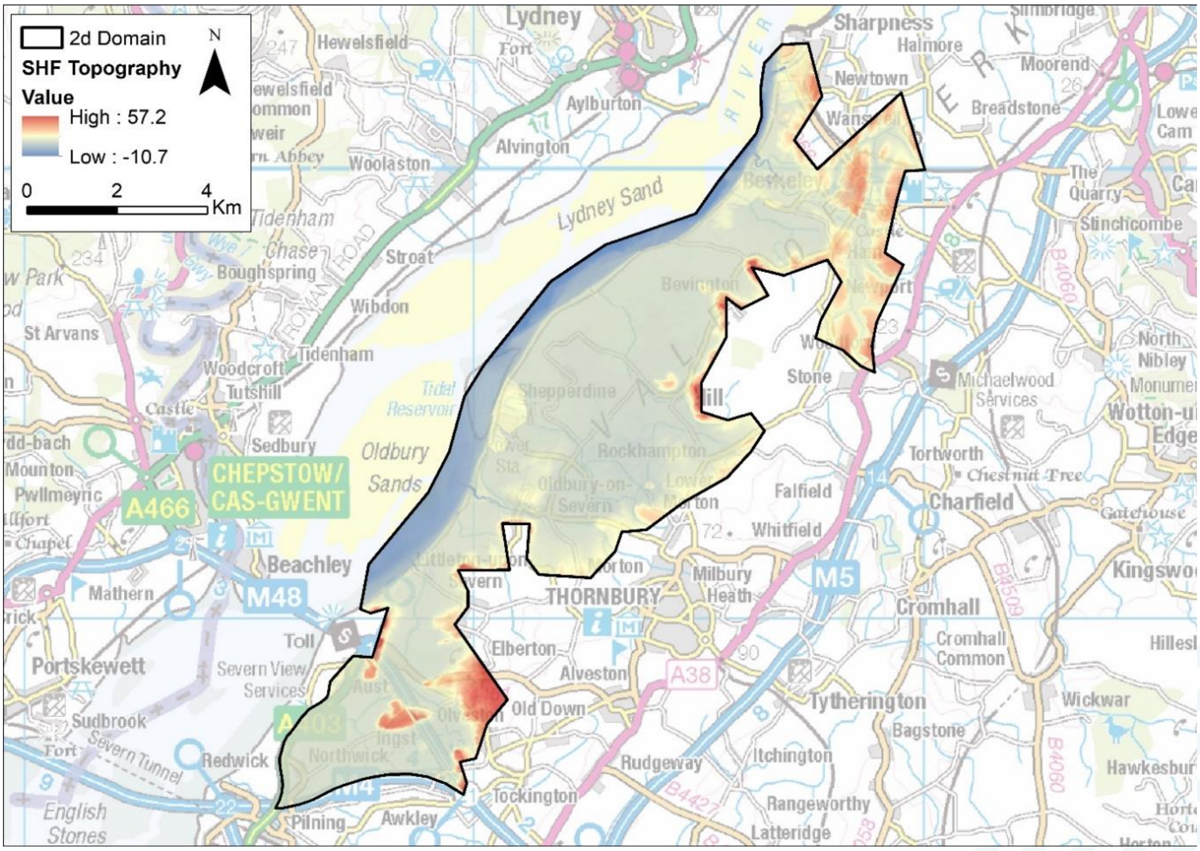

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Figure 6-11: Severn House Farm 2D model domain

6.9.9 Severn House Farm TUFLOW model geometry

Coastal defences included in the Severn House Farm model are shown on Figure 6-12. The coastal defences consist predominately of grassed earth embankments with some rock revetment present. These defences were removed from the model topography in the undefended simulations.

The topographic modifications included in the model are detailed in Table 6-10.

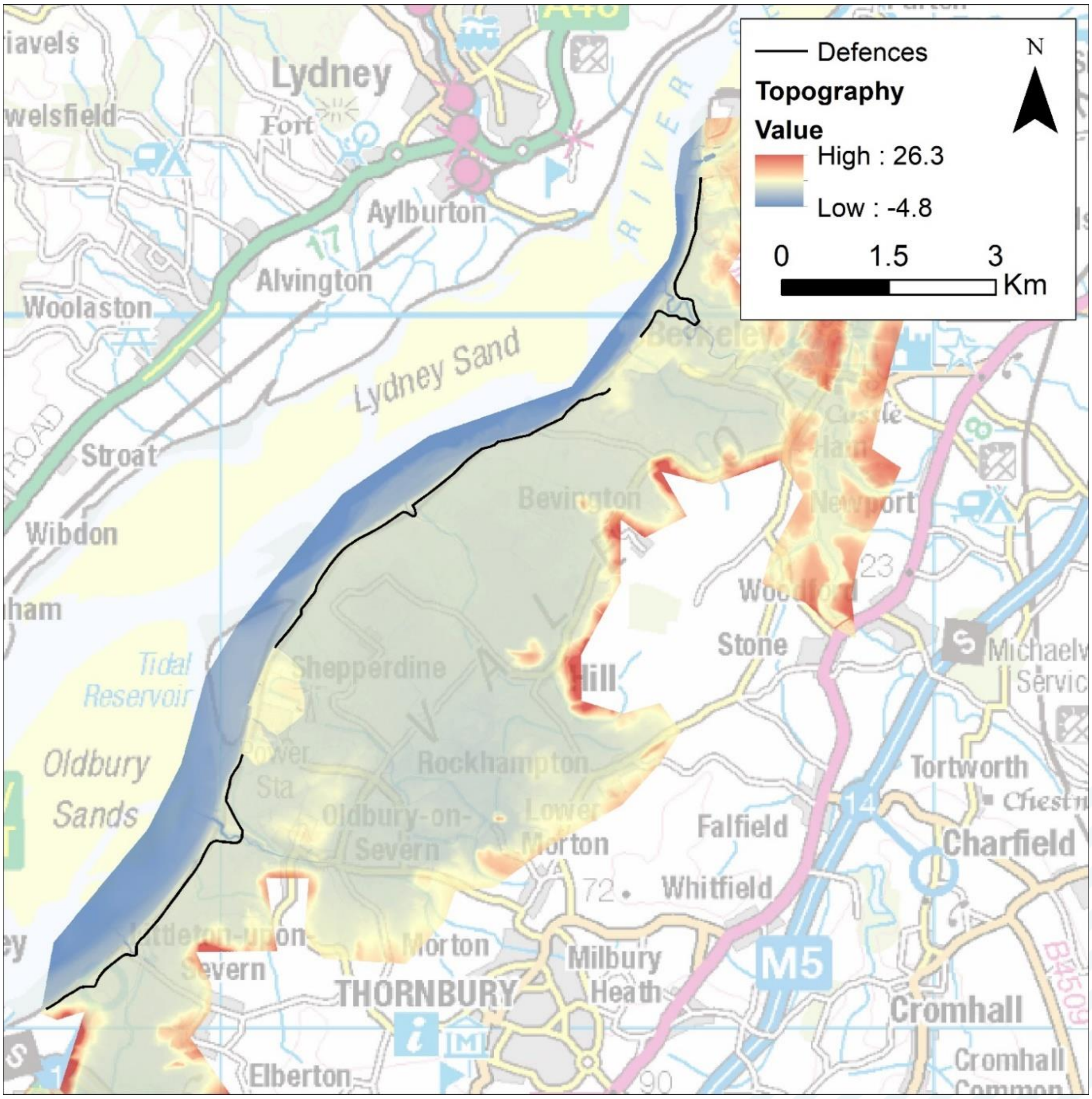

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Figure 6-12: Severn House Farm coastal defences included in the model

Table 6-10: Modifications to ground model

ID Layer Name	Command (e.g. "Read MI Z Shape AD")	Purpose of terrain modification and source of elevation data
Base model DTM data		
2m_Lidar_SHF.txt	Read Grid Zpts	TUFLOW reads in a text file of grid points attributed with elevations derived from 2m LIDAR data provided by the Environment Agency.
2m_Lidar_SHF_LT.txt	Read Grid Zpts	TUFLOW reads in a text file of grid points attributed with elevations derived from 2m low tide LIDAR data flown 2013 provided by the Environment Agency.
Additional Modifications		
2d_zsh_SHF_001_Defences_L.shp 2d_zsh_SHF_001_Defences_P.shp	Read GIS Z shape THICK	Defence crest levels stamped into the 2D domain based on Royal Haskoning 2007 survey data.
2d_zsh_SHF_Gully_001_L.shp 2d_zsh_SHF_Gully_001_P.shp	Read GIS Z Shape GULLY	Gully lines along watercourses, new for this model build. Levels taken from 2017 1m composite LIDAR data provided by the Environment Agency.
2d_zsh_SHF_udef_culvert_001_L.shp 2d_zsh_SHF_udef_culvert_001_P.shp	Read GIS Z Shape	Pathways created to reinforce channel pathway following undefended defence removal
2d_zsh_SHF_Buildings_002_R.shp	Read GIS Z Shape	Stubby Buildings. Building footprints raised by 0.3m.
2d_zsh_defence_removal_SHF_001.shp	Read GIS Z Shape	Flattens raised defences to remove them for the undefended run.

6.9.10 Severn House Farm 1D Network

There are 28 culverts within the Severn House Farm model domain as located on Figure 6-13 for the defended scenario. Table 6-11 details the key culvert attributes included in the model for the defended scenario while the source of the data is detailed in Table 6-12. 12 culverts had no data available to inform the culvert network, therefore assumptions were made as detailed in the tables.

For the undefended model configuration, in areas where defences have been removed from the model DTM, culverts were also removed from the model to allow an undefended state to be represented as accurately as possible. Figure 6-14 shows the undefended scenario culvert locations. All remaining culverts in the undefended scenario were open (no defence structures included).

The culvert data was added to the model as a 1D network line connected to a 2D boundary at the end of each culvert using the following files:

Defended:

- *1d_nwk_SHF_culverts_001_L.shp*
- *2d_bc_SHF_culverts_001_P.shp*

Undefended:

- *1d_nwk_SHF_Udef_culverts_003_L.shp*
- *2d_bc_SHF_Udef_culverts_002_P.shp*

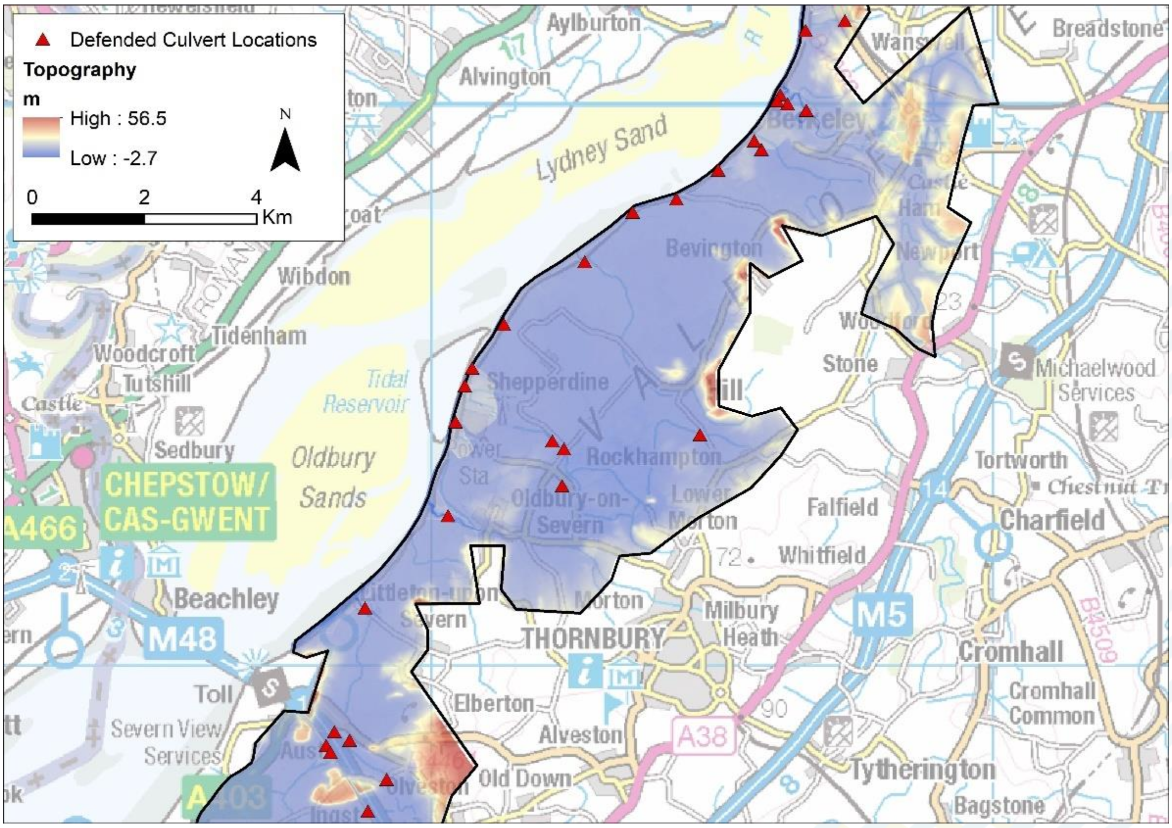

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Figure 6-13: Severn House Farm defended culvert locations

Table 6-11: Details of culvert attributes included in Severn House defended modelling

Culvert ID	Data Source							
	Comment	Type	Len_or_ANA	n_nF_Cd	US_Invert	DS_Invert	Width_or_D	Height_or_
1	Sanigar Outfall	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
2	Berkeley Pill Outfall	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
3	Woodlands Outfall	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
4	Worldsend Outfall	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
5	Hill Pill Outfall with penstock	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
6	Oldbury Power Station Outfall No. 2	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
7	Oldbury Power Station Outfall No. 1	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
8	Oldbury Pill Outfall	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
9	Littleton Pill Outfall	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
10	No data available - assumed	-	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
11	No data available – assumed	-	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
12	No data available – assumed	-	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
13	No data available – assumed	-	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
14	No data available – assumed	-	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
15	No data available – assumed	-	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
16	No data available – assumed	-	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
17	No data available – assumed	-	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
18	No data available – assumed	-	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
19	No data available – assumed	-	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
20*	Clapton Pill Outfall with actuated penstock	NAFRA and photographs	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
21	Conigre Pill - assumed	Inferred from channel size	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m
22	Conigre Outfall	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
23	Drainage ditch near Berkeley Pill	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
24	Drainage ditch near Berkeley Pill	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
25	Drainage ditch near Berkeley Pill -	Inferred from adjacent NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Inferred from adjacent NAFRA structure	Inferred from adjacent

Culvert ID	Data Source							
	Comment	Type	Len_or_ANA	n_nF_Cd	US_Invert	DS_Invert	Width_or_D	Height_or_
	assumed	structure						NAFRA structure
26	Windbound Outfall	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	NAFRA		NAFRA	NAFRA
27	Brough Head Outfall	NAFRA	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		NAFRA	NAFRA
28	Drainage ditch at Great Leaze Farm - assumed	-	Inferred from LIDAR and mapping	Manning's N Value	Inferred from LIDAR		Assumed 1 x 1m	Assumed 1 x 1m

*There are two culverts in this location, the coastal Clapton Pill Outfall with actuated penstock, and a second culvert connecting a short section of open channel with Clapton Pill. Due to model resolution we have modelled this as a single culvert based on the NAFRA culvert dimensions.

Table 6-12: Details of culvert data sources

Culvert ID	Type	Len_or_ANA	n_nF_Cd	US_Invert	DS_Invert	Width_or_D	Height_or_
1	RU	8	0.02	5.4	4	1	1
2	RU	10	0.02	2.2	1.55	1	1
3	CU	150	0.02	5.6	3.8	0.3	0
4	CU	13	0.02	4.9	4.6	0.65	0
5	RU	25	0.02	4.11	3.7	1	1
6	RU	30	0.02	8	4	1.15	1.15
7	RU	30	0.02	8.43	4.57	1.15	1.15
8	RU	11	0.02	2.45	0.8	2	2
9	RU	54	0.02	4.12	2.42	2	2
10	R	12	0.02	5.87	6.05	1	1
11	R	17	0.02	5.2	5.05	1	1
12	R	48	0.02	5.04	5.03	1	1
13	R	39	0.02	5.28	5.3	1	1
14	R	58	0.02	5.9	5.8	1	1
15	R	26	0.02	3.574	4.44	1	1
16	R	11	0.02	3.94	4.12	1	1
17	R	12	0.02	5.12	5.03	1	1
18	R	36	0.02	5.3	6	1	1
19	R	49	0.02	8.3	7.7	1	1
20	CU	85	0.02	3.3	3.2	1.6	0
21	R	28	0.02	4.7	4.75	1	1
22	RU	250	0.02	4.65	4.5	1.1	0.65
23	CU	39	0.02	7.48	6.6	0.28	0
24	CU	40	0.02	6.5	4.5	0.3	0
25	CU	24	0.02	7.25	7.4	0.28	0
26	RU	26	0.02	8.618	8.501	0.15	0.15
27	RU	40	0.02	6.5	3.9	0.9	0.9
28	R	35	0.02	3.8	4.18	1	1

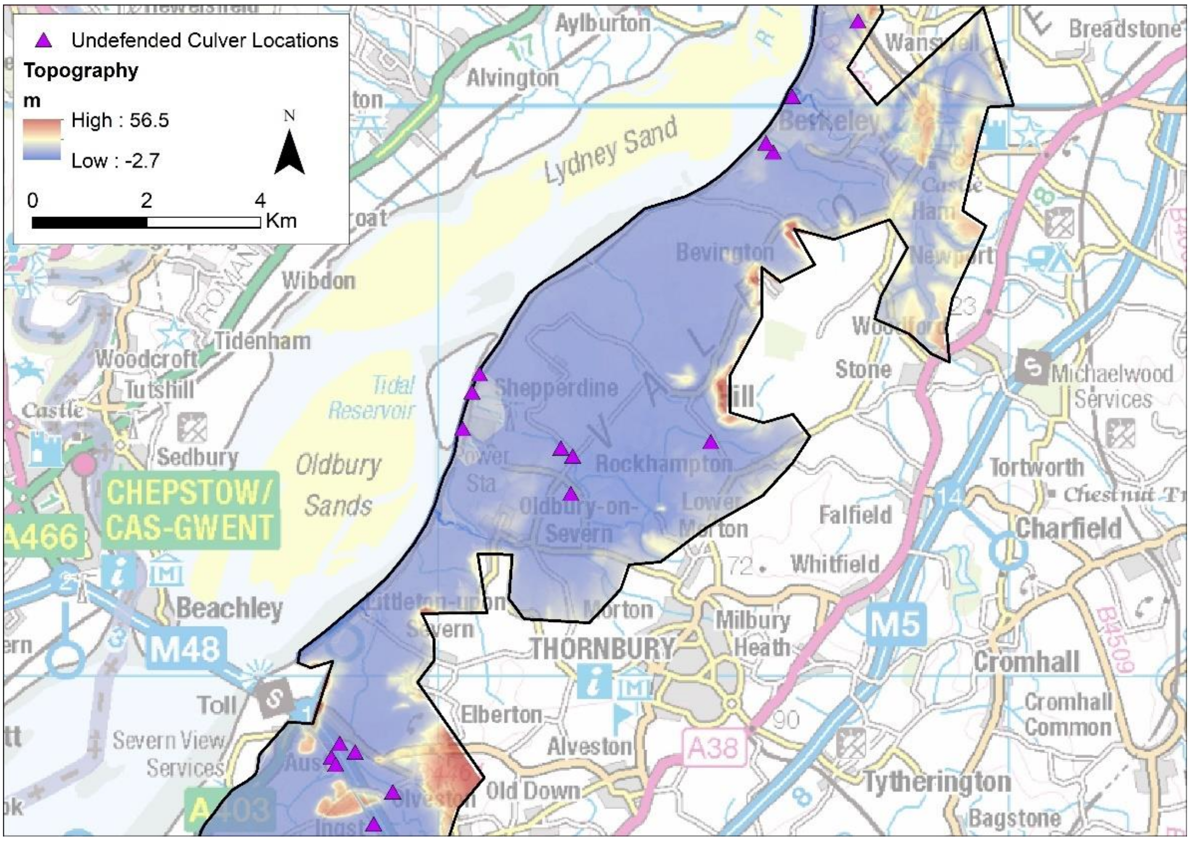

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Figure 6-14: Severn House Farm undefended culvert locations

6.9.11 Severn House Farm model boundaries

A full description of the data used to generate the model boundary conditions can be found in the Woodspring and Severn House model boundary report[13] that sits alongside this reporting. A summary of flood inundation model boundaries is provided below.

A HX offshore water level boundary was used to drive the Severn House farm model as located on Figure 6-15 and the CFB extreme sea level point used in the tide curve generation are identified. The HX boundary was used to define the spatial variation in extreme sea levels along this stretch of coastline. The data used to generate the tide curves for the Severn House Farm model is summarised in Table 6-13.

ST boundaries were used to apply wave overtopping discharges on the landward side of the coastal defence network as shown on Figure 6-16.

[13] Woodspring and Severn House Model Boundary Report. March 2020. JBA Consulting.

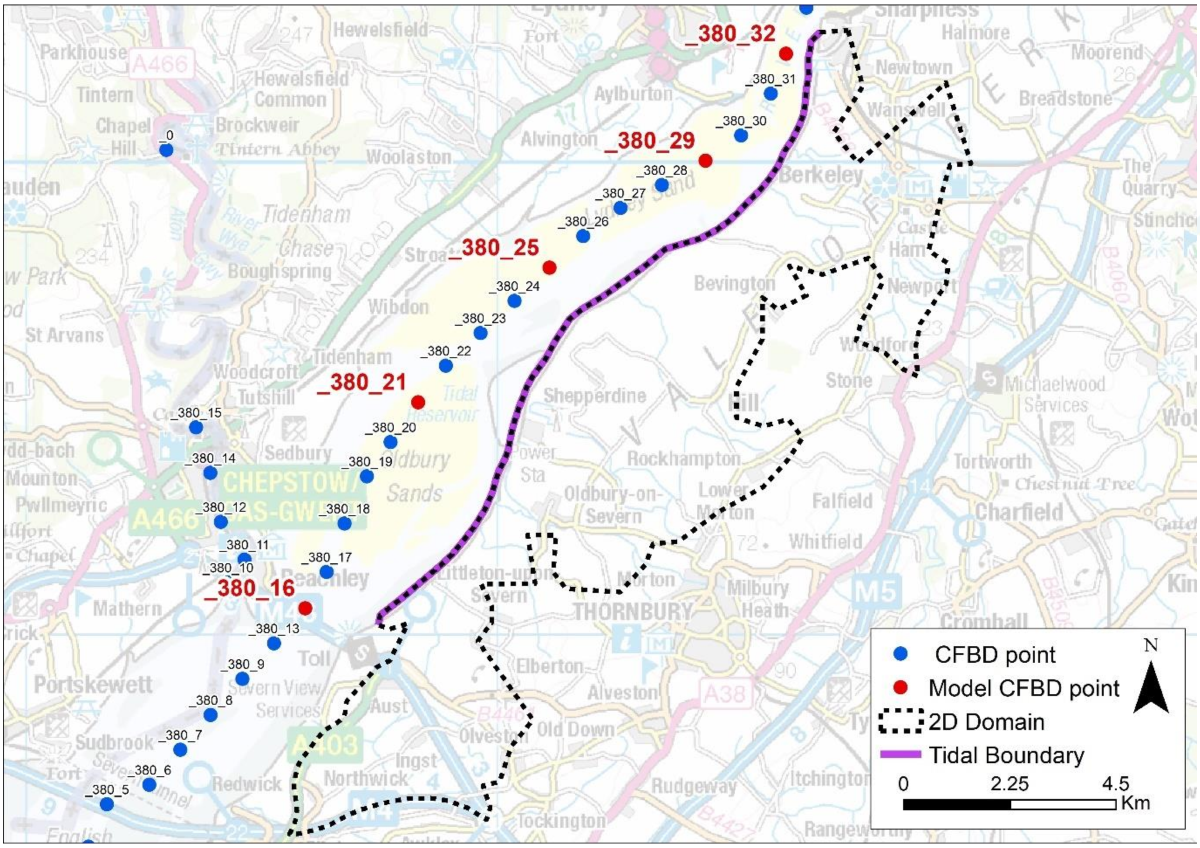

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Figure 6-15: Severn House Farm water level boundary and CFB point location

Table 6-13: CFB points used in modelling

Model	Extreme Sea Level	Astronomical Tide	Surge Shape Location	UKCP09 Grid
Severn House Farm	CFB Chainage Point _380_16	Beachley (Aust)	Avonmouth	23277
	CFB Chainage Point _380_21			23073
	CFB Chainage Point _380_25			23073
	CFB Chainage Point _380_29			23074
	CFB Chainage Point _380_32			22869

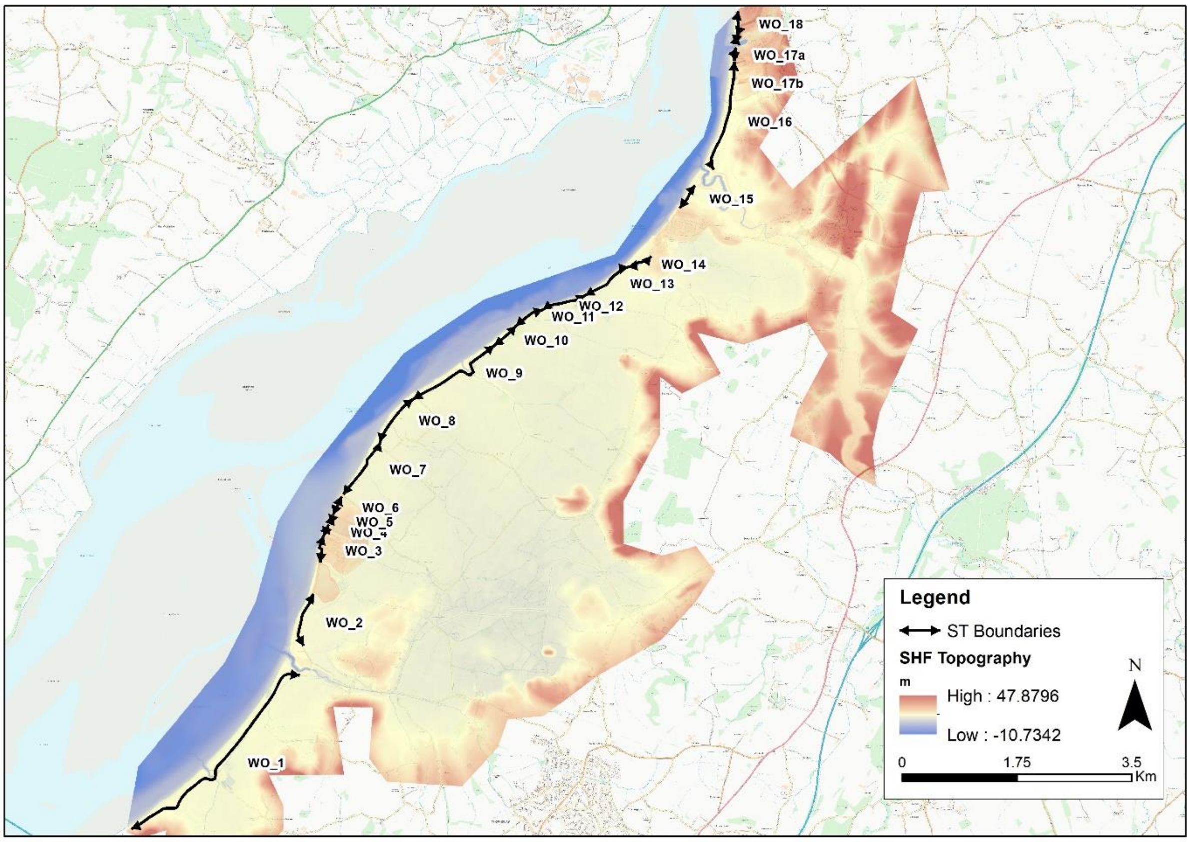

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Figure 6-16: Location of wave overtopping ST boundaries at Severn House Farm

6.9.12 Severn House Farm TUFLOW stability fixes

To address an area of instability within the model a patch of increased topographic roughness was used at Breach House where Cowhill Rhine joins the Bristol Channel. The stability patch is centred around the small channel at Cowhill Rhine. The rough topography of this channel at the 5m model resolution generated model instabilities. The size of the polygon and the roughness value chosen was appropriate to limit the instability yet still allow the progression of flow across the stability patch, albeit with increased resistance to flow. The effects of this stability patch will have a localised impact on model levels and flows. This stability fix was added to the model using the following file:

- *2d_mat_SHF_Stability_001_R.shp*

6.9.13 Undefended Modelling

Within the Severn House Farm model domain, the topography on the landward side of the tidal defence network is below the 20% AEP CFB maximum water level (the smallest modelled AEP event). Formal raised defences were removed from the model topography, these were limited to the raised embankments and structures along the banks of the River Severn. Defacto defences such as the M5 and M48 were left in the model topography. When defences are removed from the model topography, still water flooding is the primary flood risk and leads to widespread inundation of vast areas of floodplain. Wave overtopping discharges were therefore not included in the undefended model scenario, with the exception of a short stretch of coastline at Oldbury Power Station. In this location, once the raised defences were removed, the topography was above the extreme still water levels being modelled. Three undefended overtopping defence sections were therefore schematised (WO_4 to WO_6) as shown on Figure 6-17.

The raised defences were removed from the model topography using the following file:

- *2d_zsh_SHF_defence_removal_001_R.shp*

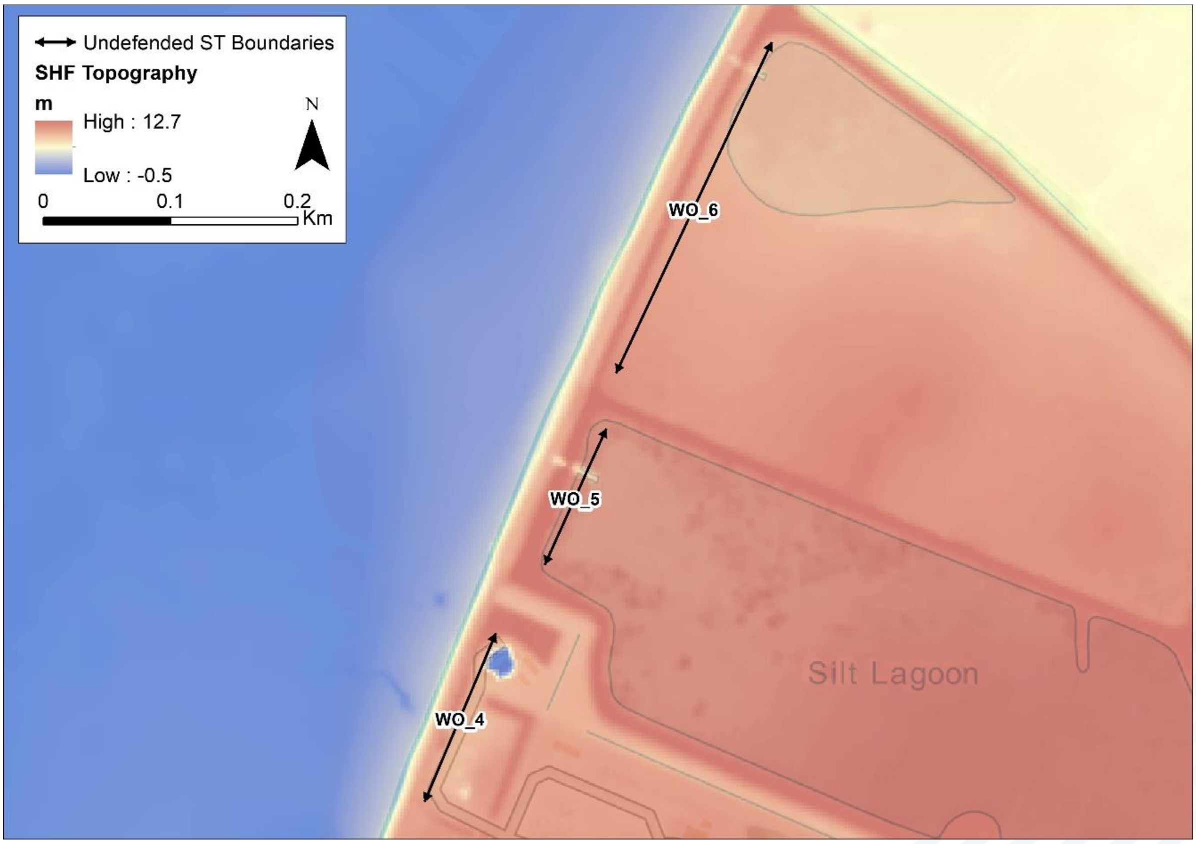

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Figure 6-17: Location of undefended wave overtopping ST boundaries in Severn House Farm model

6.10 Calibration and validation of the TUFLOW models

The TUFLOW model was calibrated against the January 4th 2014 and February 8th 2016 flood events. Both storms caused flooding at multiple locations along the Wessex coastline. The model was run using wave and wind data extracted from WaveWatch III point 573 as well as historic water level and surge data from the Hinkley and Avonmouth tide gauges. The data was fed into the JBA ForeCoast module that relates offshore inputs to nearshore model results. The offshore data used was the WaveWatch III wave data and gauged water level data. The ForeCoast module then looked up the nearest offshore condition from a modelled ensemble dataset of 4,704 model simulations and related it back to the nearshore modelled overtopping result. The system was simulated for the Hindcast dataset from 1992-2016 resulting in a continuous 24-year record. There was some flood risk associated with wave overtopping

The results could then be compared against known flood history where available, in the form of photographs and anecdotal information on the flood extents and depths, in a calibration process to validate our model setup. At the locations modelled, there was limited flood history to compare the model results against. Simulating the event and assessing the outputs is also a way of validating the model; if the result showed significant flood risk when there is no reported flood history then the faith in the model system would be low.

6.10.1 Calibration Results: Woodspring Bay

The modelled January 14th 2014 event showed limited flood risk across the model domain (Figure 6-18). There was some flood risk associated with wave overtopping at Marine Lake, Sand Bay and Wick St Lawrence as shown on Figure 6-19, Figure 6-20 and Figure 6-21 respectively. Flood depths at these locations are largely related to spray overtopping with depths mostly below 0.05m with some areas reaching 0.20m.

The modelled February 8th 2016 event shows similar flood risk to the 2014 event, with limited flood risk across the model domain (Figure 6-22). There was some flood risk associated with wave overtopping at Marine Lake, and Wick St Lawrence as shown on Figure 6-23 and Figure 6-24.

Flood depths were smaller in some locations when compared with the January 14th 2014 event. Within Sand Bay, for the 2016 event there was no overtopping modelled flood risk (Figure 6-25).

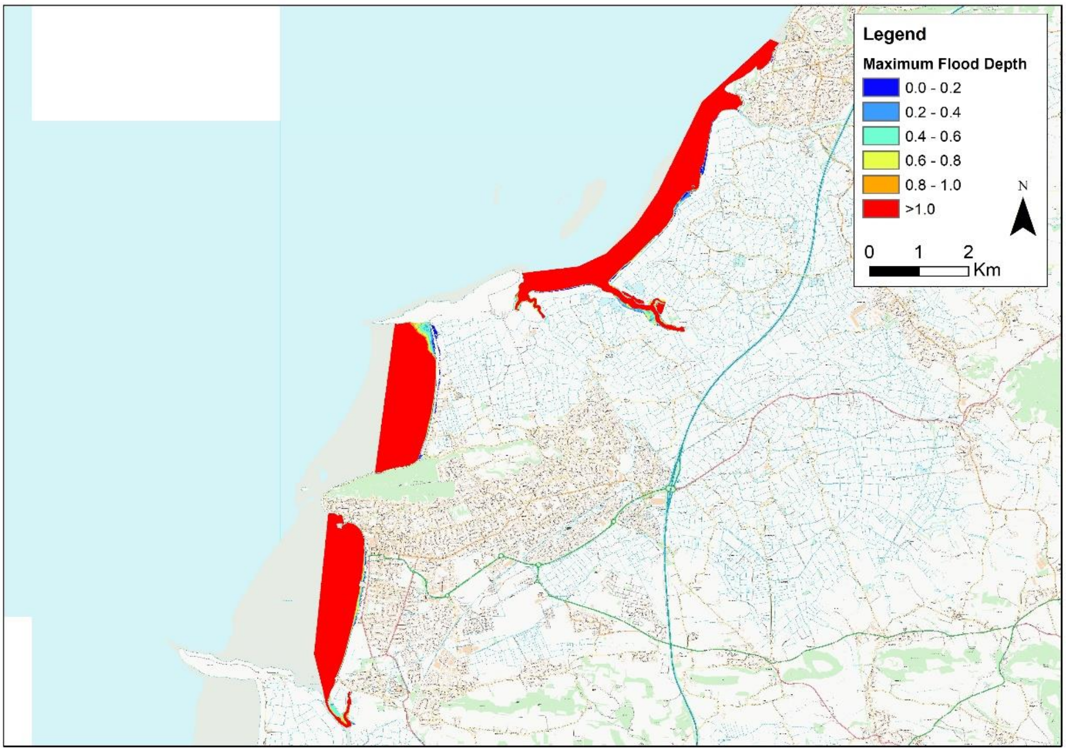

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Figure 6-18: Woodspring Bay 2014 event entire model domain flood depth grid

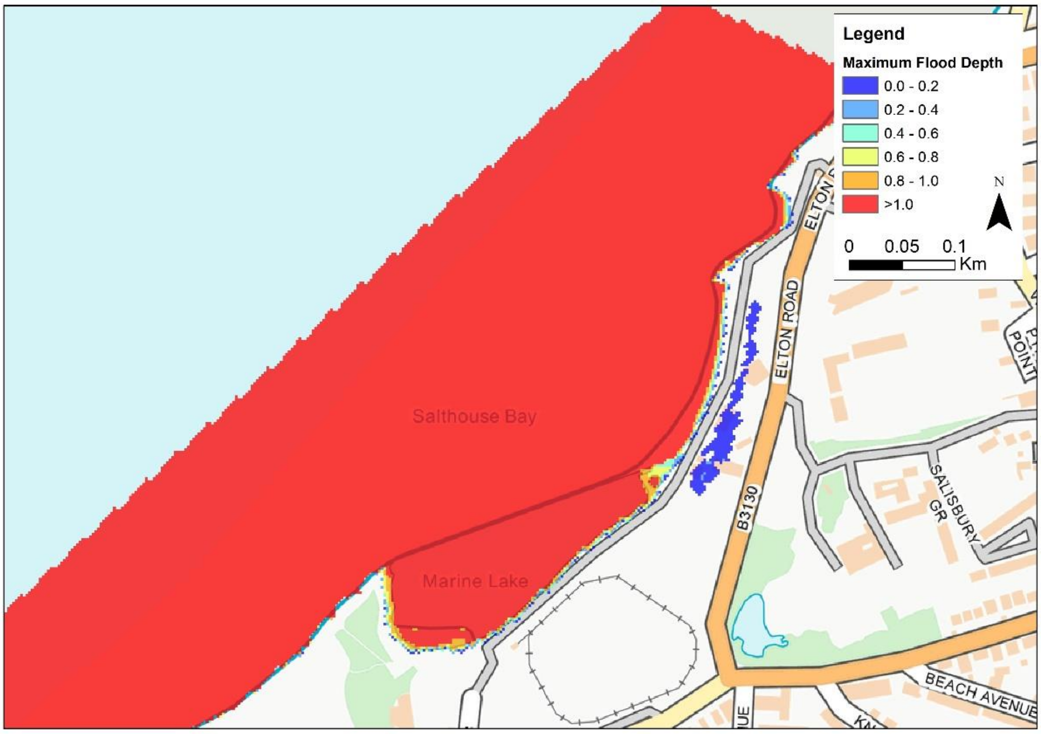

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Figure 6-19: Marine Lake 2014 event flood depth grid

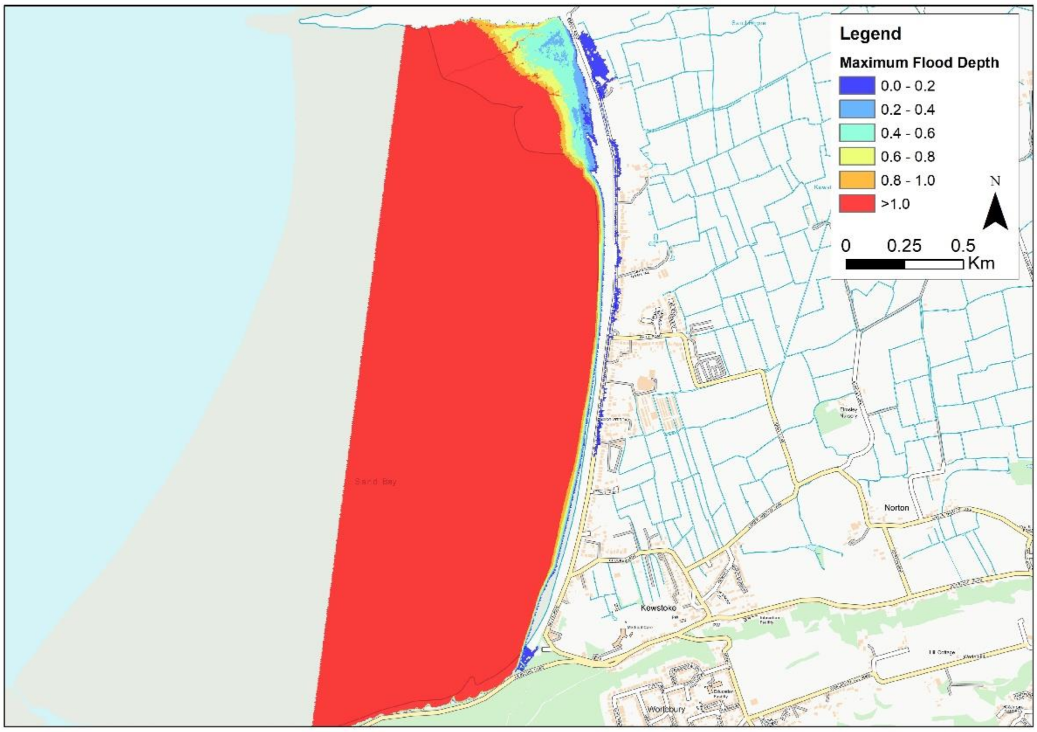

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Figure 6-20: Sand Bay 2014 event flood depth grid

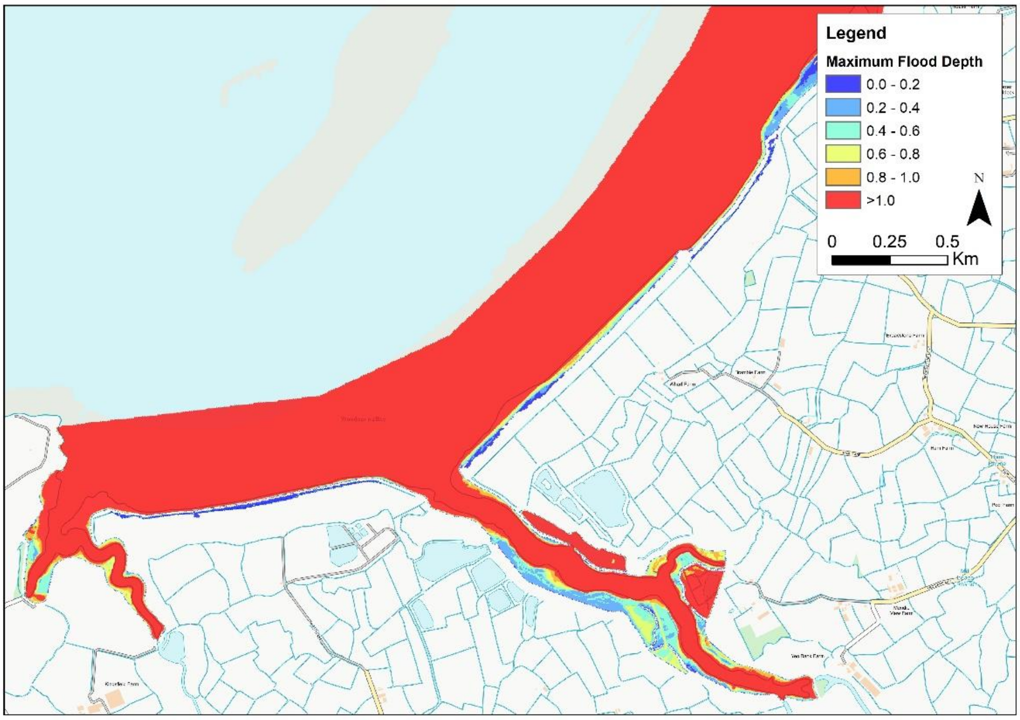

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Figure 6-21: Woodspring Bay 2014 event flood depth grid

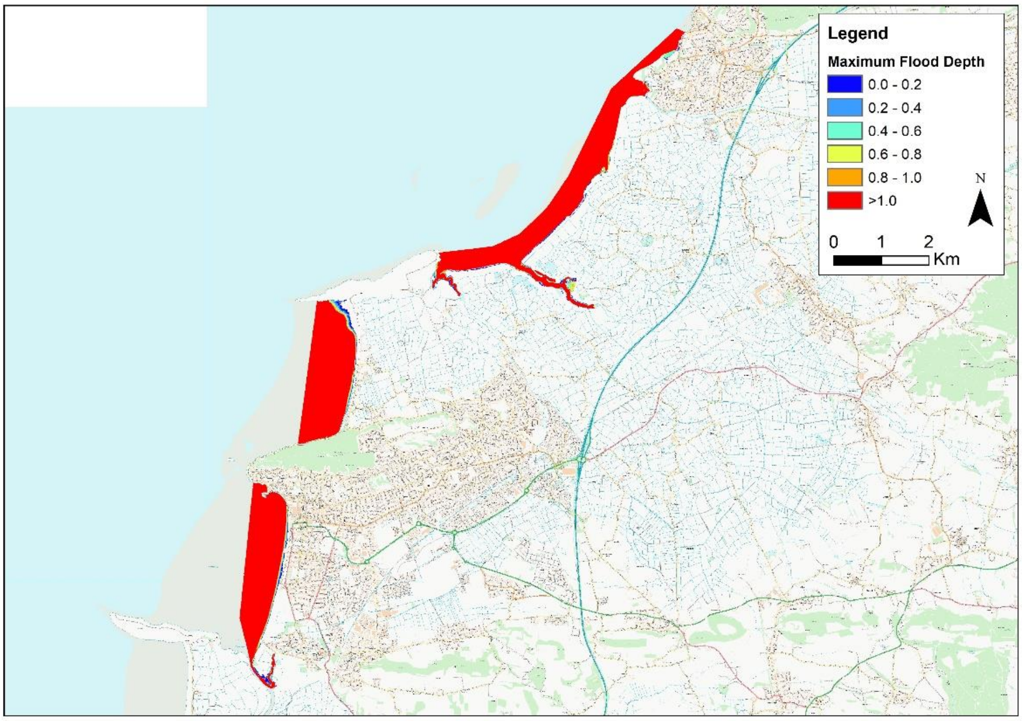

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Figure 6-22: Woodspring Bay 2016 event entire model domain flood depth grid

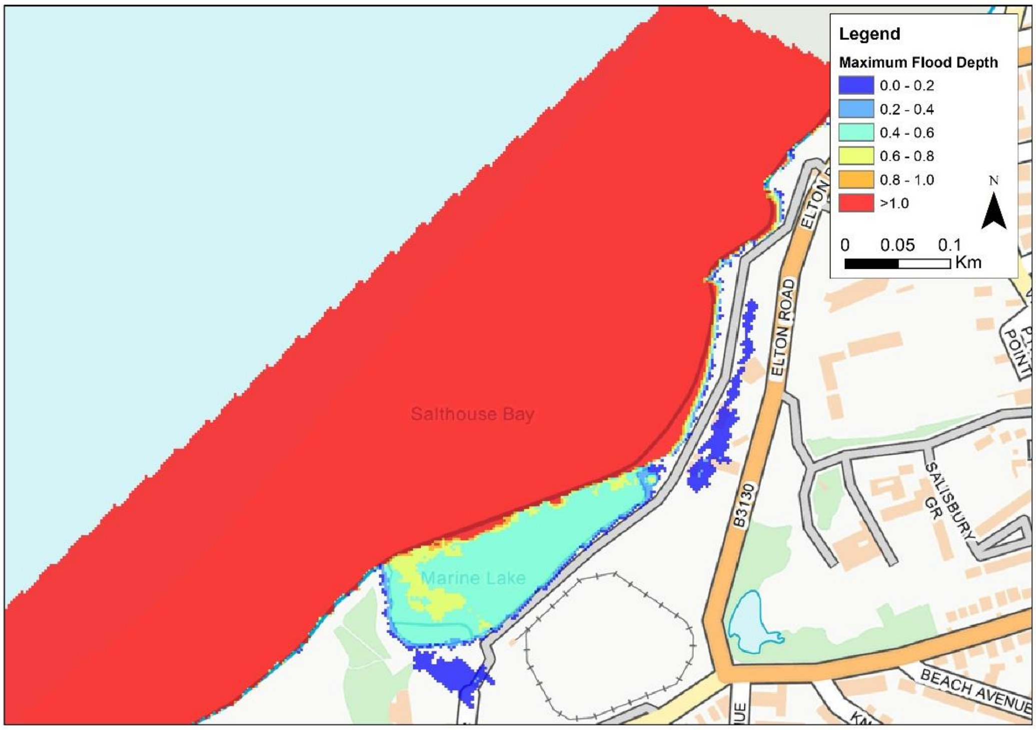

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Figure 6-23: Marine Lake 2016 event flood depth grid

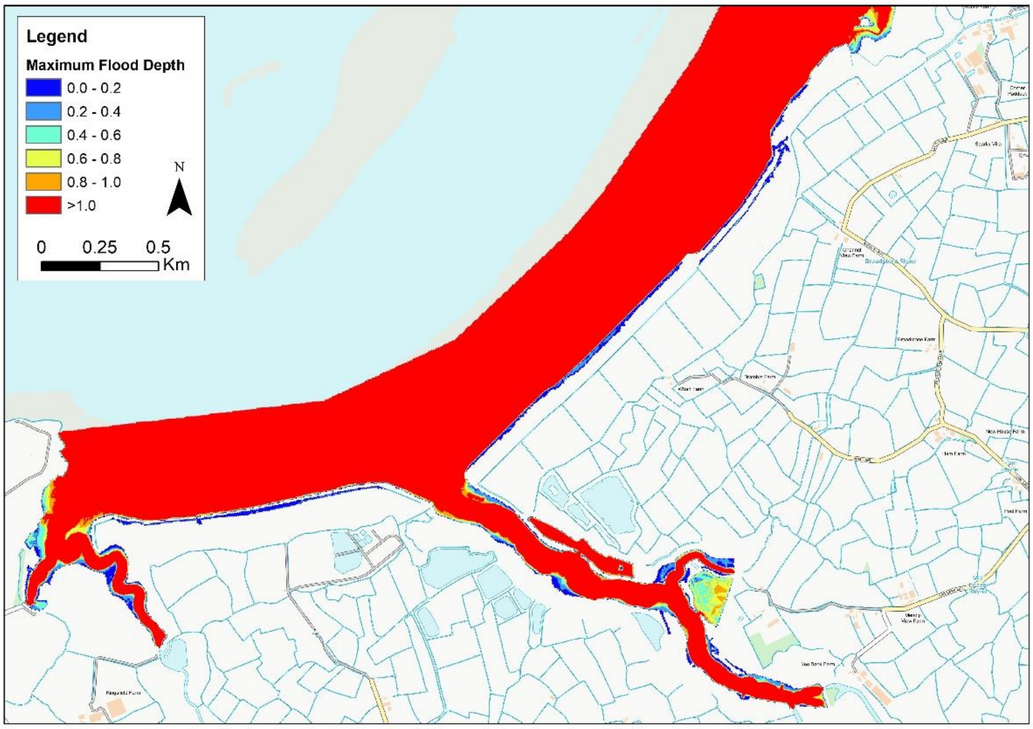

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Figure 6-24: Woodspring Bay 2016 event flood depth grid

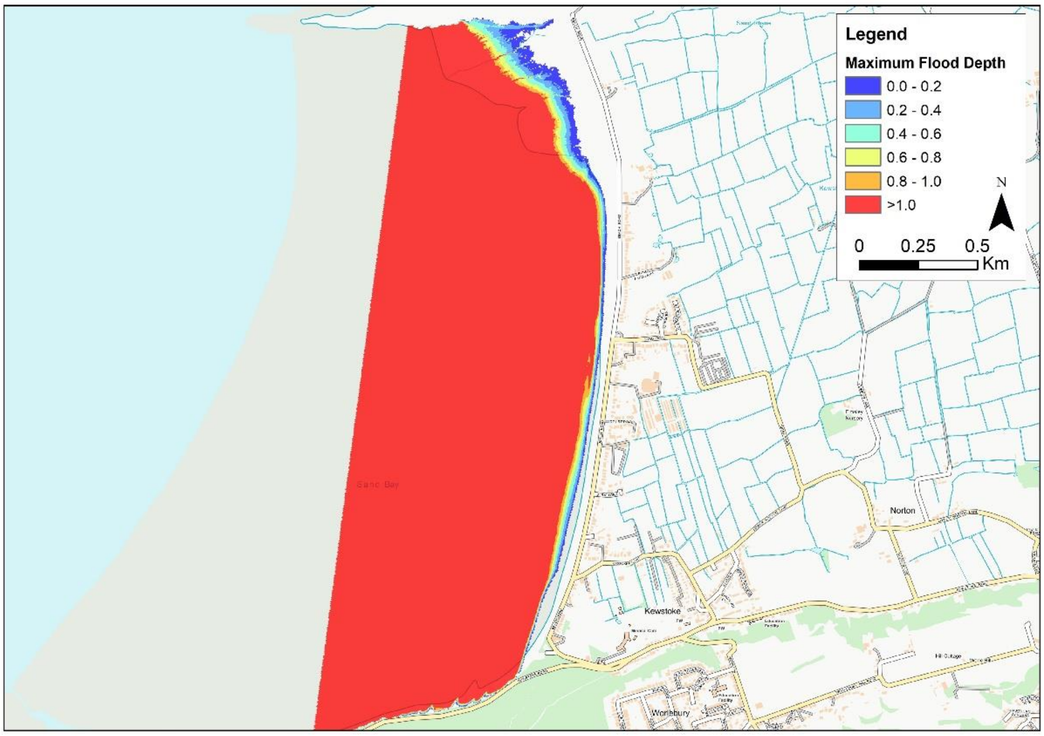

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Figure 6-25: Sand Bay 2016 event flood depth grid

6.10.2 Calibration Results: Severn House Farm

The modelled January 14th 2014 event showed limited flood risk across the model domain (Figure 6-26). There was some flood risk associated with wave overtopping at Severn House Farm and at Oldbury Power Station as shown on Figure 6-27 and Figure 6-28 respectively. Flood depths at these locations are related to spray overtopping with depths being mostly below 0.05m. The raised embankments surrounding Oldbury Power Station silt lagoons are picked up in the model grid, however a lack of defence data here is the likely reason for the early onset of flooding, and therefore more accurate representation of the defences at Oldbury Power Station could be considered in future modelling. Some flood depths reach 0.10m at Severn House Farm.

The modelled February 8th 2016 event shows similar flood risk to the 2014 event, with limited flood risk across the model domain (Figure 6-29). There was some flood risk associated with wave overtopping in the same locations as the 2014 event at Severn House Farm and at Oldbury Power Station as shown on Figure 6-30 and Figure 6-31respectively. The raised embankments surrounding Oldbury Power Station silt lagoons are picked up in the model grid, however a lack of defence data here is the likely reason for the early onset of flooding, and therefore more accurate representation of the defences at Oldbury Power Station could be considered in future modelling. At Severn House Farm, there is only wave overtopping risk north of Severn Lane during the 2016 event. For the 2014 event there is also risk to the south of Severn Lane.

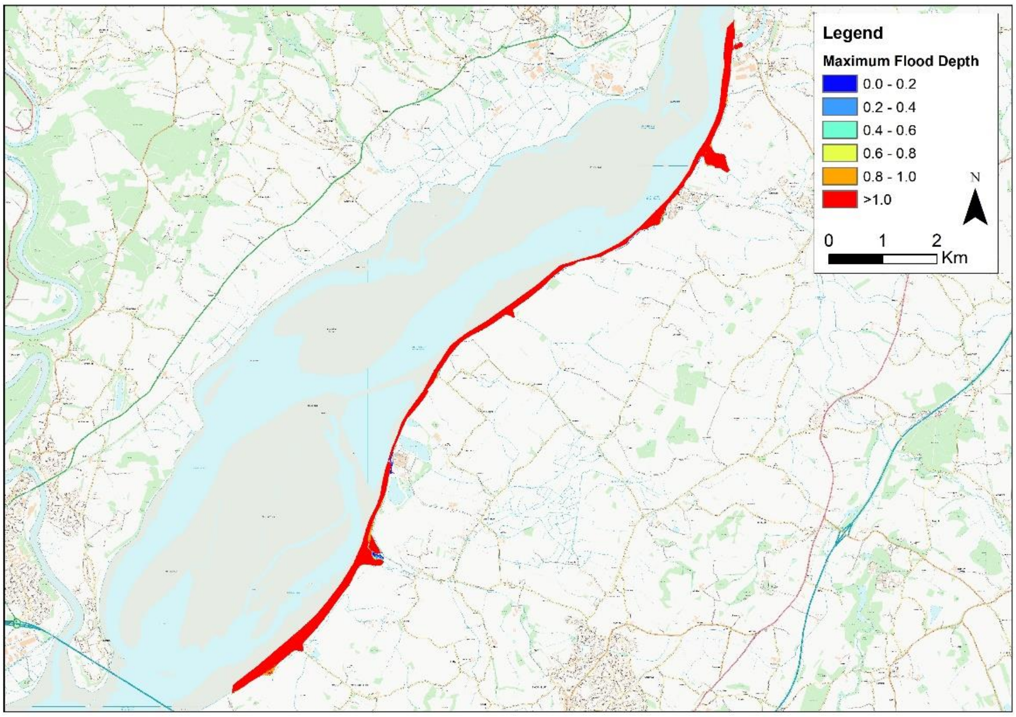

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Figure 6-26: Severn House Farm 2014 event entire model domain flood depth grid

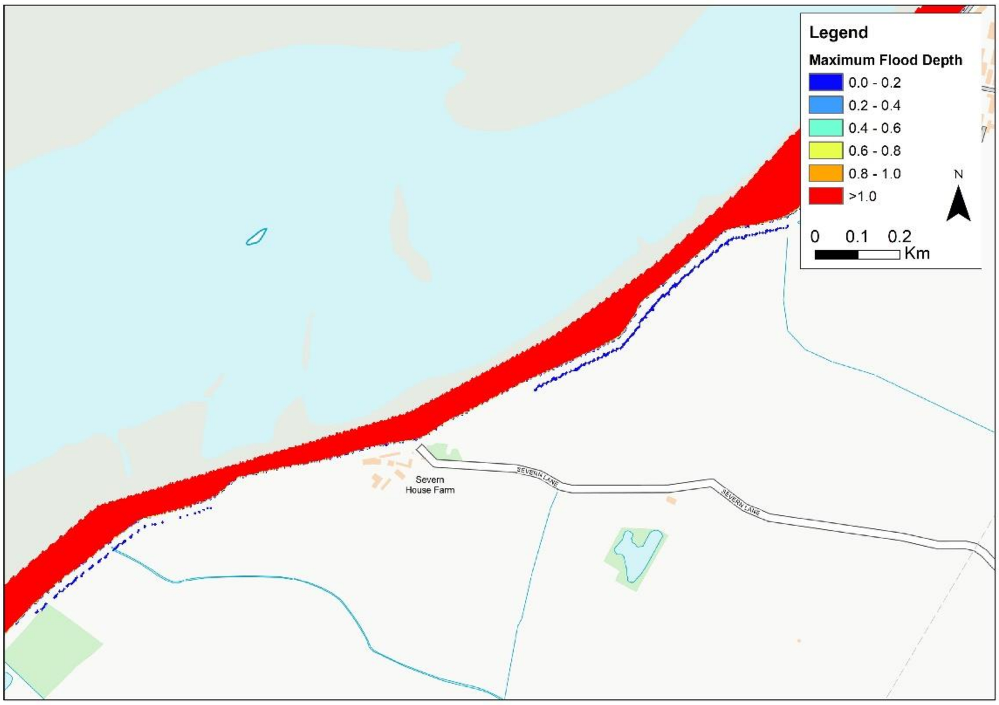

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Figure 6-27: Severn House Farm 2014 event depth grid

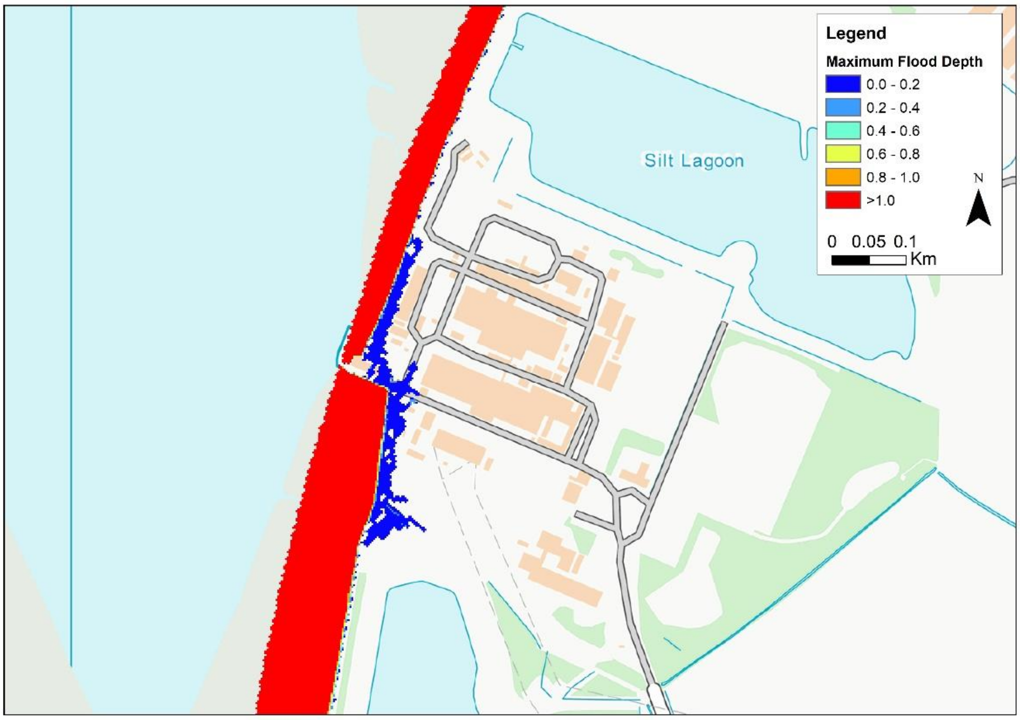

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Figure 6-28: Oldbury Power Station 2014 event depth grid

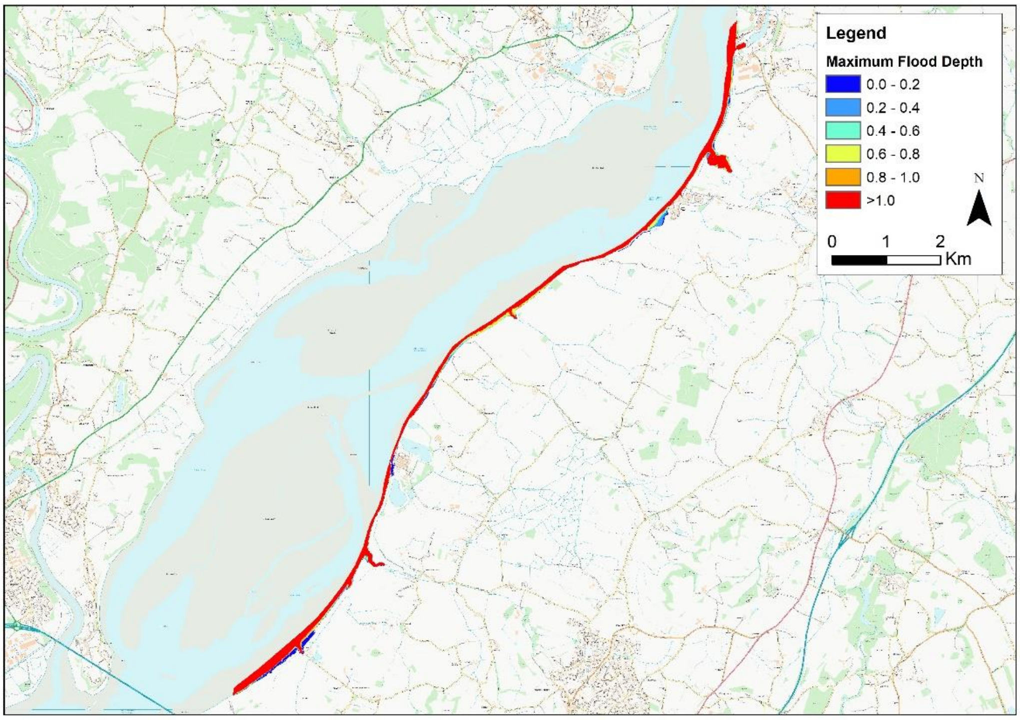

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Figure 6-29: Severn House Farm 2016 event entire model domain flood depth grid

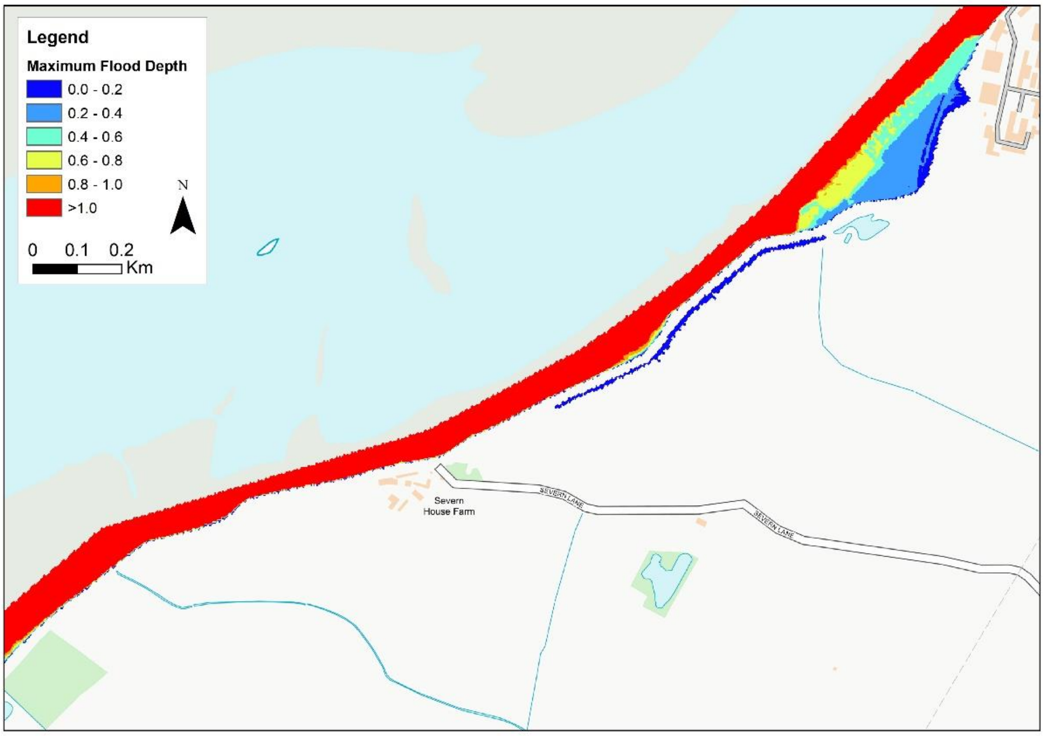

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Figure 6-30: Severn House Farm 2016 event depth grid

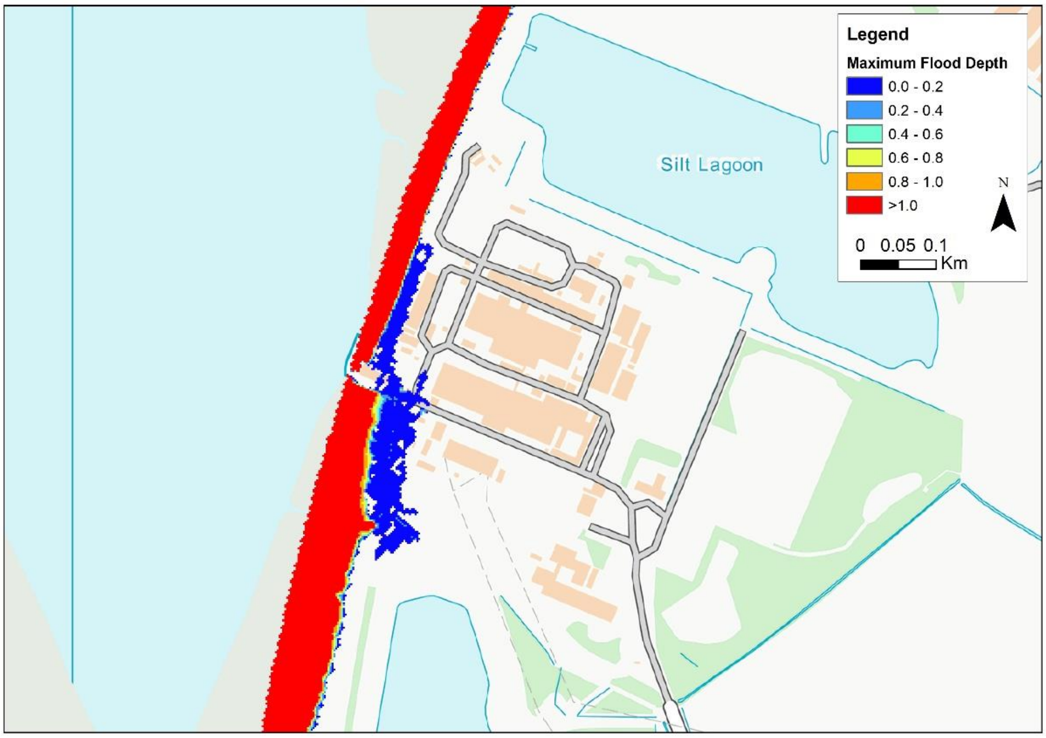

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Figure 6-31: Oldbury Power Station 2016 event depth grid

7 Defence removal simulations

At Woodspring Bay there are two areas where the coastal defence network includes both a primary and a secondary defence line. At Wick St Lawrence, the secondary, more landward defence, is the larger main line defence with the primary defence along the coastal frontage being smaller, but acting to break waves before they reach the secondary defence (Figure 7-1). At Kingston Seymour the primary defence is the biggest and most robust, with the rear defences consisting of small earth embankments designed to contain overtopped floodwaters (Figure 7-2). Two defence removal scenarios were modelled to help provide evidence of the value of the defences and the importance of their continued maintenance. The defence removal scenarios modelled are as follows:

- *Wick St Lawrence: The primary defence was removed (flattened) in the wave transformation and flood inundation model topography. The secondary defence was maintained in its current form.*
- *Kingston Seymour: The primary defence was maintained in its current form while the secondary defence was removed from the flood inundation model.*

The defences were removed within the flood inundation model using a Triangular Irregular Network (TIN) to flatten the defence down to base ground level; the removal extents are shown on Figure 7-1 and Figure 7-2. The modifications made in the wave model mesh for the above scenarios are detailed in Chapter 4.1.1.

The model scenarios and events simulated are detailed in Figure 7-1.

Table 7-1: Defence removal scenarios simulated

Location	Defence Removed	AEP % simulated
Wick St Lawrence	Primary	2.0, 1.33, 1.0, 0.5, 0.5 UKCP09 2068
Kingston Seymour	Secondary	2.0, 1.33, 1.0, 0.5, 0.5 UKCP09 2068

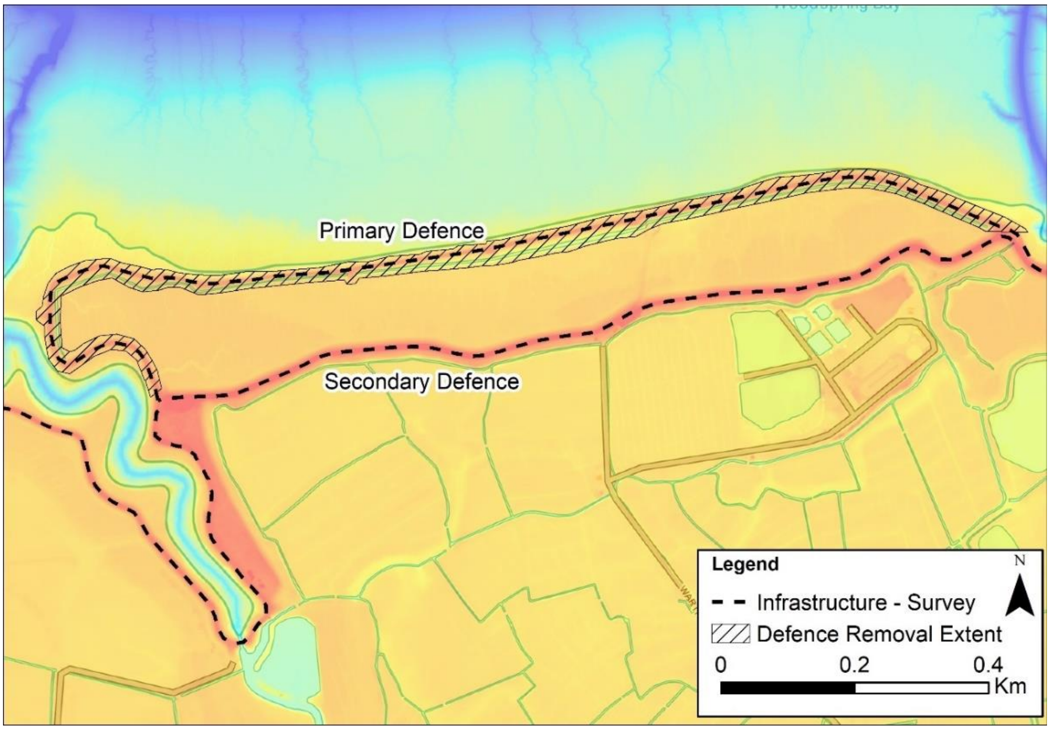

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Figure 7-1: Wick St Lawrence primary and secondary defence

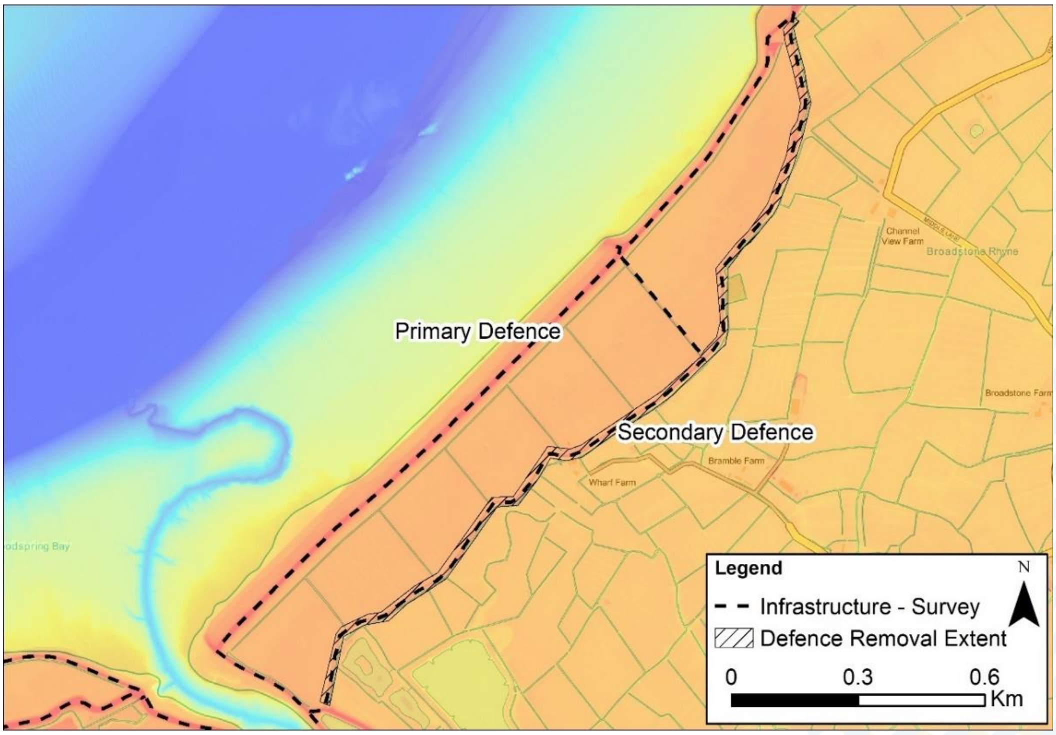

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Figure 7-2: Kingston Seymour primary and secondary defence

8 Marsh Loss Analysis

To further understanding of the impact of marsh erosion where there is currently a 'Hold The Line' policy at Woodspring Bay, a marsh loss scenario was simulated using the coastal model suite.

The defended and undefended design simulations were modelled using a 2D SWAN wave transformation model that transposed offshore wave conditions across the marsh to the foot of the coastal defence network. To more easily model the lowering of the marsh, to represent potential future erosion, SWAN 1D was used. By running SWAN in 1D mode using cross shore profiles of the marsh it was much easier to represent a lowering of the marsh when compared to altering the topography within the 2D flexible mesh.

A test was undertaken to compare the results of running the 2D and 1D SWAN models for an example site driven with the same boundary conditions. The results were comparable and therefore the use of SWAN 1D was deemed appropriate for use in the marsh loss modelling and comparison to the 2D SWAN baseline results.

To model the marsh loss scenario, the following methodology was adopted:

- *Wave results were extracted from 2D model nodes at the foreshore and used as boundary conditions for the SWAN 1D simulations. SWAN 2D nodes used for each of the 1D SWAN profiles are detailed in Table 8-1 and the SWAN 1D profile locations shown on Figure 8-1.*
- *The SWAN 2D topography (existing situation) was extracted across the marsh foreshore. The profile was lowered by 0.5m to represent future erosion of the marsh.*
- *A SWAN 1D model for was setup for 11 cross shore profiles (ID 51 through 62) to represent the marsh loss topography (lowering of 0.5m of marsh out to a location on the foreshore where the beach slope became more gentle). An example of the baseline marsh and marsh loss profile for ID 51 and 60 are shown on Figure 8-2 Figure 8-3 respectively.*
- *The EurOtop ANN schematisations were modified to represent the eroded beach state (lowering the toe level by 0.5m.*
- *The SWAN 1D model was simulated using the design model worst-case wave and water level joint probability conditions for five AEP events (10%, 5%, 2%, 0.5% and 0.1% AEP).*

The AEP conditions simulated for IDs 51 through 62 and resultant overtopping rate is detailed in Appendix C under Woodspring Bay Marsh Loss.

Table 8-1: 2D SWAN nodes used as 1D SWAN boundary conditions

Location ID	SWAN 2D node
51	117055
52	129108
53	129108
54	127185
55	127236
56	132947
57	130963
58	130939
59	134830
60	136810
61	128458
62	119120

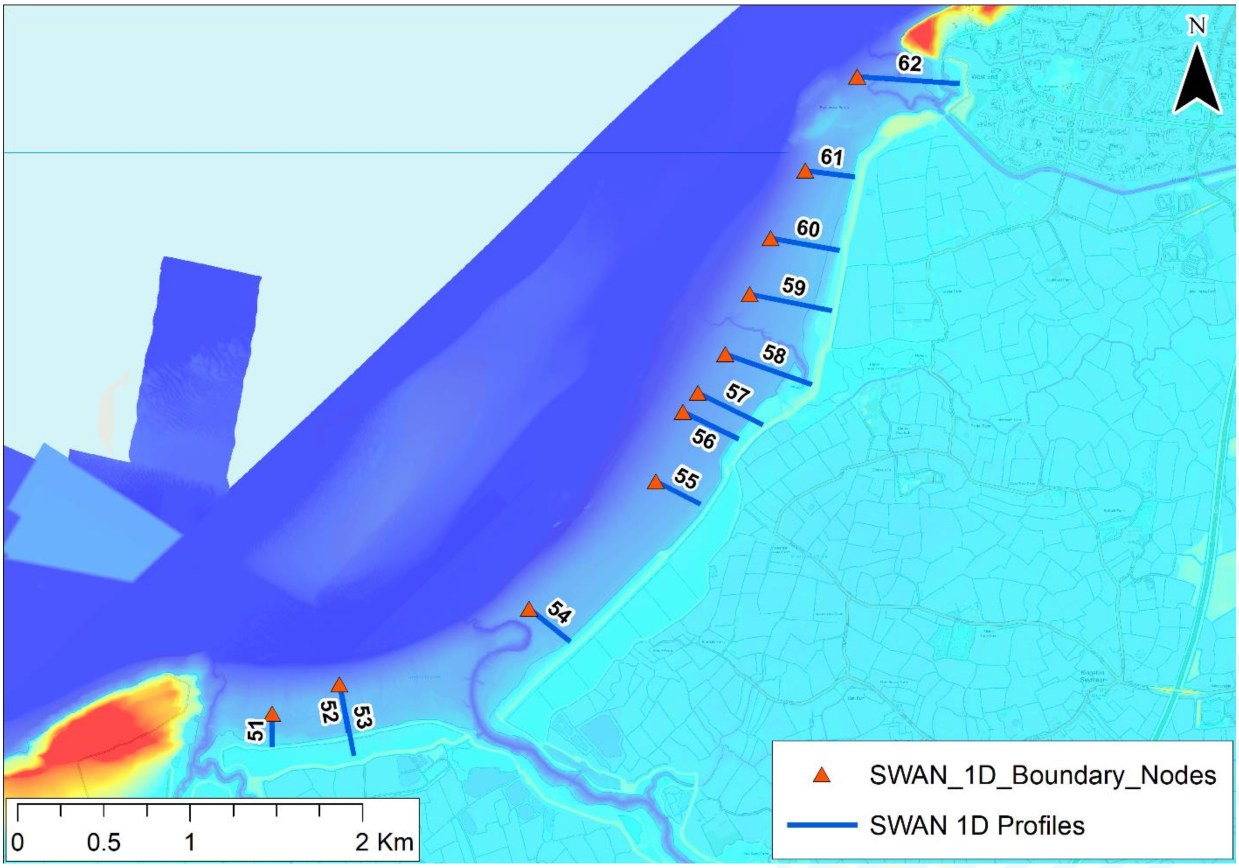

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Figure 8-1: Location of marsh loss SWAN 1D profiles

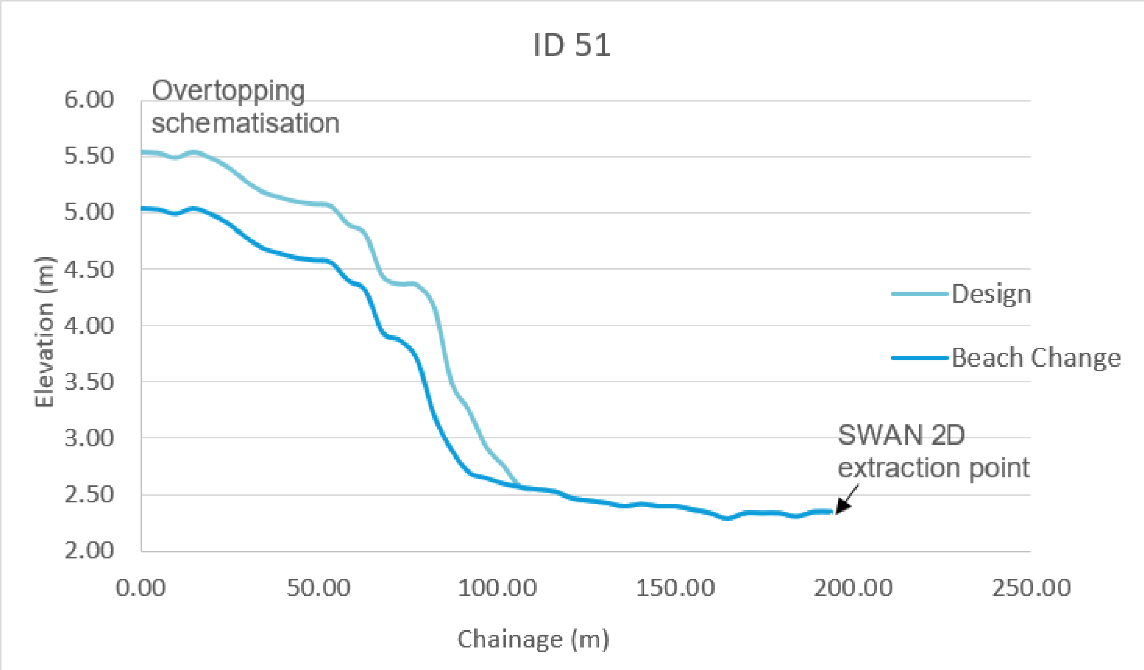

Figure 8-2: Baseline design marsh profile against marsh loss profile ID51

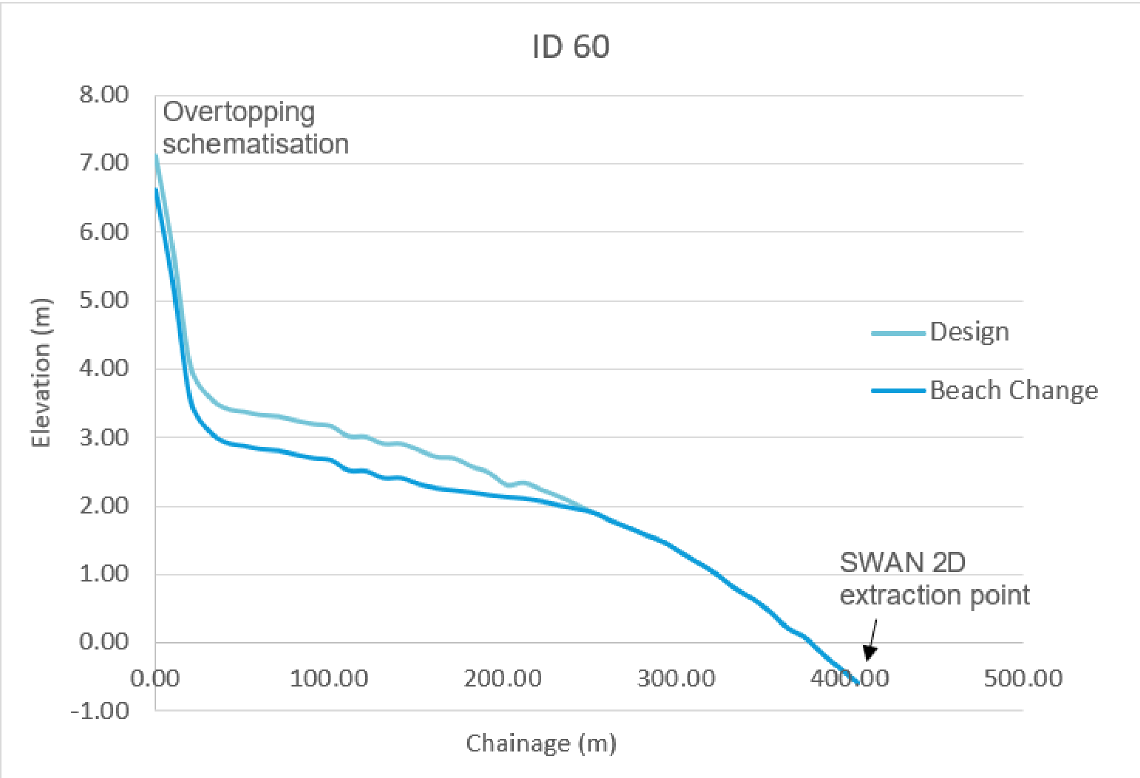

Figure 8-3: Baseline design marsh profile against marsh loss profile ID60

9 Defence breach modelling

A significant proportion of the study area in the Severn House Farm model domain is potentially vulnerable to defence breach flooding given the low-lying topography and the reliance on flood defences. A series of defence breach scenarios were set up and simulated for a host of events to assess the impact of a defence breach.

9.1 Breach methodology

Four breach locations were determined by the Environment Agency (Table 9-1). The breach locations are shown graphically on Figure 9-1.

Breach modelling was undertaken using the 2017 Breach of Defences Guidance[14] and each key component of the breach modelling is discussed in the remainder of this chapter. The key breach setup information is detailed in Table 9-2 and the events simulated for each breach in Table 9-3.

Breaches in the Severn House Farm model were considered in isolation from one another. Defence breaches were run separately so that the maximum volume of water can pass through the single breach and not be funnelled elsewhere by another breach. However, in reality, if one section of a defence is likely to fail then adjoining sections would likely fail. This was not considered as part of this modelling.

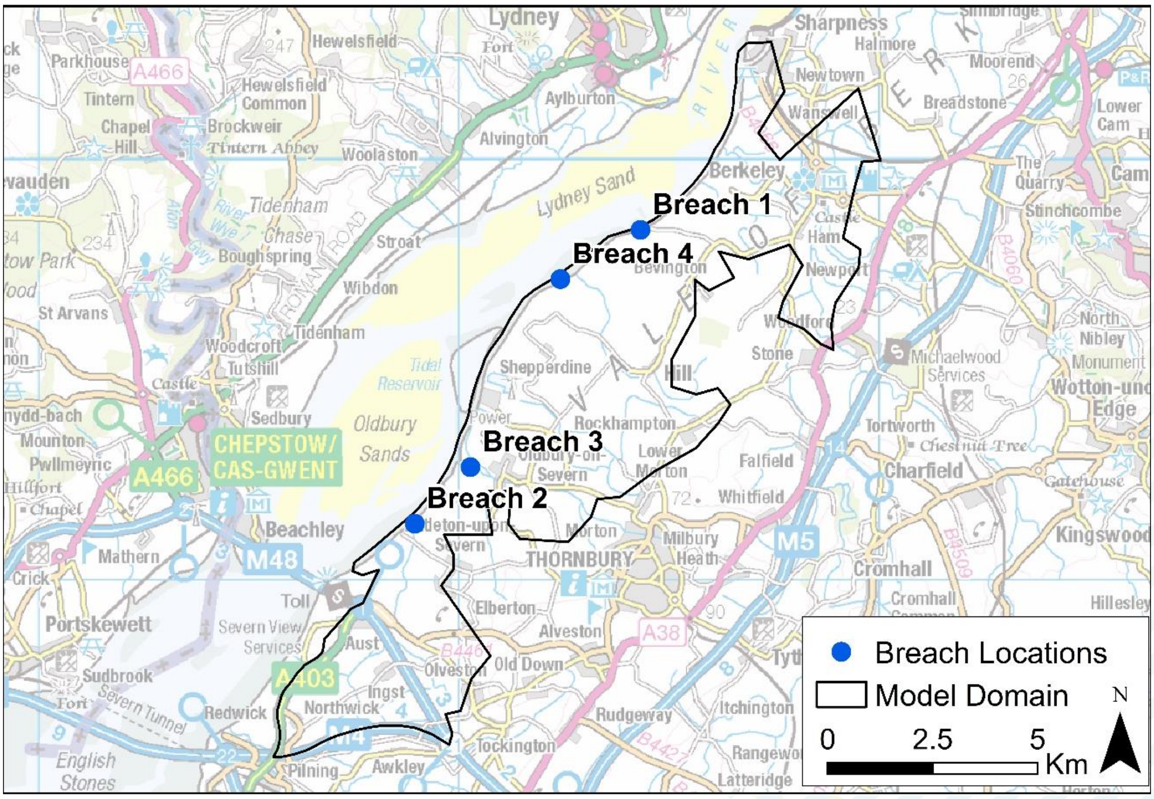

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Figure 9-1: Breach locations

14 Breach of Defences Guidance. Modelling and Forecasting Technical Guidance Note. Environment Agency. 2017

Table 9-1: Breach locations

Breach model number	Breach name	NGR
1	Severn House Farm	ST6430098321
2	Whale Wharf	ST5895391329
3	Oldbury Outfall	ST6028892671
4	Hill Pill	ST6237497197

Table 9-2: Breach scenarios

Breach model number	Breach name	Defence type	Breach Width (m)	Trigger	Restore Interval (hrs)	Breach Information
1	Severn House Farm	Concrete revetment, clay/earth embankment core	50	Water level reaches half defence height	30	Breach of the concrete revetment embankment. Note; this was modelled using a 50m earth bank defence type to be conservative as discussed in chapter 9.1.3.
2	Whale Wharf	Earth bank	50	Water level reaches half defence height	30	Breach of the grassed earth embankment
3	Oldbury Outfall	Tidal Gate	Open Gate	Low tide preceding the peak modelled tidal cycle	12.25hrs (1 tidal cycle) - Low tide following the peak modelled tidal cycle	Oldbury outfall was modified to use operational controls based on model simulation time. This enabled failure of the sluice gate on the trough prior to the peak tide and emergency restoration of the sluice on the following low tide.
4	Hill Pill	Earth bank	50	Water level reaches half defence height	30	Breach of the grassed earth embankment

Table 9-3: Breach events simulations

Breach number	Breach name	Event (% AEP) model simulations		
		1.33%	0.5%	0.5% NPPF 2068
1	Severn House Farm	✓	✓	✓
2	Whale Wharf	✓	✓	✓
3	Oldbury Outfall	✓	✓	✓
4	Hill Pill	✓	✓	✓

9.1.1 Breach trigger

In a river or 'non-wave' tidal scenario the trigger for breach failure is when the water level reaches three-quarters of the defence crest height. If there is wave loading on a structure along the open coast, the breach trigger is either when water level reaches half of the defence crest height, or when wave overtopping starts. The chosen breach locations were assessed and for all locations a wave loading was assumed. The water level at all breach locations would reach half the defence height before overtopping initiates and therefore this water level trigger was used for all breach locations bar breach_3. The trigger levels were determined using the breach toe level and the defence crest level (refer to 9.1.4) and detailed in Table 9-4. Water level loadings on the coastal side of the defence structure were taken from the 2D model domain using a point trigger location.

At breach_3 a tidal gate failure was modelled, and the trigger was based on the gates failing on low tide preceding the peak level with emergency closure effected during the following low tide. An operational control file was used to open and close the gate based on the boundary conditions timing of low tide.

Table 9-4: Trigger level

Breach model number	Trigger level	Crest level	Toe level
1	8.28	10.05	6.50
2	8.41	9.27	7.55
3	N/A	N/A	N/A
4	7.92	9.33	6.50

9.1.2 Breach width

Breach widths were based on Environment Agency guidance as detailed in Figure 9-2. Breach 1, 2 and 4 were based on an earth bank defence type within an estuary/tidal river. These breaches were included as 50m wide defence failures in the 2D model domain. Note that breach_1 at Severn House Farm is actually a clay/earth embankment with reinforced concrete revetment. The current condition of the revetment blocks is not good and during a tidal event it is not known how effective the concrete revetment would be. Therefore, rather than adopting the reinforced concrete breach width of 20m, a more conservative approach was adopted using a 50m breach width as per an earth bank.

At breach_3 the two tidal gates of Oldbury Outfall represented in the 1D structure units, were opened to represent gate failure.

9.1.3 Time and duration of breach

Breach failure and emergency closures times were based on Environment Agency guidance as detailed in Figure 9-2, whereby the 'time to close' refers to the hours following the breach.

Breach failure time, for defences other than tidal gates, was set based on the time the water level on the coastal side of the defence structure reached the trigger level (refer to chapter 9.1.1). For tidal gates the failure time was based on the time of low tide preceding the peak modelled tidal cycle.

The breach failure duration was set to 0.1 hours for all breaches. In reality, a breach in an earth embankment would open over the course of several hours, while a concrete structure failure would be almost instantaneous with the collapse of the structure. In many previous modelling studies, breach failures were simplified to an instantaneous breach to avoid any instabilities in the model caused by a gradual lowering. Therefore, the choice of a failure over 0.1 hours is a suitable compromise.

Breach closure times differs based on defence type and locality to urban or rural land use. For defences other than tidal gates, the earth bank defence type was used (note that earth bank defence type was used at Severn House Farm to be conservative as discussed in chapter 9.1.1). Breach locations were away from any large population centres and it was decided that breach closure time for all breach locations would be based on rural. Tidal gates were closed on the low tide following the modelled peak tide as stipulated by the guidance.

9.1.4 Breach toe level and crest level

The breach was lowered to the lowest level of the ground behind the raised defence that was being breached. This was achieved using four points at the four corner elevations of the breach polygon extent in the modelling. The intervening points inside the breach extent were then interpolated based on these selected ground levels. High-resolution LIDAR was used to inform the breach toe level as survey was not available.

The crest level across the 50m breach width was determined by taking an average of the surveyed spot crest levels in the breach vicinity.

Both the defence crest level and the breach toe level were used to determine the trigger level based on half the raised defence height.

Source	Defence Type	Breach Width (m)	Time to close – urban (hrs)	Time to close – rural (hrs)
Estuary/Tidal River	Earth Bank	50	30	30
	Reinforced Concrete	20	18	18
Open Coast	Earth Bank	200	44	56
	Earth Bank with facing	100	44	56
	Dunes	100	44	56
	Shingle Bank	100	30	30
	Reinforced Concrete	50	18	30
River	Earth Bank	40	30	56
	Reinforced Concrete	20	18	18
Tidal/Coastal	Tidal Gates	Gate width	Gates fail on low tide preceding the peak level with emergency closure effected during the following low tide	

Figure 9-2: Recommended breach parameters, Environment Agency guidance

10 Model limitations

The approaches taken in this study incorporate the most advanced, appropriate methods currently available for coastal modelling. The data used in the modelling was the most recent available at the time.

Whilst all due care was taken, the results should be viewed with a margin of caution given the inherent uncertainty in coastal modelling. The following summarises key limitations and recommendations:

- All wave overtopping calculations assume a static beach profile, representing a snapshot in time. Wave conditions are assumed to remain constant throughout the progression of the tidal curve, changes in overtopping rates are therefore a result of the changing water level conditions.
- The seabed is subject to constant change and the bathymetry used in the SWAN model is representative of normal seabed conditions.
- The crest level survey used in the modelling dates from 2007. It is recommended that new topographic survey is commissioned for any future modelling.
- Nearshore local winds are not represented in the wave transformation model and overtopping wave momentum is not included in the flood inundation model. In some instances, this can lead to an underestimation of flood risk due to wave overtopping where momentum carries floodwater across sloping topography.
- The Severn Estuary wave model calibration raised concerns about the WW3 model hindcast which shows a tendency towards overpredicting wave conditions, especially for the more extreme waves greater than 5m. It is recommended that this is investigated further.
- Diffraction and reflection are likely underestimated due to the use of a phase averaged wave model.
- The Neural Network 2 tool used to calculate wave overtopping can provide nonsensical results. The tool always gives an overtopping result, unlike its predecessor. Due to the range of conditions calculated for this study and use of a single defence profile, the resultant overtopping rate the tool provides can bounce around depending on when conditions are within the training data or outside of it. An example of this problem occurred when the water level was some distance from the defence crest. At this point there is very little underlying training data, so the tool extrapolates and provides a value which is often much larger than when the water level is closer to the crest. The results sometimes don't increase with AEP and can lead to a discharge curve with large spikes on the rising and falling limbs of the tide. We have had to assess results on a site by site basis to determine a suitable threshold to use based on the underlying training data so the results the tool gives are more sensible. In some cases, overtopping rates have been taken from the neighbouring AEP value where a sensible result could not be obtained from the software.
- Channel conveyance will be overestimated when river channels are smaller than the 5m model cell resolution.
- TUFLOW HPC does not provide the same standard of model stability warning as TUFLOW Classic. Additional model performance checks were undertaken to assess model health and stability as follows:
    - The model was simulated using TUFLOW Classic in the first instance to help identify model issues based on warning messages or poor mass error.

- Checks were made on model timesteps; excessively small timesteps would be a strong indicator of poor model health. A high occurrence of repeated timesteps would indicate an issue in the model data or set up.
- A thorough assessment of the model results was undertaken. Water level fluctuations, flow patterns, performance at boundaries and links were thoroughly assessed.

11 Baseline results summary

TUFLOW flood inundation models were used to simulate a range of design extreme events to map the present day and future coastal flood risk along the Wessex north coast. The TUFLOW models generated cover the following domains:

- Woodspring Bay from Uphill in the south to Clevedon in the north
- Severn House Farm from Aust in the south to Sharpness in the north.

The results from these model simulations are discussed in this chapter and were used to inform updates to the Flood Warning procedures (supplied as a standalone Flood Warning report supplied alongside this project report[15]).

11.1 Woodspring Bay

11.1.1 Defended scenario

Figure 11-1 shows the maximum flood extents for the defended scenario at Woodspring Bay for the smallest (10% AEP) to the largest (0.1% AEP) present-day design events. This area of coastline is subject to flooding from wave overtopping and extreme still water levels, and inundation of properties is present from the smallest event (10% AEP). The model was extended to the south-west to include Weston-super-Mare to allow flood water to flow across the low-lying topography between Woodspring Bay and Weston-super-Mare, however the flood risk at Weston-super-Mare is not being considered as part of this project as wave overtopping was not included. The areas of interest include Woodspring Bay, Clevedon Marine Lake, Sand Bay and Kewstoke.

During the smaller present-day events, the flood mechanism at Woodspring Bay is limited to overtopping of the primary defences and no properties are at flood risk. Still water flooding contributes to the flood risk at Woodspring Bay from the 0.5% AEP, event where the extreme water level breaches the north bank of the Congresbury Yeo Tidal Banks scheme and begins to inundate the adjacent fields. During the largest present-day event (0.1% AEP), a combination of still water and overtopping results in widespread inundation behind Woodspring Bay, from Collum Farm Drive in the west, to Kingston Seymour in the east. Several properties along Ham Lane and Wick Road are at risk in the 0.1% AEP event where flood depths reach up to 0.80m.

The structure at Blind Yeo begins to overtop from the 10% AEP event, although volumes are very small, more relating to spray overtopping, and simply sit near the mouth of the Blind Yeo channel. This is the case for all present-day events and the 0.5% AEP 2068 epoch events; in the 0.1% AEP event, overtopped flood waters propagate along Blind Yeo past Kenn Road but remain in bank. During the 2118 epoch, the area is completely inundated.

At Sand Bay, during the present-day events the flood risk is a result of waves overtopping the dunes onto Beach Road. This occurs initially in the north of Sand Bay during the smallest event (10% AEP). Overtopping occurs onto Beach Road in the central bay during the 0.5% AEP event. As the event severity increases during the 0.1% AEP, larger volumes of overtopping in the northern bay result in flood waters travelling inland and southwards, across the low-lying fields, reaching beyond Elmsley Lane and as far as the holiday park on Sand Road. Overtopping of the dunes onto Beach Road at Kewstoke initially occurs in the largest present-day event (0.1% AEP) and results in two properties being inundated.

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[15] Woodspring Bay and Severn House Farm Flood Forecasting Report v1.0. JBA Consulting, August 2020.

At Clevedon Marine Lake, overtopping first occurs during the 10% AEP event at the western end of the lake where there is a low section of defence. During this event, flood waters enter the car park behind the defence and then run onto Salthouse Road. By the 1.3% AEP event, wave overtopping occurs over the higher section of defence behind the central and eastern ends of Marine Lake. Flood waters in this event start to inundate the northern end of the park to depths of 0.03m, while flood waters flow from Salthouse Road onto Old Church Road. During the largest present day event (0.1% AEP), flood waters overtopping Marine Lake and the promenade fronting Elton Road, flow onto Salthouse Road and into the park, where they continue to propagate along Old Church Road and the adjacent roads, resulting in flood risk to eight residential properties and 13 commercial properties in this area. Flood depths in the park behind Marine Lake become significant from the 0.1% AEP event where depths reach 0.34m in the south east of the park. Additionally, from the 3.3% AEP event, overtopping of the sea wall results in flooding of some of the commercial properties between the promenade and Elton Road.

Figure 11-2 shows the defended 0.5% AEP present day flood extent, overlain the 0.5% AEP 2118 projection under both UKCP09 and NPPF guidance. Under sea level rise conditions, large areas of Woodspring Bay are inundated, with flow paths reaching inland beyond the M5 infrastructure. Significant still water flooding occurs from the 0.5% AEP 2068 UKCP09 event, with flood waters overwashing the banks of the Congresbury Yeo Estuary. In the largest climate change event (0.5% AEP 2118 NPPF), widespread still water flooding results in 32,799 properties being inundated.

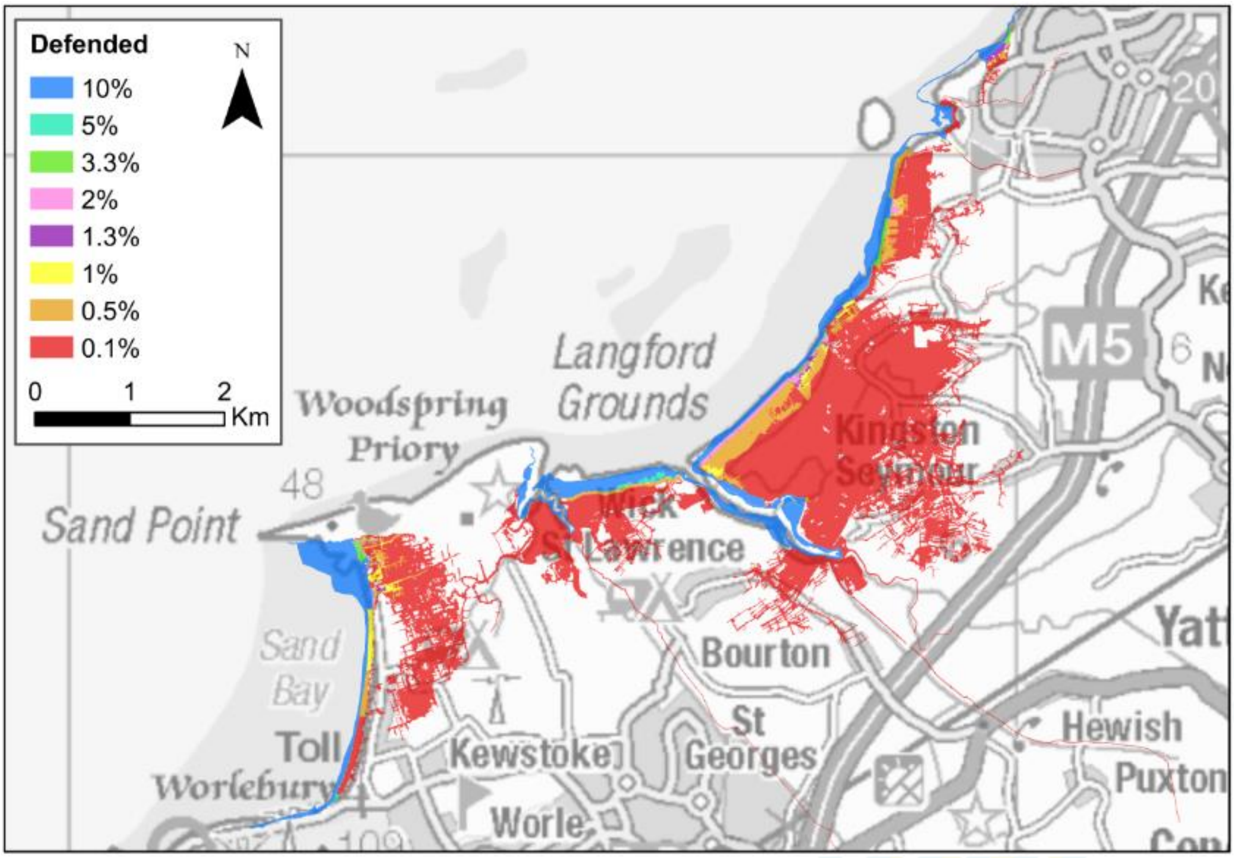

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Figure 11-1: Woodspring Bay Defended scenario present day flood extents

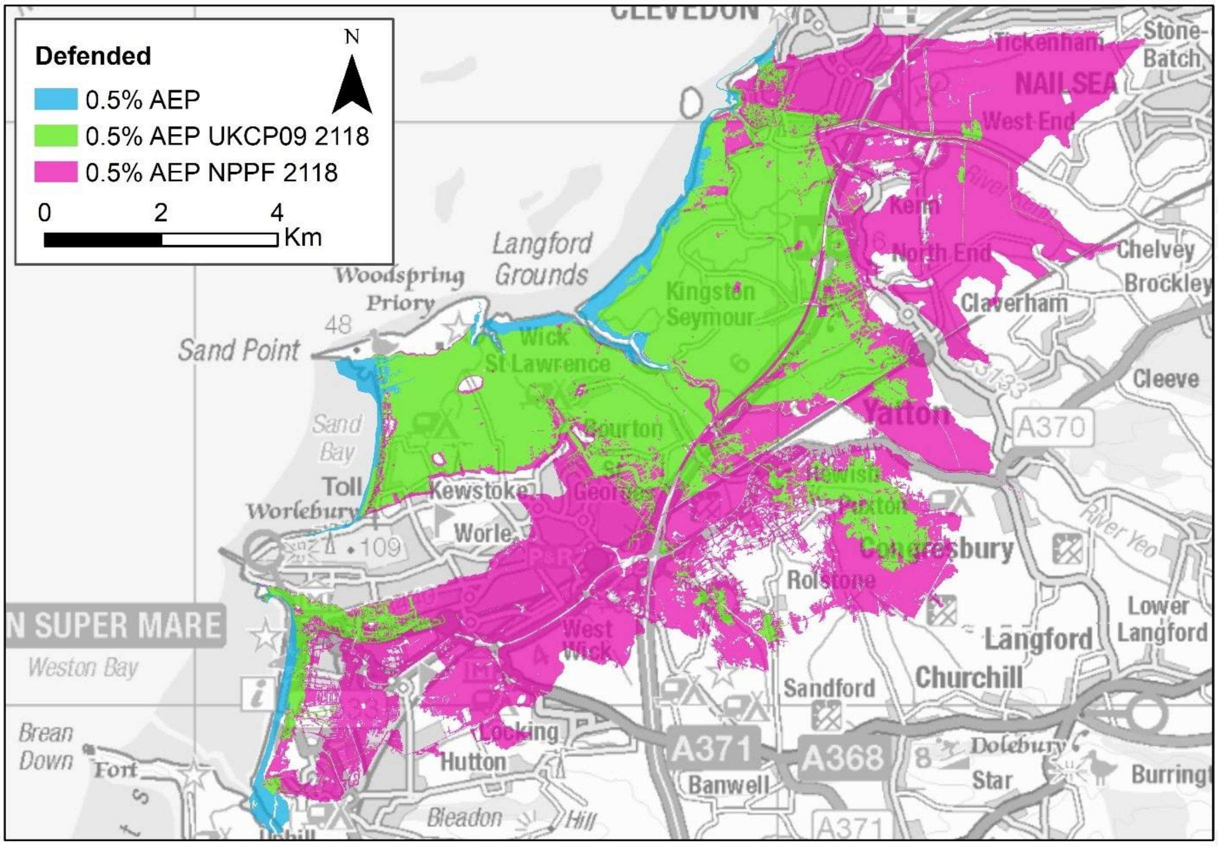

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Figure 11-2: Woodspring Bay Defended scenario 0.5% AEP present day and climate change (2118) comparison

11.1.2 Undefended scenario

Figure 11-3 shows the maximum flood extents for the undefended scenario at Woodspring Bay for the smallest (10% AEP) to the largest (0.1% AEP) present day undefended design events. The undefended flood extents show extensive flooding compared to the defended scenario, where still water flooding is the dominant flood mechanism. Overtopping is applied along the coastal frontage along Sand Bay, but is elsewhere excluded in the undefended modelling due to the topography being generally well below the maximum extreme water levels.

During the smallest present-day undefended event (10% AEP), tidal inundation reaches north of Kewstoke where flood waters meet from both Sand Bay and Woodspring Bay, generating flood depths of >1.50m. Further north at Woodspring Bay, large volumes of flood water inundates the low-lying topography between Woodspring Bay and the M5 infrastructure, whereby flow up the Congresbury Yeo Estuary initially spills onto the floodplain from the undefended river channels. Once the water level exceeds the level of the undefended coastline, widespread flooding occurs where flood waters pass well beyond the M5 infrastructure and spread in the surrounding areas, reaching towns such as Congresbury and Horsecastle. At Clevedon Marine Lake, from the smallest event (10% AEP) large volumes of water flow over Marine Lake and inundate many properties in the south of Clevedon and West End. As the present-day event severity and frequency increase, there is continued widespread still water flooding. In the largest present day event (0.1% AEP), flood depths reach >1.80m along Salthouse Road behind Marine Lake.

Figure 11-4 shows the undefended 0.5% AEP present-day flood extent, overlain the 0.5% AEP 2118 projection under both UKCP09 and NPPF guidance. The undefended climate change flood extents show extensive flooding compared to the undefended present-day scenarios, where still water flooding is the dominant flood mechanism. When compared to the equivalent defended design event, the flood risk is much greater for the undefended scenario. For the largest undefended climate change event (0.5% AEP 2118 NPPF), flood depths increase by almost 2.50m behind Marine Lake on Old Church Road when compared to the equivalent defended event and lead to the inundations of 39,915 properties across the area.

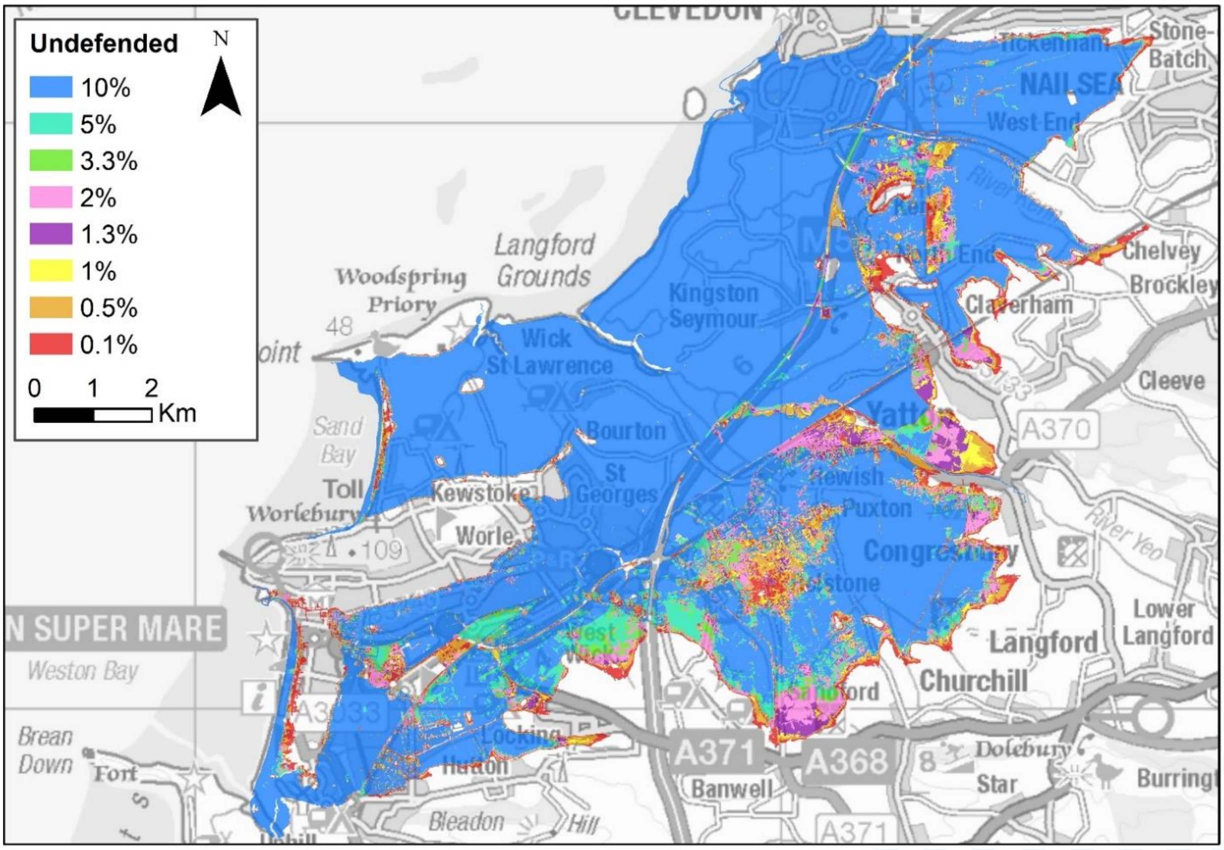

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Figure 11-3: Woodspring Bay Undefended scenario present day flood extents

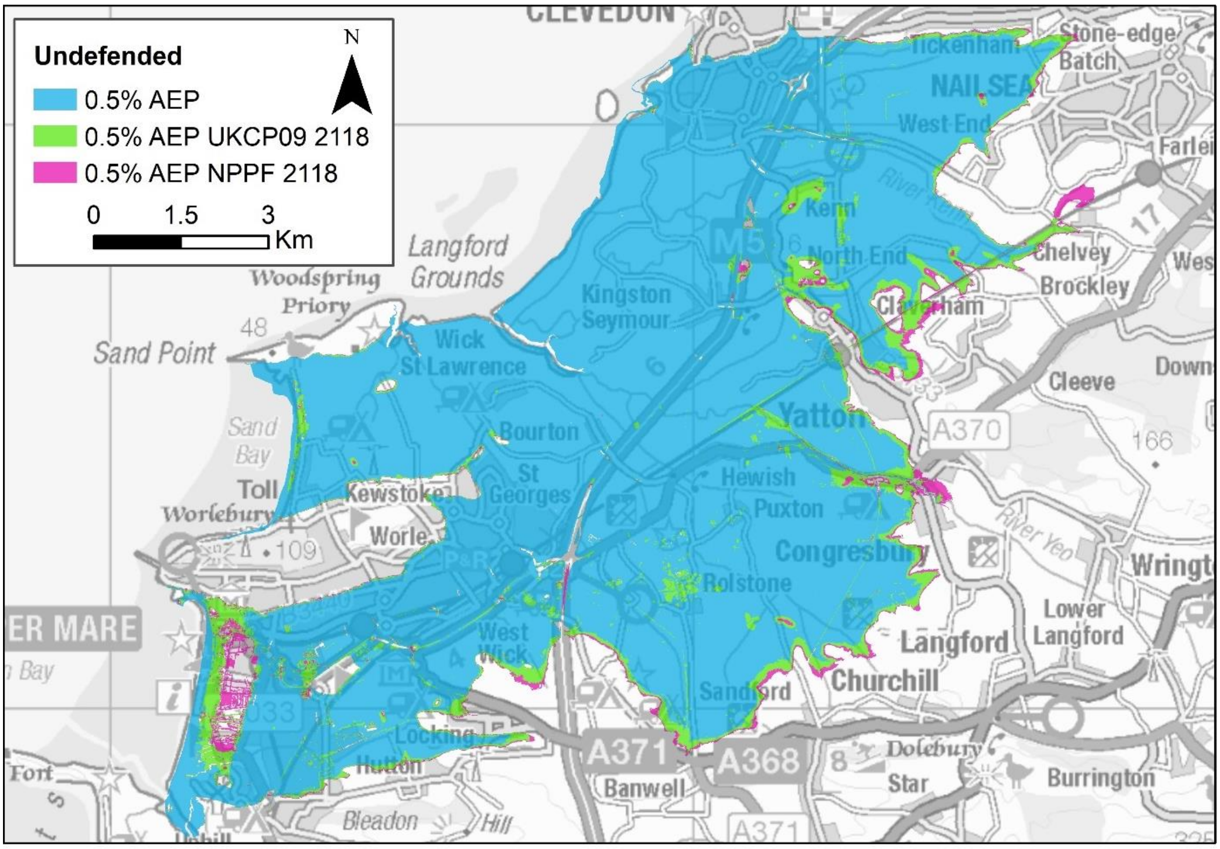

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Figure 11-4: Woodspring Bay Undefended scenario 0.5% AEP present day and climate change (2118) comparison

11.1.3 Property counts

Property counts for the modelled events at Woodspring Bay are shown in Table 11-1.

Note the property count approach is discussed in Chapter 16.

Table 11-1: Woodspring Bay property counts for the defended and undefended scenario

Scenario	AEP	Properties at flood risk				
		Commercial	Residential	Critical Infrastructure	Unclassified	Total
Defended	10%	28	4	1	13	46
	5%	30	4	1	13	48
	3.3%	34	4	1	15	54
	2%	34	6	1	16	57
	1.3%	38	6	1	17	62
	1%	38	6	1	17	62
	0.5%	49	8	5	26	88
	0.1%	61	100	10	86	257
	0.5% UKCP09 2068	66	89	10	82	247
	0.5% UKCP09 2118	577	3,402	46	1,006	5,031
	0.5% NPPF 2068	136	386	13	262	797
	0.5% NPPF 2118	1,838	27,511	194	3,256	32,799
Undefended	10%	961	23,396	126	2,544	27,027
	5%	1,062	25,301	140	2,741	29,244
	3.3%	1,100	25,934	143	2,845	30,022
	2%	1,228	27,053	154	3,090	31,525
	1.3%	1,304	27,481	159	3,245	32,189
	1%	1,367	27,792	165	3,363	32,687
	0.5%	1,497	28,540	178	3,639	33,854
	0.1%	1,731	29,673	195	4,015	35,614
	0.5% UKCP09 2068	1,756	29,762	197	4,041	35,756
	0.5% UKCP09 2118	2,176	31,242	231	4,393	38,042
	0.5% NPPF 2068	1,856	30,026	204	4,098	36,184
	0.5% NPPF 2118	2,452	32,541	261	35,254	39,915

11.1.4 Flood Zones

Updated Flood Zones were generated at Woodspring Bay and shown on Figure 11-5.

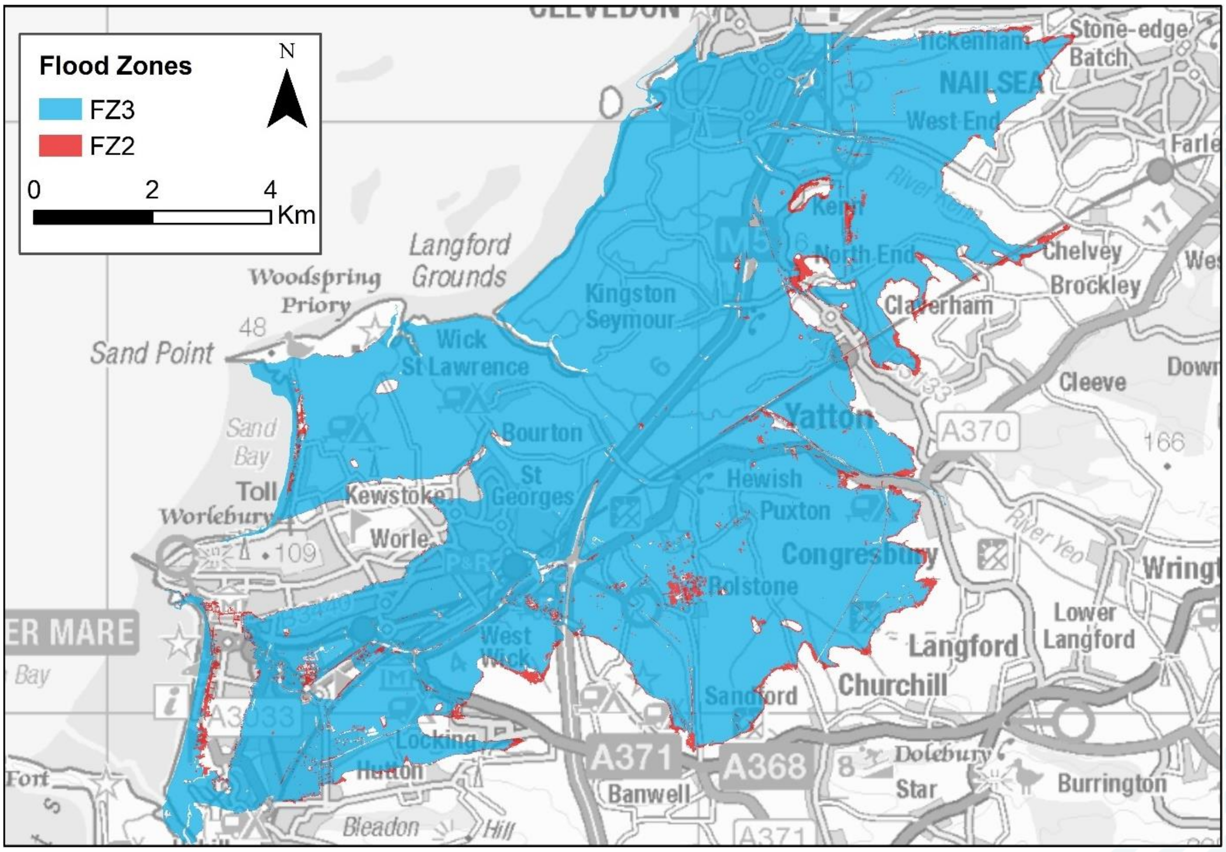

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Figure 11-5: Woodspring Bay Flood Zones

11.2 Severn House Farm

11.2.1 Defended scenario

Figure 11-6 shows the maximum flood extents for the defended scenario at Severn House Farm for the smallest (20% AEP) to the largest (0.1% AEP) present day design events.

This area along the Severn Estuary from Aust to Sharpness, is subject to flooding from wave overtopping and extreme still water levels, and inundation of properties is present from the smallest present-day design event (20% AEP). The key areas of interest in the Severn House Farm model include Oldbury-on-Severn, Shepperdine, Berkeley and Sharpness.

During the smaller more frequent present-day events, flood risk along the estuary is due to waves overtopping the grassed earth embankments that spans the Severn Estuary banks. Flood risk is limited to the immediate area behind the Severn Estuary banks during the smaller events (20%, 10%, 5%, 3.3%, 2% AEPs). Wave overtopping initiates during the 10% AEP event, leading to small volumes immediately behind the embankment between Oldbury-on-Severn and Shepperdine. By the 5% AEP, small volumes of overtopping are passing over Berkeley Pill, and by the 2% AEP overtopping volumes reach the edge of Nupdown. Additional overtopping volumes are evident during the 1.3% and 1% AEP events around the Nupdown area in particular, leading to three and five properties being inundated respectively.

Small volumes of overtopping impact Oldbury Power Station from the 5% AEP event at the grassed area to the front of the site, and as of the 10% AEP event small volumes of overtopping begin to propagate along the road network into the site and reach depths of up to 0.10m. At Berkeley Power Station and Technology Centre, wave overtopping begins to impact the road network at the site as of the 1.3% AEP event, with more significant flooding of the site occurring during the 0.5% AEP event. Any flood defences at the power stations were not available to be included in the modelling, and this is likely the cause the early onset of flooding at these important locations.

As the event severity increases, a combination of still water and wave overtopping flooding results in significant flood risk to Nupdown, Shepperdine and Oldbury Naite as flood waters travel inland. There is a large jump in the number of properties inundated from the 58 properties in the 0.5% AEP event to 669 during the 0.1% AEP event, where flood depths reach >1.40m on Church Road in Oldbury-on-Severn. The 0.1% AEP event also leads to Shepperdine and Berkeley being inundated and flood water begins to propagate along Saniger Lane in Sharpness.

Figure 11-7 shows the defended 0.5% AEP present day flood extent, overlain the 0.5% AEP 2118 projection under both UKCP09 and NPPF guidance. Under sea level rise conditions, large areas of the Severn House Farm model domain are inundated, with flow paths reaching inland up to the M4 infrastructure in the south and Rockhampton in the east. During the largest defended climate change event (0.5% AEP 2118 NPPF), widespread still water flooding results in 1,870 properties being inundated.

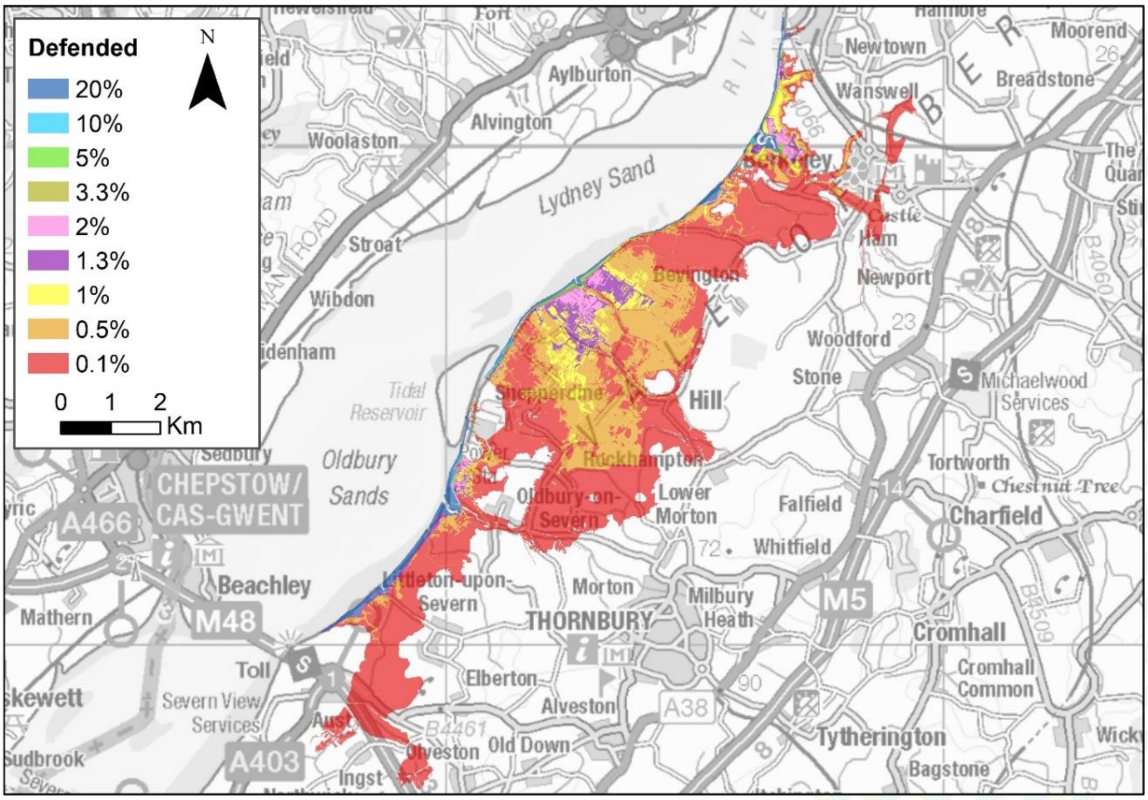

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Figure 11-6: Severn House Farm Defended scenario present day flood extents

![JBA consulting]

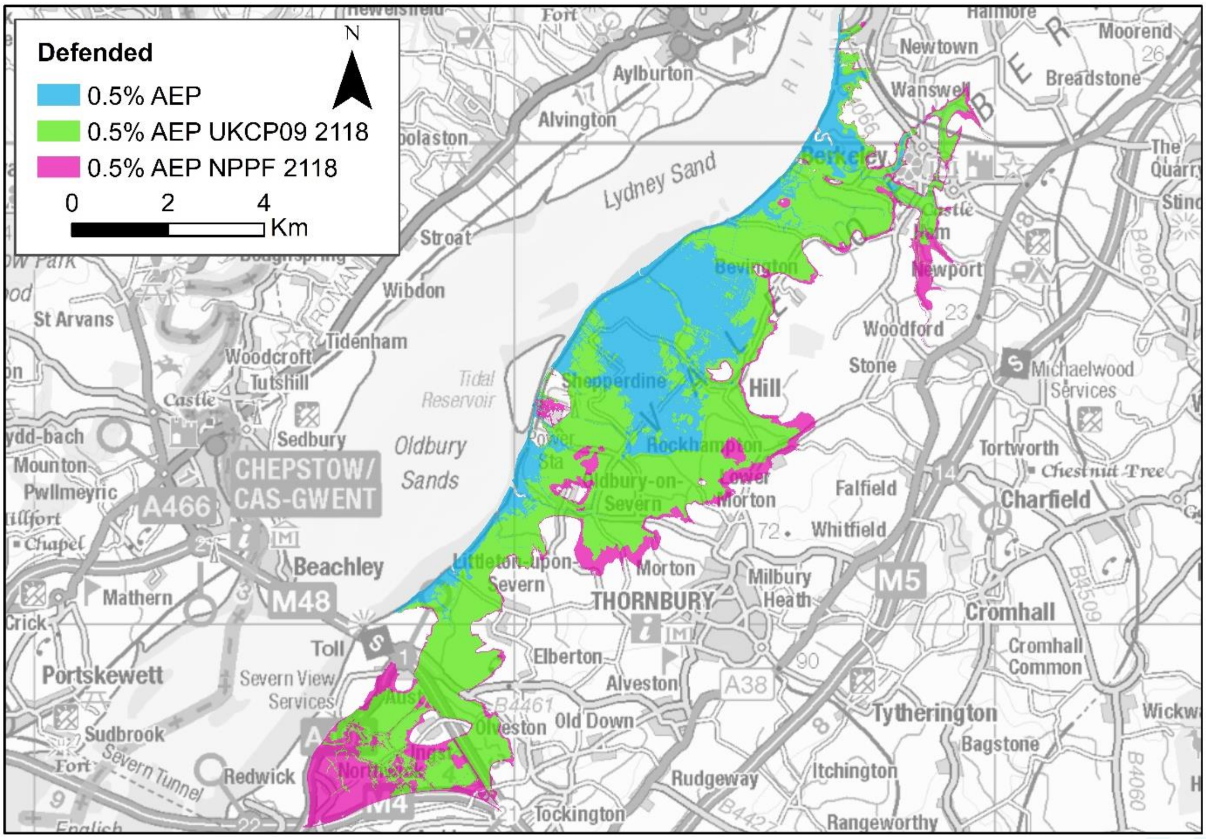

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Figure 11-7: Severn House Farm Defended scenario 0.5% AEP present day and climate change (2118) comparison

11.2.2 Undefended scenario

Figure 11-8 shows the maximum flood extents for the undefended scenario at Severn House Farm for the smallest (20% AEP) to the largest (0.1% AEP) present day design events. The undefended flood extents show extensive flooding compared to the defended scenario, where still water flooding is the dominant flood mechanism. Overtopping is applied behind the embankments at Oldbury Power Station but is elsewhere excluded in the undefended modelling as topographic levels are well below extreme water levels.

During the smallest present-day undefended event (20% AEP), flood waters initially propagate up Oldbury Pill, spilling onto the adjacent fields. This is followed by still water flooding of the undefended coastline, impacting Oldbury-on-Severn, and further north inundating Severn House Farm. During the maximum extreme sea level in the present-day undefended 20% AEP event, flooding is widespread, with flow paths reaching inland up to the M4 infrastructure in the south and Rockhampton in the east. As the event severity and frequency increases, much of the low-lying topography becomes inundated with 1,590 properties at flood risk during the largest present-day undefended event (0.1% AEP).

Figure 11-9 shows the undefended 0.5% AEP present-day flood extent, overlain the 0.5% AEP 2118 projection under both UKCP09 and NPPF guidance. The undefended climate change flood extents show greater inundation compared to the undefended present-day scenarios, where still water flooding is the dominant flood mechanism. When compared to the equivalent defended design event, the flood risk is greater for the undefended scenario, particularly in the south where flood waters propagate beyond the M48 in the undefended modelling (0.1% AEP event). For the largest undefended climate change event (0.5% AEP 2118 NPPF), the undefended extent is only marginally greater than the defended scenario. This is because the dominant flood mechanism is still water flooding in both the defended and undefended modelling. However, the main difference between the defended and undefended scenario is seen at Oldbury Power Station, which is fully inundated during the undefended scenario. Flood depths on the roads at Oldbury Power Station are >0.50m deeper in the undefended when compared to the defended equivalent.

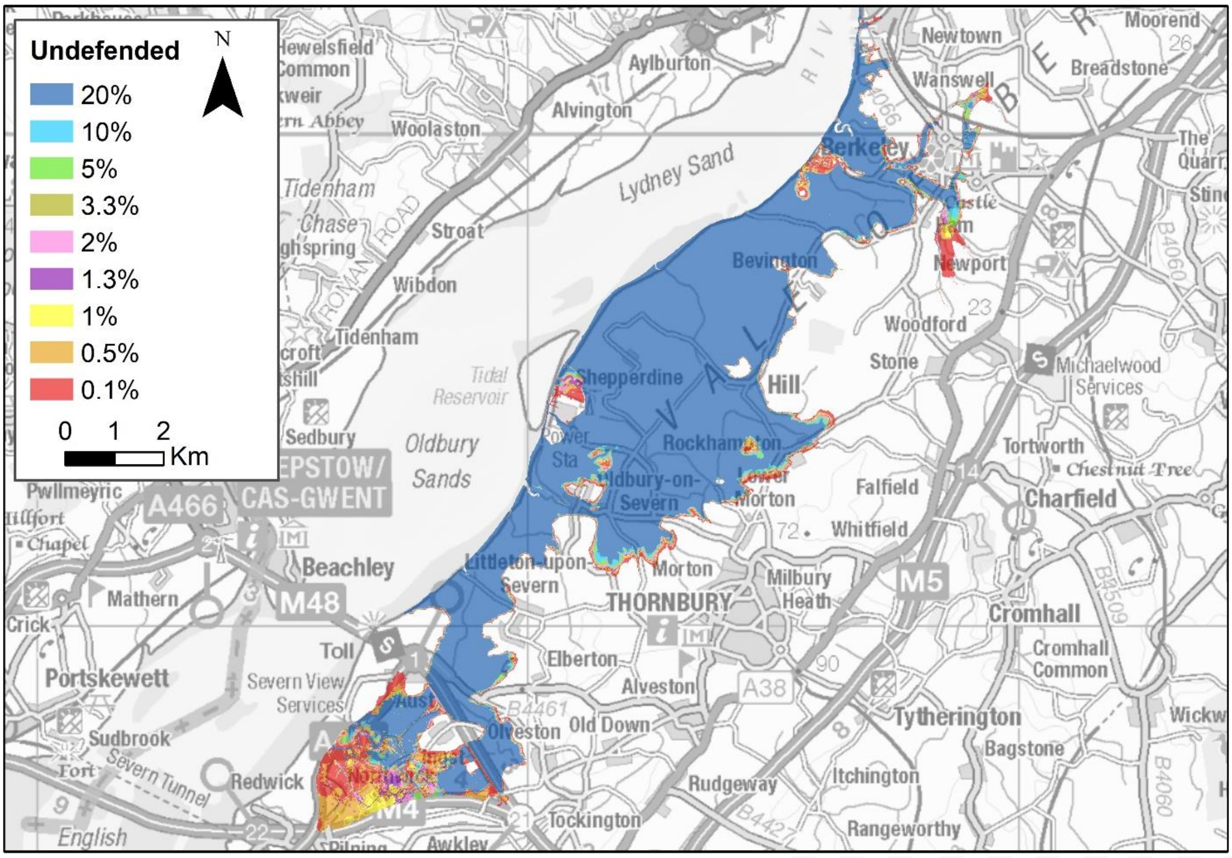

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Figure 11-8: Severn House Farm Undefended scenario present day flood extents

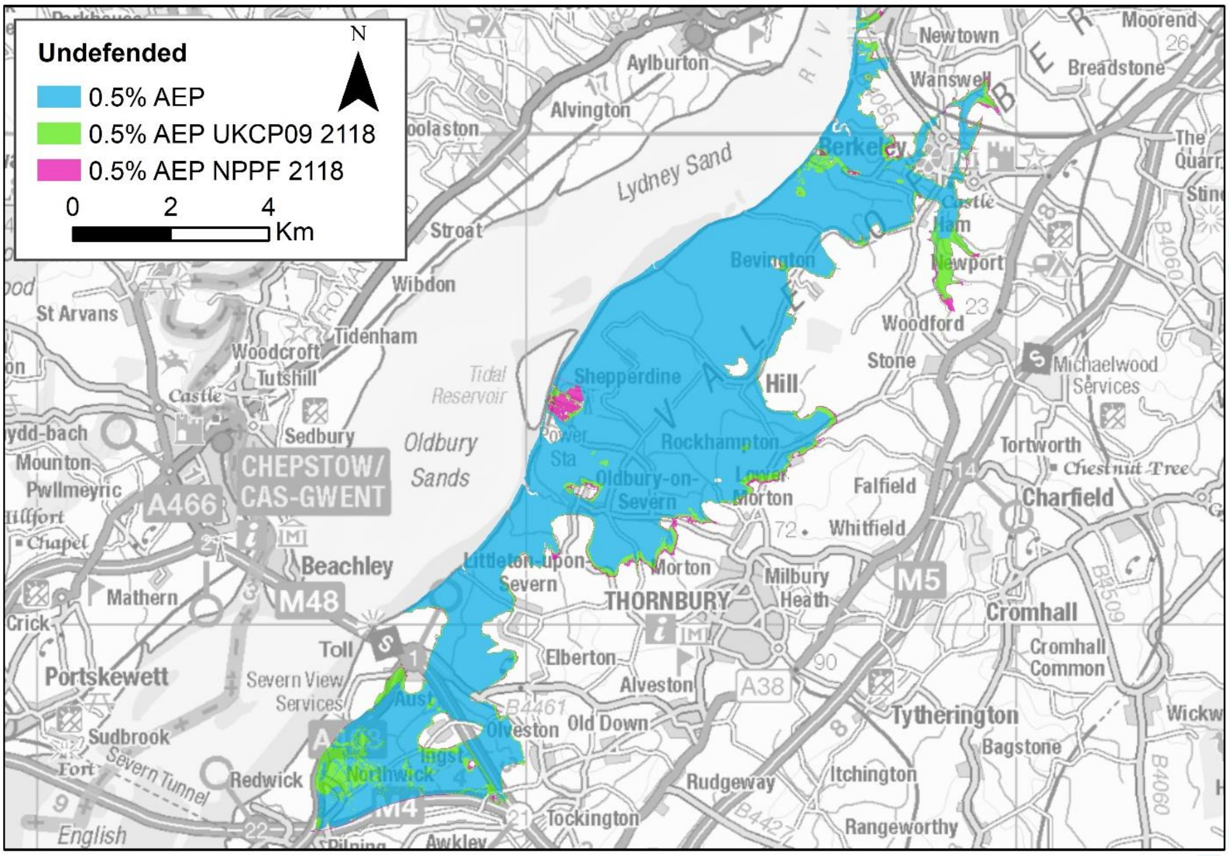

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Figure 11-9: Severn House Farm Undefended scenario 0.5% AEP present day and climate change (2118) comparison

11.2.3 Property counts

Property counts for the modelled events at Severn House Farm are shown in Table 11-2. Note the property count approach is discussed in Chapter 11.3.

Table 11-2: Severn House Farm property counts for the defended and undefended scenario

Scenario	AEP	Properties at flood risk				
		Commercial	Residential	Critical Infrastructure	Unclassified	Total
Defended	20%	1	0	1	3	5
	10%	1	0	2	5	8
	5%	1	0	2	6	9
	3.3%	1	0	2	6	9
	2%	1	0	2	7	10
	1.3%	1	0	2	8	11
	1%	2	0	3	9	14
	0.5%	2	5	3	48	58
	0.1%	29	152	10	478	669
	0.5% UKCP09 2068	23	134	9	832	998
	0.5% UKCP09 2118	41	280	14	832	1,167
	0.5% NPPF 2068	34	183	10	600	827
	0.5% NPPF 2118	56	504	23	1,287	1,870
Undefended	20%	33	258	12	707	1,010
	10%	33	270	12	744	1,059
	5%	33	281	14	781	1,109
	3.3%	34	286	14	793	1,127
	2%	35	302	15	826	1,178
	1.3%	36	305	15	842	1,198
	1%	37	311	15	856	1,219
	0.5%	40	345	15	910	1,310
	0.1%	46	417	19	1,108	1,590
	0.5% UKCP09 2068	45	413	18	1,087	1,563
	0.5% UKCP09 2118	54	492	23	1,189	1,758
	0.5% NPPF 2068	48	425	20	1,181	1,674
	0.5% NPPF 2118	56	542	23	1,318	1,939

11.2.4 Flood Zones

Updated Flood Zones were generated at Severn House Farm and are shown on Figure 11-10.

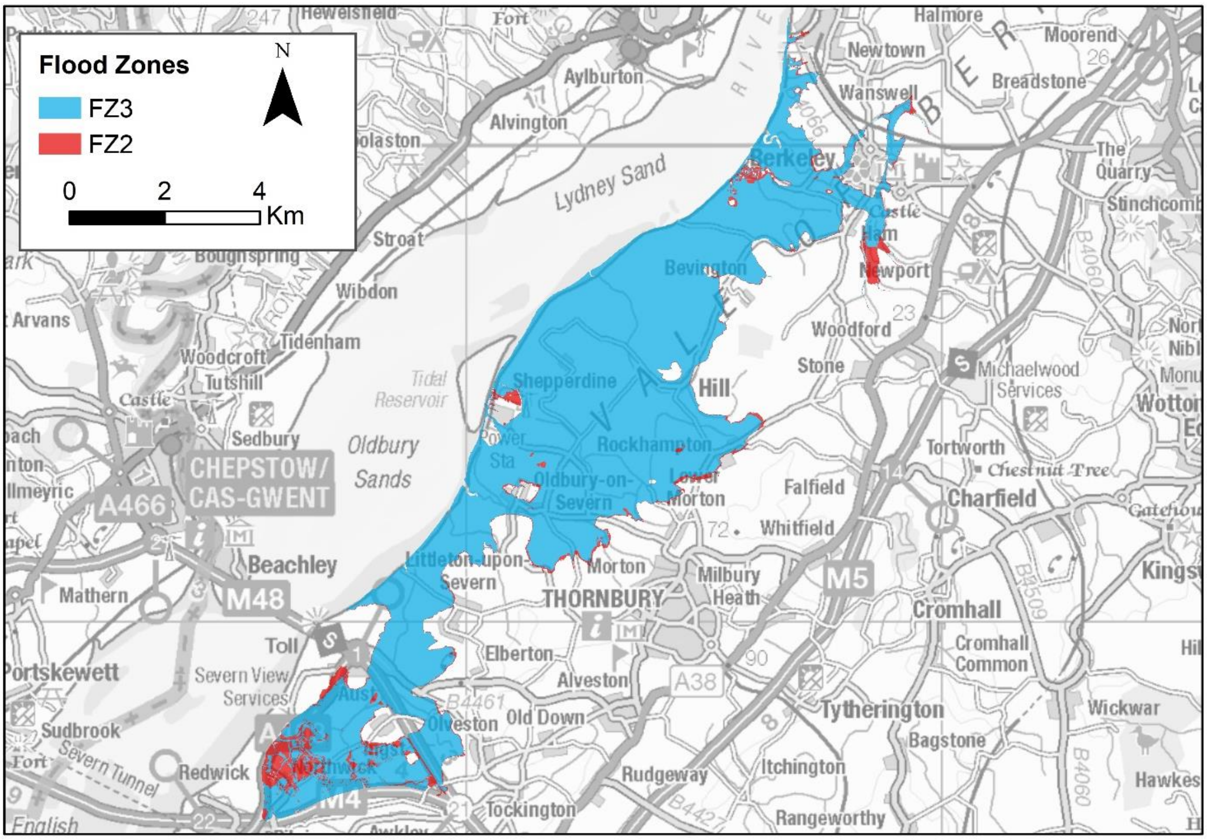

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Figure 11-10: Severn House Farm Flood Zones

11.3 Defended and undefended extent discussion

Following the removal of the formal defence network from the Severn House Farm model topography, the flood extents and depths generally increase as would be expected, due to far larger volumes of floodwater propagating onto the low-lying topography of the area.

However, under sea level rise conditions in the 2068 and 2118 epoch, in some parts of the domain the undefended maximum extent is smaller than the defended. The reason for this is that during the undefended scenario, the removal of the defence network allows flood water to drain from the flood plain with each successive tidal cycle. During the defended scenario, flood water is trapped by the defences and not allowed to return to sea. Consequently, over successive tidal cycles still water and wave overtopping flood waters propagate further into the model domain and to greater depths in some cases than the undefended flood waters.

An example of this occurs north east of Berkeley and south of the domain at Aust as illustrated on Figure 11-11 and Figure 11-12 respectively.

In the Woodspring Bay model, defended overtopping inflows were included at Sand Bay frontage in the undefended model simulations. They were included because there are some areas of high ground at Sand Bay after defences are removed, and still water flooding alone is limited across the frontage. Therefore, the defended inflows were included to prevent the undefended flood extents from being smaller than the defended. However, as the Sand Bay sand dunes were flattened along the coastal frontage in the undefended scenario, the overtopping inflows could drain directly back to sea, while in the defended the raised defences force more overtopping landwards. The flattening of the dune system is the reason that the undefended extents are still smaller than the defended extents, in a few small areas at Sandy Bay, despite the defended overtopping being included.

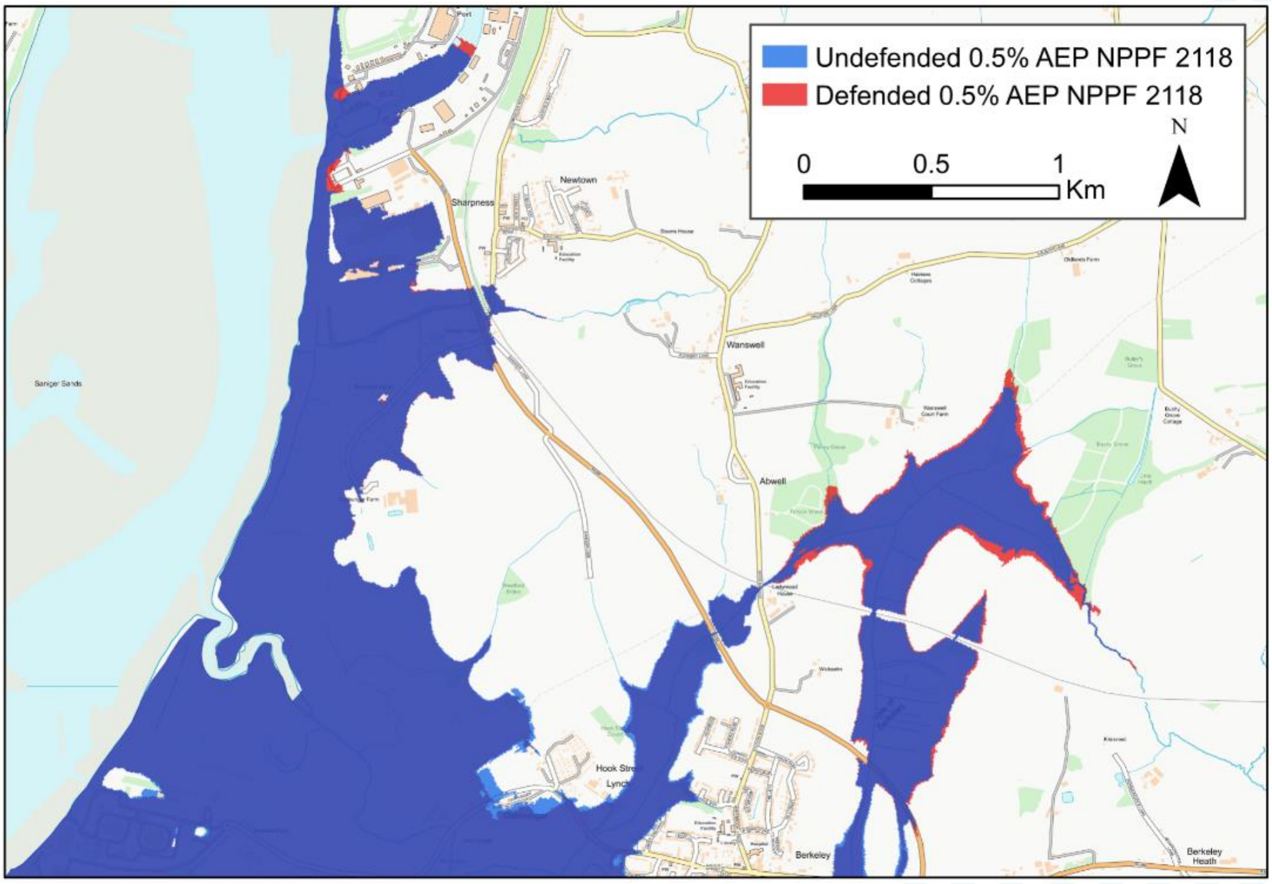

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Figure 11-11: Severn House Farm defended and undefended comparison at Berkeley

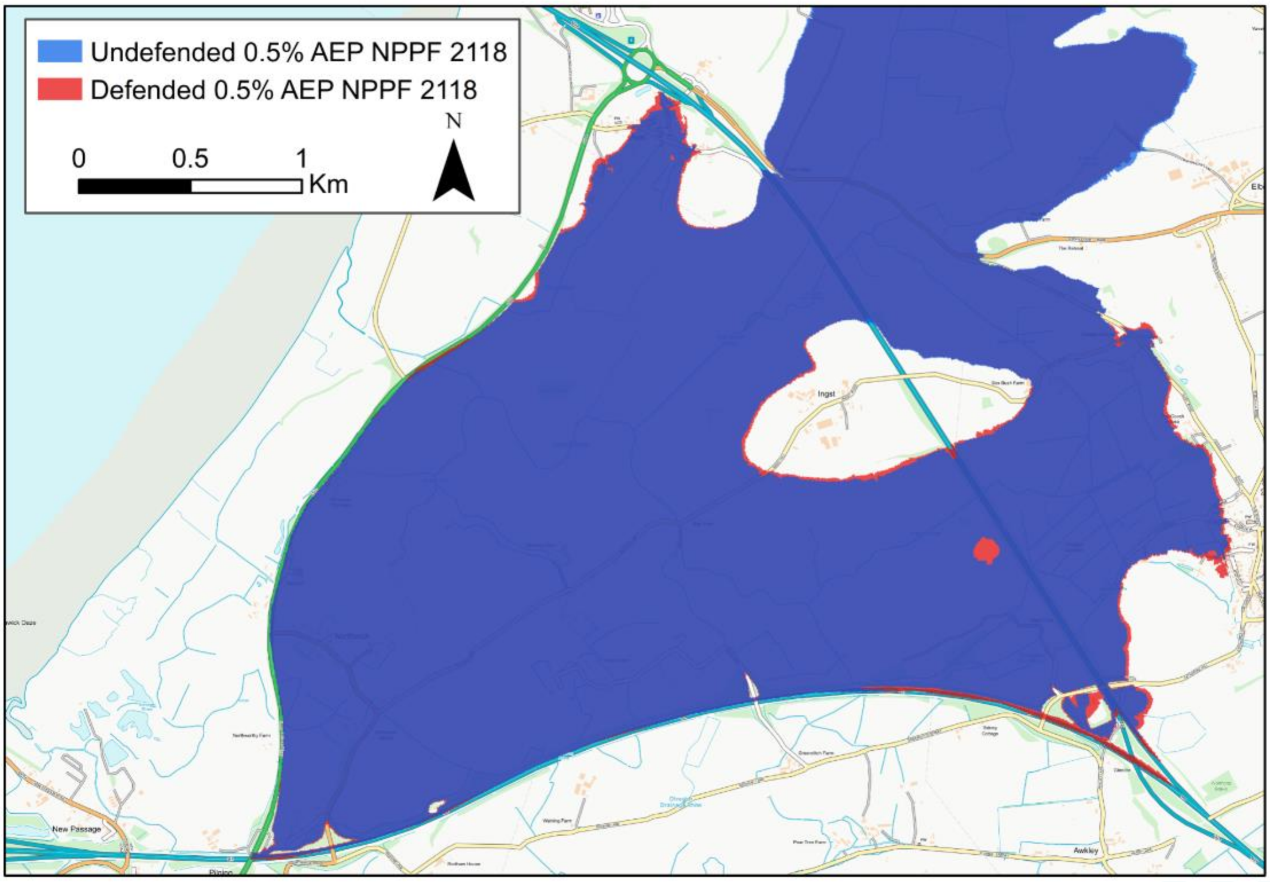

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Figure 11-12: Severn House Farm defended and undefended comparison at Aust

12 Woodspring Bay marsh loss results summary

Marsh erosion at Woodspring Bay was modelled to investigate the impact of an eroded beach state compared to the current 'Hold The Line' policy. This involved lowering the existing topography by 0.5m to represent future erosion of the marsh, and calculating overtopping rates for the eroded marsh.

The eroded marsh model simulations were compared to the baseline defended scenario to assess the role that the marsh plays in protecting Woodspring Bay against tidal flooding. Figure 12-1 shows the present-day modelled flood extents for the Marsh Loss scenario and Figure 12-2 shows a comparison of the present-day 0.5% and 0.1% AEP events for the Marsh Loss scenario and the baseline defended.

During the smaller more frequent events (10, 5 and 2% AEP), the difference between the Marsh Loss scenario and the baseline defended is minimal. The marsh loss leads to a small additional volume of overtopping between the primary and secondary defences at Wick and Kingston Seymour and the fields between Blind Yeo and Broadstone Rhyne. During the 0.5% AEP event, the marsh loss leads to additional overtopping volumes over the secondary defence at Wick, but the volumes are small enough to limit the flood risk to the immediate area behind the secondary defence. Due to the relatively small impact that the marsh loss was shown to have on modelled overtopping volumes, the number of properties at risk during the 10% and 5% AEP events remains the same as the defended baseline (Table 12-1). One and two additional properties are inundated during the 2% and 0.5% AEP events when compared to the defended baseline respectively. However, during the largest present-day Marsh Loss event (0.1% AEP), considerable additional volumes of overtopping are seen behind the secondary defence at Wick, and Kingston Seymour where flood waters reach the M5 infrastructure, and additional risk further north in particular around Lower Strode Road. The additional flood risk generally leads to flood depths increasing by up to 0.15m from that of the baseline and an additional 43 properties being inundated compared to the baseline defended.

It is noted that during the 0.1% AEP event, the marsh loss leads to larger volumes of wave overtopping and more significant flood risk than the baseline defended. However, to the western end of Woodspring Bay, around Woodspring Farm and Kingsfield Farm, the 0.1% AEP marsh loss flood extent is slightly reduced from that of the defended baseline (Figure 12-3). The reason for this is that the maximum water level reaching the River Banwell inlet seems to be impacted by the overtopping volumes at overtopping profile WO_10 within the Woodspring Bay flood inundation model. The overtopping fills up the area between the primary and secondary defence at Wick and then flows over the primary embankment to the west. This restricts the offshore water level boundary propagation in this location such that peak levels are reduced by 0.005m. This slight reduction in maximum water level leads to a reduction in flood extent in this location in the 0.1% AEP event, as this location is driven by still water flood risk. There is no overtopping profile in this location, and still water flooding occurs over the Priory car park and River Banwell embankments.

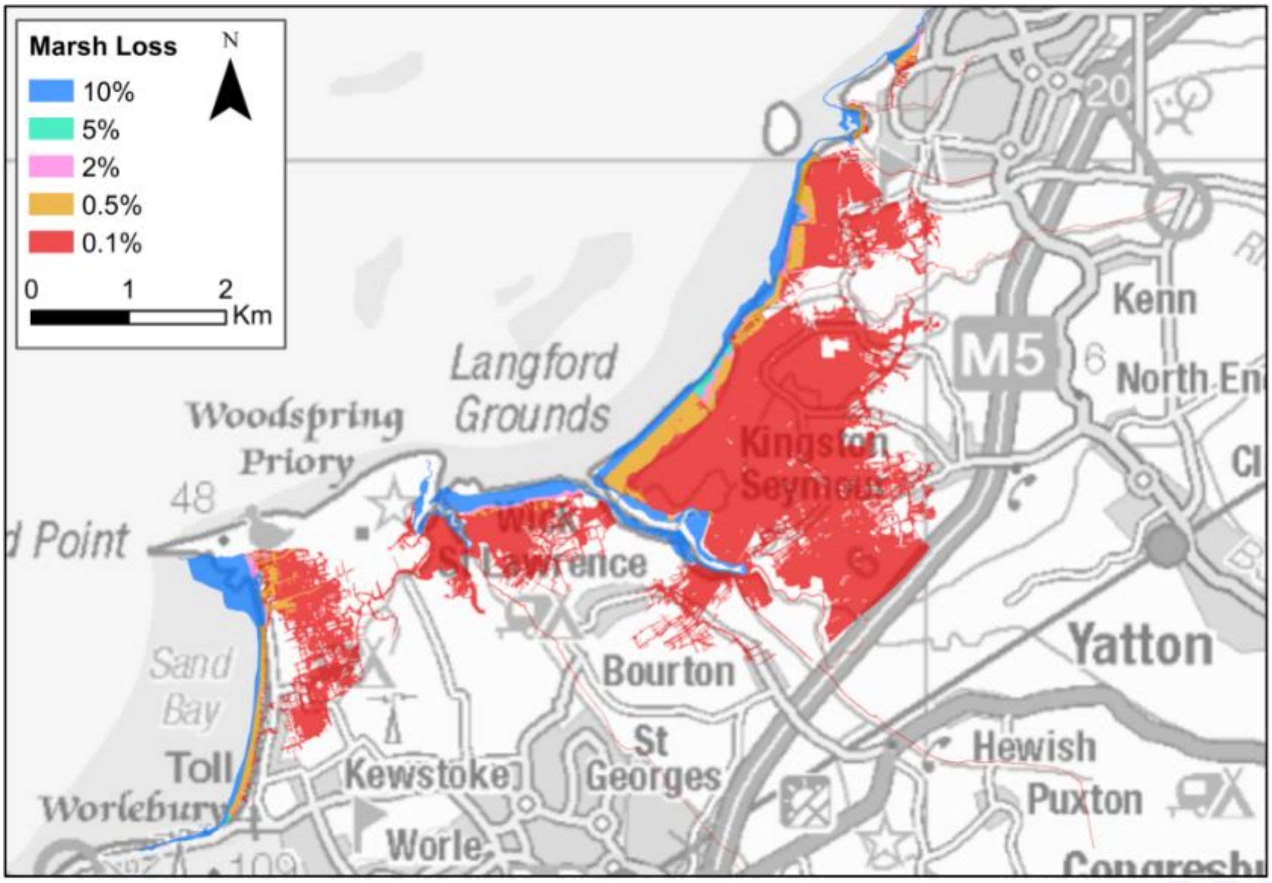

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Figure 12-1: Woodspring Bay Marsh Loss scenario present day flood extents

Table 12-1: Woodspring Bay property counts for the marsh loss scenario

Scenario	AEP	Properties at flood risk				
		Commercial	Residential	Critical Infrastructure	Unclassified	Total
Marsh Loss	10%	28 (28)	4 (4)	1 (1)	13 (13)	46 (48)
	5%	30 (30)	4 (4)	1 (1)	13 (13)	48 (48)
	2%	34 (34)	6 (6)	1 (1)	17 (16)	58 (57)
	0.5%	49 (49)	8 (8)	5 (5)	28 (26)	90 (88)
	0.1%	67 (61)	103 (100)	10 (10)	120 (86)	300 (257)

*Numbers in brackets represent the defended property counts

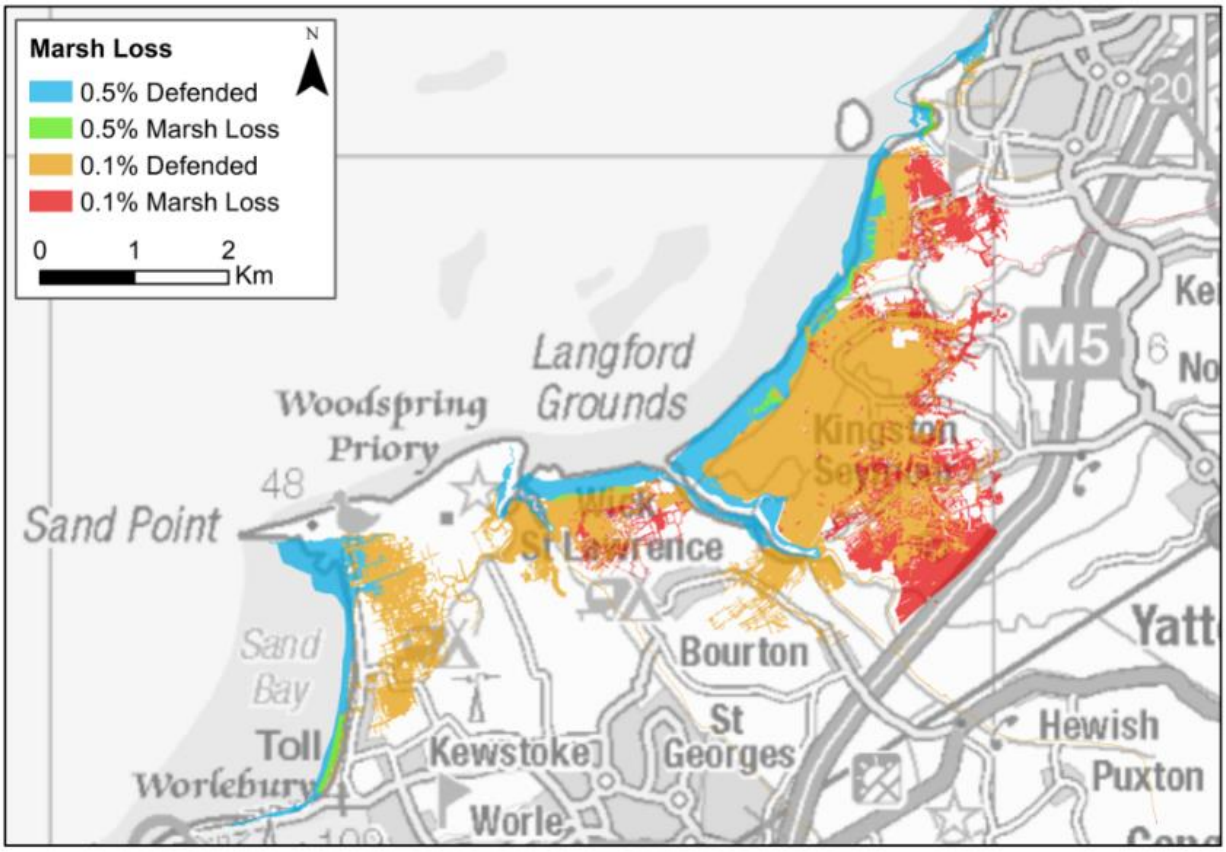

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Figure 12-2: Woodspring Bay Marsh Loss scenario comparison with Defended scenario

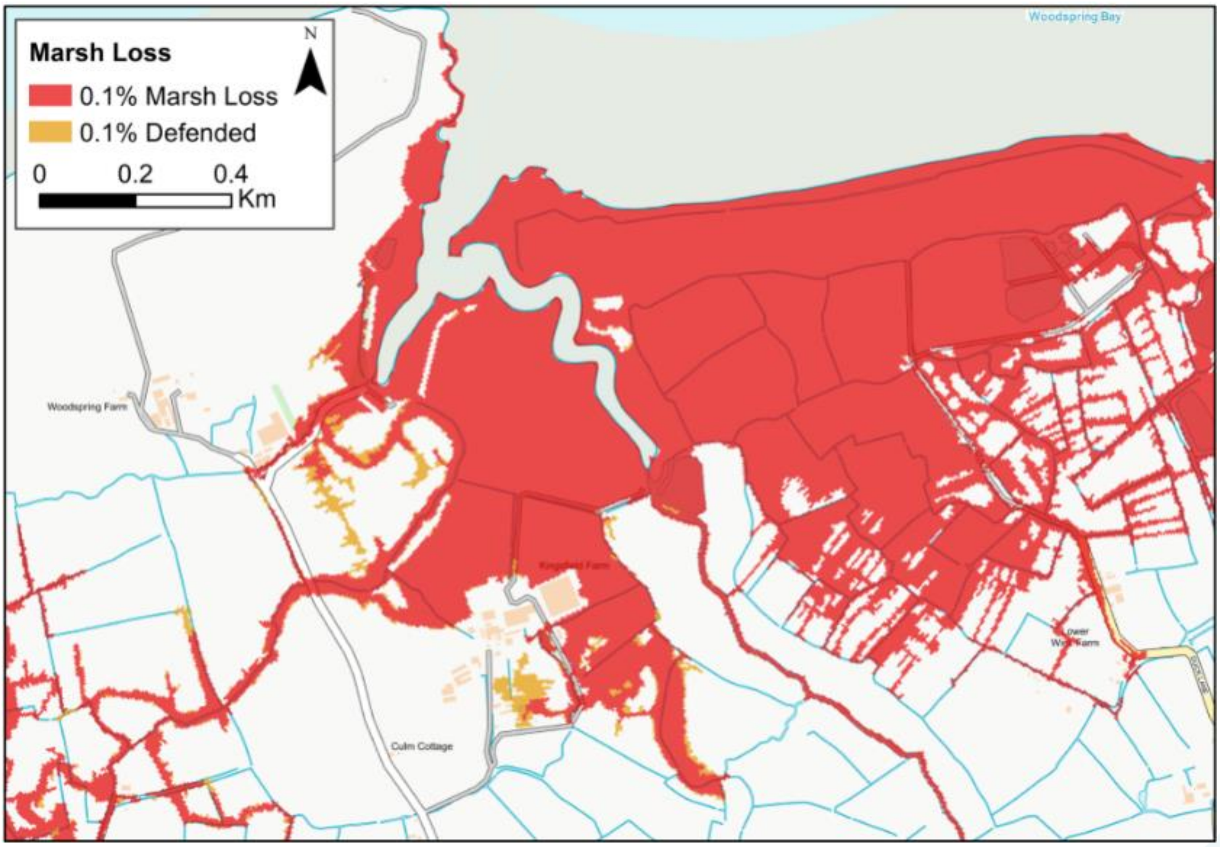

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Figure 12-3: 0.1% AEP marsh loss and baseline comparison

13 Woodspring Bay Wick St Lawrence defence removal results summary

The primary defence at Wick St Lawrence was removed from the Woodspring Bay model topography to investigate the role the primary defence has in protecting Woodspring Bay from tidal flooding. The results from the primary defence removal scenario was compared to the baseline defended results. Figure 13-1 shows the present-day and climate change (UKCP09 2068) flood extents for the 0.5% AEP event for both the defended baseline and the Removal Wick scenarios. During the present-day AEP events (2%, 1.3%, 1% and 0.5%), the difference between the Wick defence removal scenario flood extents and the defended baseline are minimal. Wave overtopping over the secondary defence at Wick occurs during the 0.5% AEP in the baseline flood modelling, however the same overtopping occurs during the 2% AEP when the primary defence is removed. The flood risk is however limited to the immediate area behind the secondary defence for all present-day events. This leads to an additional one property being inundated during the 2%, 1.3% and 1% AEPs while during the 0.5% AEP the property counts stays the same when compared to the defended baseline.

During the 0.5% AEP UKCP09 2068 climate change model simulation, a bigger impact is seen as a consequence of removing the primary defence. Additional flood risk is seen around Warth Lane and at Woodspring Farm and Kingsfield Farm. The difference is relatively small with flood depths generally increasing by up to 0.05m. The number of properties at risk in the Wick defence removal scenario remain the same as the baseline defended for all of the modelled events (Table 13-1).

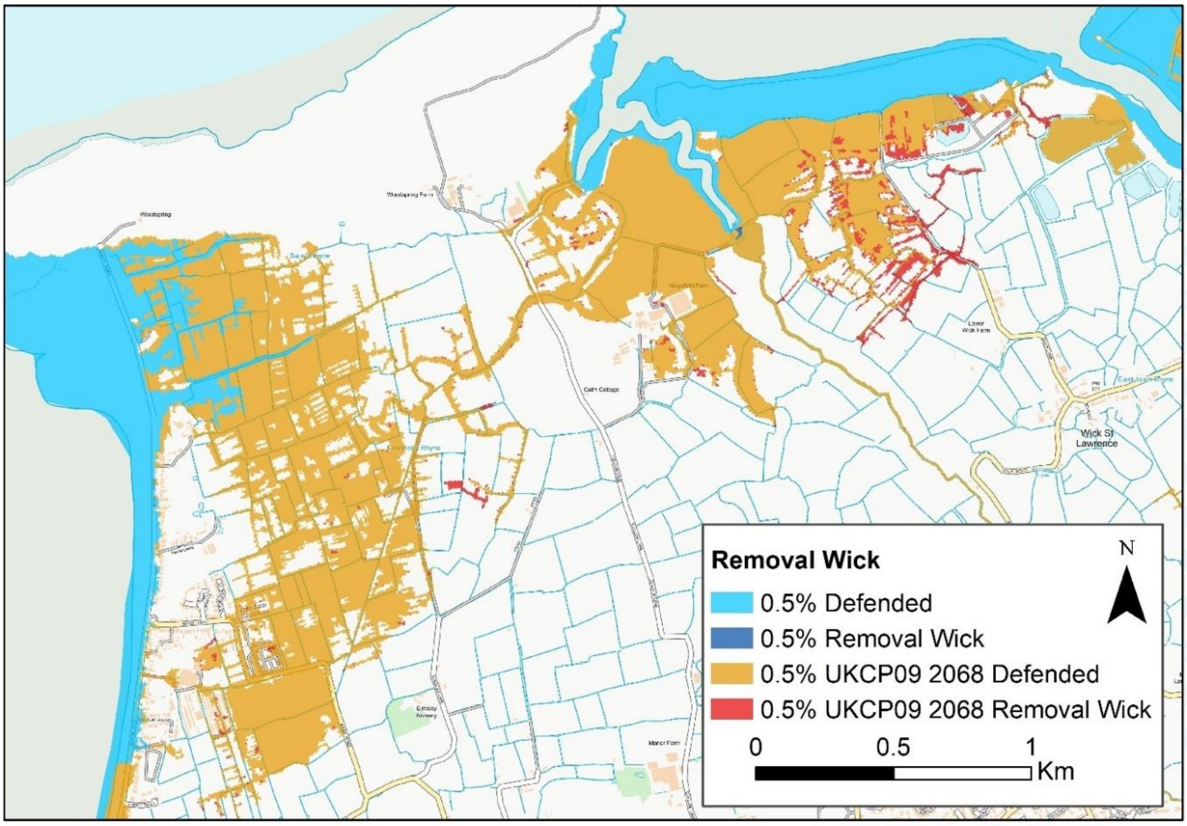

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Figure 13-1: Woodspring Bay Wick St Lawrence Defence Removal scenario comparison with Defended

Table 13-1: Woodspring Bay property counts for the Wick St Lawrence defence removal scenario

Scenario	AEP	Properties at flood risk				
		Commercial	Residential	Critical Infrastructure	Unclassified	Total
Defence removal Wick St Lawrence	2%	34 (34)	6 (6)	1 (1)	17 (16)	58 (57)
	1.3%	38 (38)	6 (6)	1 (1)	18 (17)	63 (62)
	1%	38 (38)	6 (6)	1 (1)	18 (17)	63 (62)
	0.5%	49 (49)	8 (8)	5 (5)	26 (26)	88 (88)
	0.5% AEP UKCP09 2068	66 (66)	89 (89)	10 (10)	82 (82)	247 (247)

*Numbers in brackets represent the defended property counts

14 Woodspring Bay Kingston Seymour defence removal results summary

The secondary defence at Kingston Seymour was removed from the Woodspring Bay model topography to investigate the role the secondary defence has in protecting Woodspring Bay from tidal flooding. The results from the Kingston Seymour defence removal scenario were compared to the baseline defended results. Figure 14-1 shows the present-day and climate change (UKCP09 2068) flood extents for the 0.5% AEP event for both the defended baseline and the Kingston Seymour defence removal scenarios. During the present-day AEP events (2%, 1.3%, 1% and 0.5%), the Kingston Seymour defence removal scenario flood extents are larger than the baseline defended, however the risk to properties remains the same as the baseline in the 2% and 1.3% AEP events while a single additional property is inundated during the 1% and 0.5% AEP events. Some additional flood risk to Back Lane and around Wharf Farm is evident but does not lead to additional property inundation. The removal of the defence leads to a greater impact under sea level rise conditions. During the largest climate change event (0.5% AEP UKCP09 2068), there are 29 more properties at risk in the Kingston Seymour defence removal scenario as flood waters reach as far as the M5 infrastructure and lead to further inundation of Yeo Bank Lane, Ham Lane, Middle Lane and Back Lane (Table 14-1). Flood depths generally increase by between 0.15 and 0.20m from that of the baseline scenario.

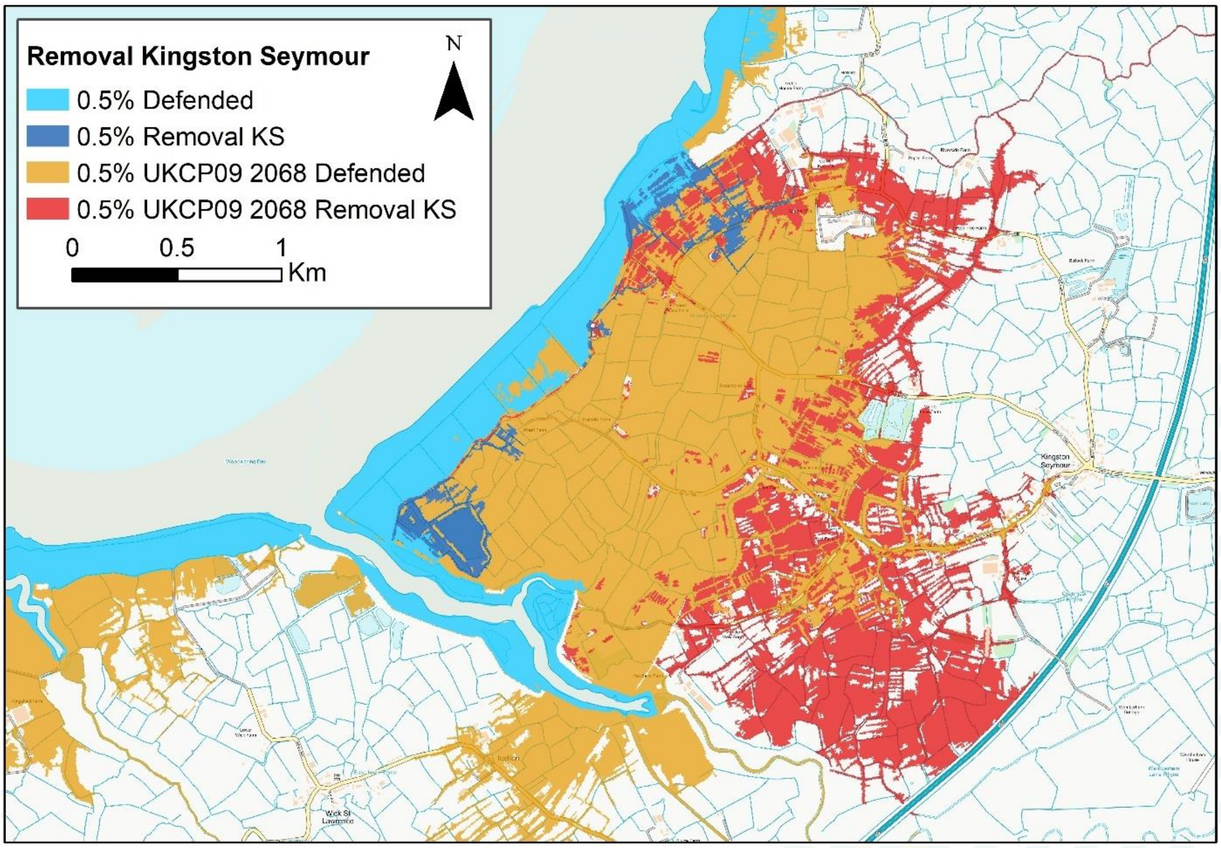

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Figure 14-1: Woodspring Bay Kingston Seymour Defence Removal scenario comparison with Defended

Table 14-1: Woodspring Bay property counts for the Kingston Seymour defence removal scenario

Scenario	AEP	Properties at flood risk				
		Commercial	Residential	Critical Infrastructure	Unclassified	Total
Defence removal Kingston Seymour	2%	34 (34)	6 (6)	1 (1)	16 (16)	57 (57)
	1.3%	38 (38)	6 (6)	1 (1)	17 (17)	62 (62)
	1%	38 (38)	6 (6)	1 (1)	18 (17)	63 (62)
	0.5%	49 (49)	8 (8)	5 (5)	27 (26)	89 (88)
	0.5% UKCP09 2068	67 (66)	95 (89)	10 (10)	104 (82)	276 (247)

*Numbers in brackets represent the defended property counts

15 Severn House Farm breach results summary

Four separate breach scenarios were modelled to investigate the impacts of a defence breach, and consider the importance of the existing flood defence network in the Severn House Farm study area for three AEP events (1.3%, 0.5% and 0.5% NPPF 2068). The results from the breach scenarios were compared to the baseline defended flood extents. Figure 15-1 through Figure 15-4 show the defended scenario flood extents overlain the breach scenario flood extents for the present-day 0.5% AEP event for breaches 1 to 4. The number of properties at flood risk in each of the breach scenarios, and change from the defended baseline in brackets, are shown in Table 15-1.

Breach 1 involves a defence breach of the earth embankment at Severn House Farm. In all three of the modelled AEP events, a breach of the Severn House Farm embankment results in greater flood extents than the baseline defended scenario, but the difference from the baseline is less significant under sea level rise conditions. During the smallest event (1.3% AEP), a failure of the defence results in widespread flooding of the fields behind the breached embankment and flood waters propagate across Nupdown Lane in the east, Severn lane in the north, and as far as Oldbury Naite in the south. During the 0.5% AEP event, flood waters reach as far south as Duckhole and inundate more of Shepperdine Road when compared to the defended baseline. Flood depths in the fields behind the breach reach 1.10m and lead to 166 additional properties being inundated when compared to the defended baseline. During the largest climate change event (0.5% AEP NPPF 2068), Breach 1 shows a marginally greater flood extent than the baseline defended but with much greater depths, for example Breach 1 has depths that are ~0.60m larger than the defended baseline at Severn House Farm.

Breach 2 involves a defence breach of the earth embankment at Whale Wharf. In all three of the modelled AEP events, a breach of the Whale Wharf embankment results in greater flood extents than the baseline defended scenario, but the difference from the baseline is less significant under sea level rise conditions. During the smallest event (1.3% AEP), a failure of the defence results in flood waters propagating into the adjacent fields behind the breach location surrounding Lower Farm Rhine in the east and propagating along the low topography towards Redhill Lane in the south. During the 0.5% AEP event, the maximum breach flood extent inundates several properties at the end of Lower Cowhill Lane, whereas during the defended baseline, flood water remains within Lower Farm Rhine. During the largest climate change event (0.5% AEP NPPF 2068), the breach flood waters extend beyond the M4 in the south, with depths of 0.30m greater at Whale Wharf than the defended baseline, and leading to an additional 28 properties being inundated.

Breach 3 involves a sluice gate failure of Oldbury Outfall tidal gate. In all three of the modelled AEP events, the defence breach results in greater flood extents than the baseline defended scenario, but the difference from the baseline is less significant under sea level rise conditions. During the smallest event (1.3% AEP), a failure of the sluice gate results in the inundation of several properties in Oldbury-on-Severn as flood water propagates along Cowhill Wharf Rhine, and then continues to propagate along Oldbury Naite Wharf to reach Oldbury Naite. During the 0.5% AEP event, flood waters continue to propagate from the gate failure location, beyond Oldbury Naite, and combine with the still water flooding from the north, leading to 84 more properties at flood risk compared to the baseline defended. During the largest climate change event (0.5% AEP NPPF 2068), the flood extent for the gate failure is similar to the defended baseline. Properties are inundated in particular in Oldbury-on-Severn and Oldbury Naite but only 11 additional properties are inundated when compared to the defended baseline.

Breach 4 involves a defence breach of the earth embankment at Hill Pill. In all three of the modelled AEP events, the defence breach results in greater flood extents than the baseline defended scenario, but the difference from the baseline is less significant under sea level rise conditions. During the smallest event (1.3% AEP), a breach of the defence results in widespread flooding of the area behind the breach location, inundating Shepperdine, Nupdown and north of Oldbury Naite. During the 0.5% AEP event, the breach flood waters propagate further into Oldbury Naite reaching depths of >0.60m. During the largest climate change event (0.5% AEP NPPF 2068), the flood extent for the defence breach is similar to the defended baseline, but with greater depths as larger volumes of water have contributed to the flood risk in the area.

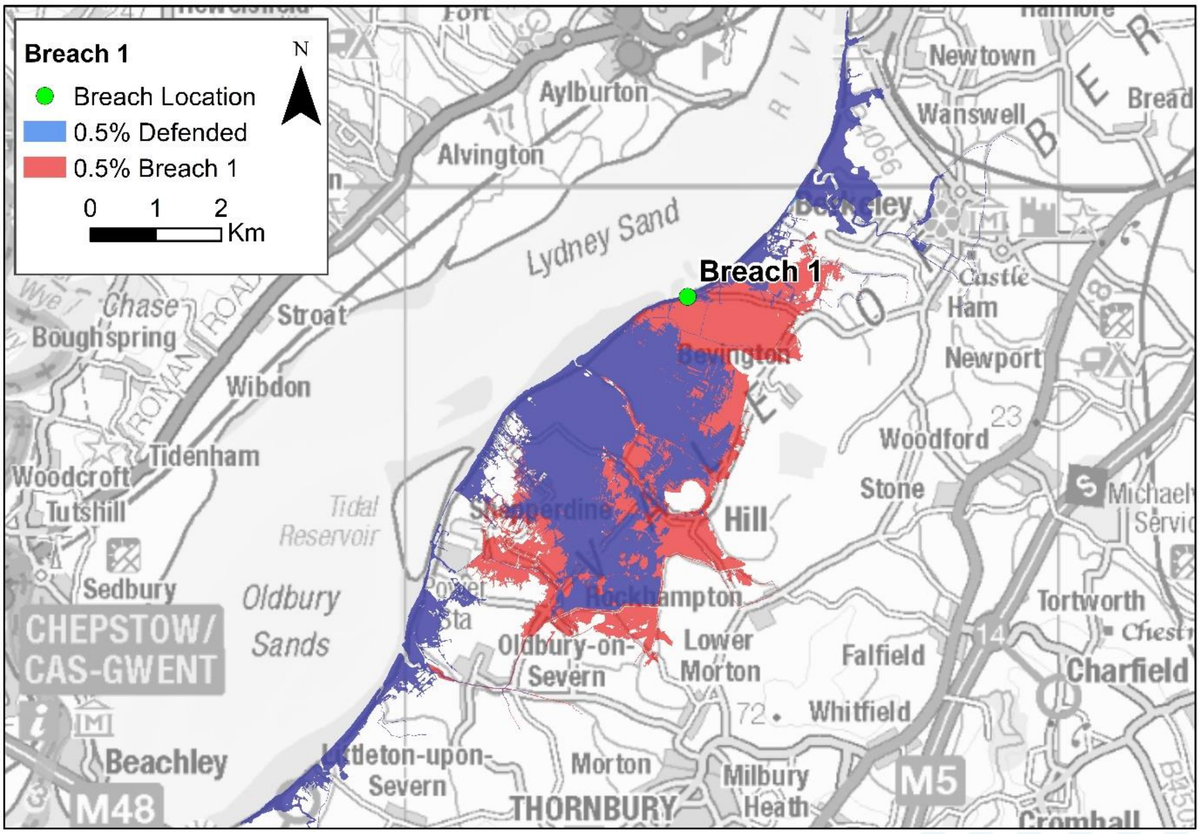

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Figure 15-1: Severn House Farm Breach 1 scenario compared to the baseline defended for the present-day 0.5% AEP event

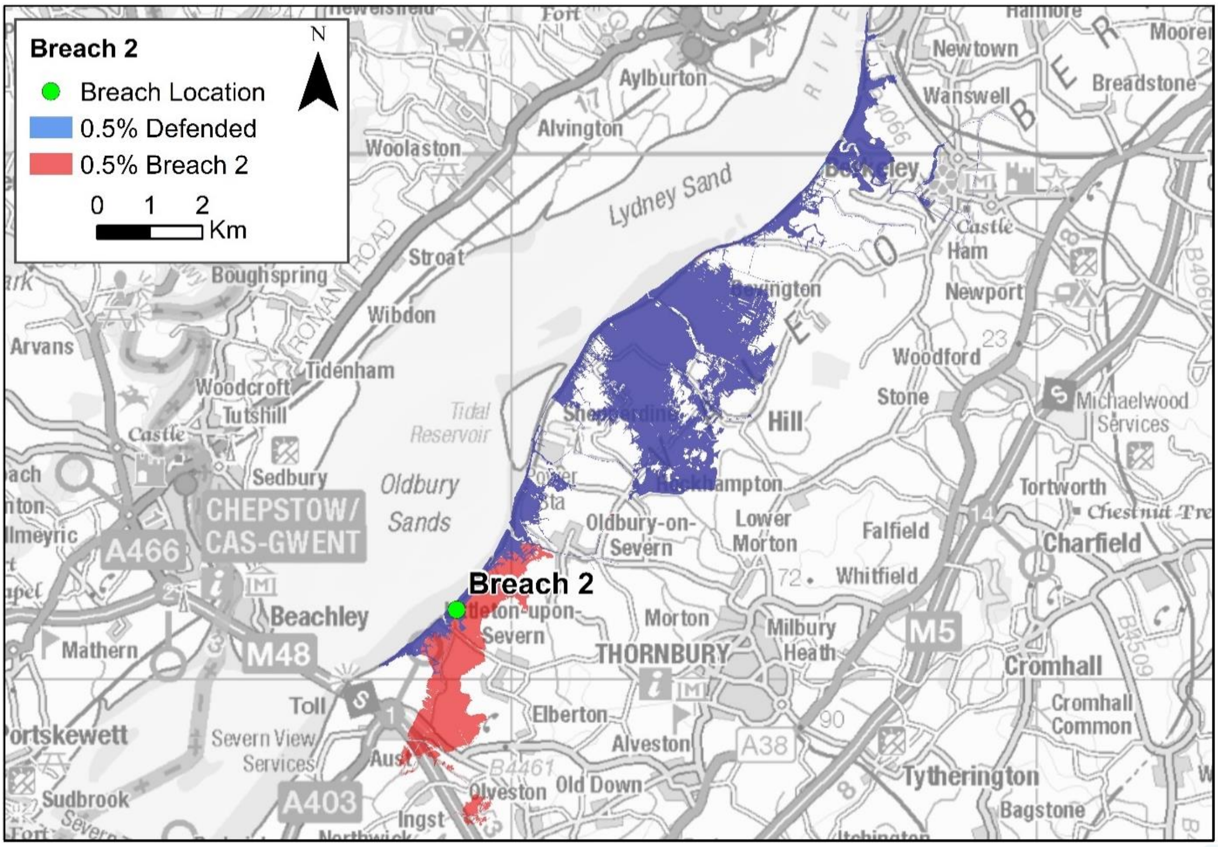

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Figure 15-2: Severn House Farm Breach 2 scenario compared to the baseline defended for the present-day 0.5% AEP event

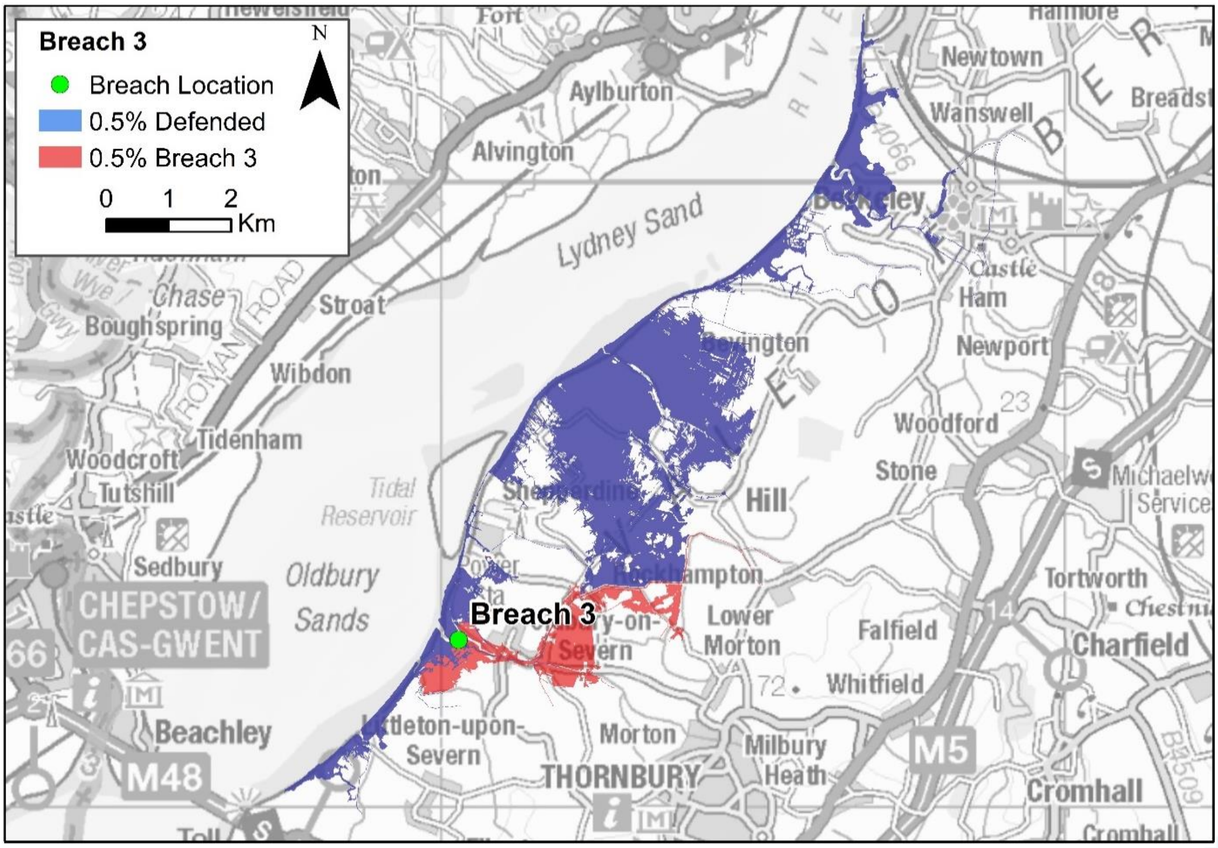

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Figure 15-3: Severn House Farm Breach 3 scenario compared to the baseline defended for the present-day 0.5% AEP event

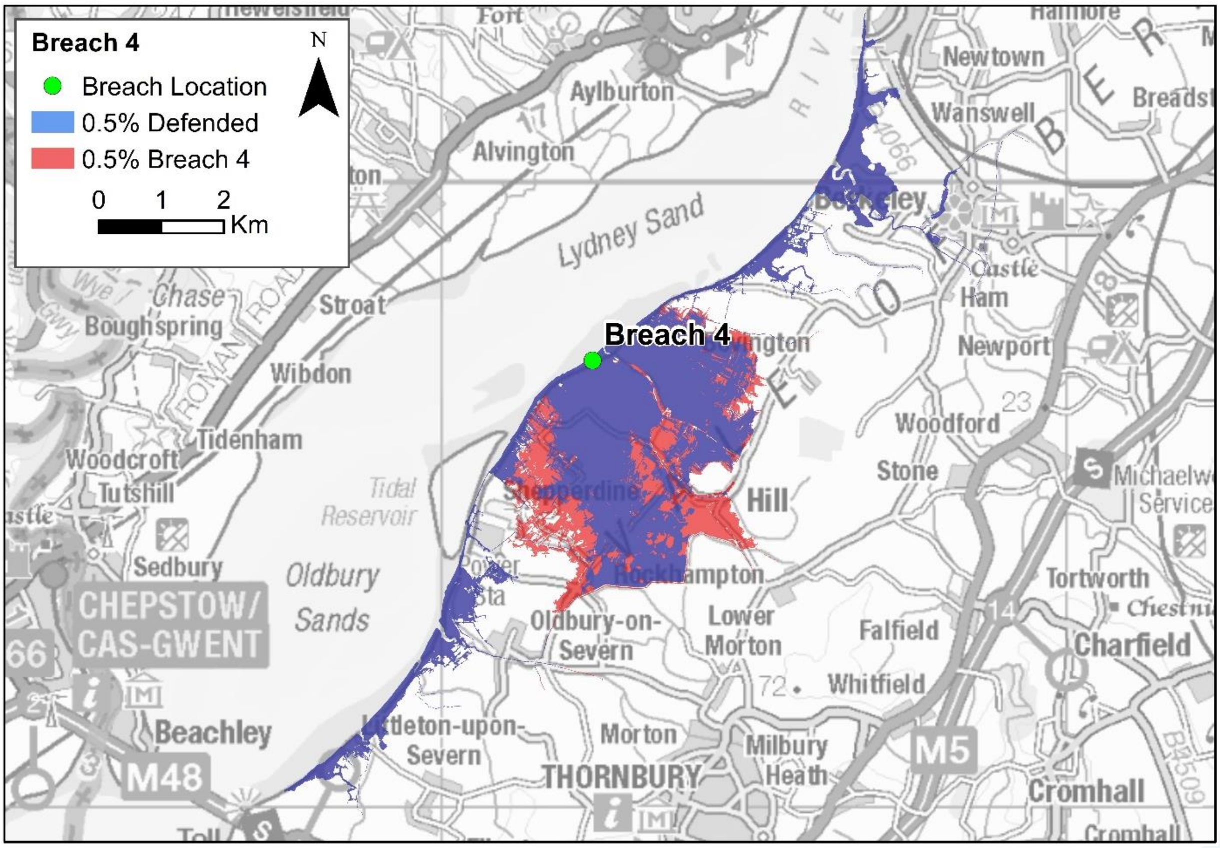

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Figure 15-4: Severn House Farm Breach 4 scenario compared to the baseline defended for the present-day 0.5% AEP event

Table 15-1: Severn House Farm property counts for the breach scenarios

Scenario	AEP	Properties at flood risk				
		Commercial	Residential	Critical Infrastructure	Unclassified	Total
Breach 1	1.3%	3 (+2)	23 (+23)	3 (+1)	78 (+70)	107 (+96)
	0.5%	5 (+3)	56 (+51)	4 (+1)	159 (+111)	224 (+166)
	0.5% NPPF 2068	35 (+1)	190 (+7)	10 (0)	625 (+25)	860 (+33)
Breach 2	1.3%	4 (+3)	2 (+2)	2 (0)	21 (+13)	29 (+18)
	0.5%	5 (+3)	8 (+3)	3 (0)	70 (+22)	86 (+28)
	0.5% NPPF 2068	34 (0)	194 (+11)	10 (0)	617 (+17)	855 (+28)
Breach 3	1.3%	6 (+5)	35 (+35)	5 (+3)	41 (+33)	87 (+76)
	0.5%	7 (+5)	42 (+37)	6 (+3)	87 (+39)	142 (+84)
	0.5% NPPF 2068	34 (0)	186 (+3)	10 (0)	608 (+8)	838 (+11)
Breach 4	1.3%	2 (+1)	9 (+9)	2 (0)	73 (+65)	86 (+75)
	0.5%	3 (+1)	38 (+33)	3 (0)	139 (+91)	183 (+125)
	0.5% NPPF 2068	34 (0)	185 (+2)	10 (0)	613 (+13)	842 (+15)

*Numbers in brackets represent change from the defended baseline

16 Property count approach

The National Receptor Dataset (NRD) was used to count properties inundated by counting the number of points that sit within the modelled flood extents for each scenario and AEP. Note; if the building footprint was used to count properties, rather than the NRD point as used in this project, the number of properties inundated would likely increase as it would include buildings where a small section of building is flooded but not the NRD point itself.

The NRD property dataset was modified before being used in the property counts to exclude properties that should not be counted. Excluded properties are based on National Flood Risk Assessment (NAFRA) exclusions and taken from Appendix D of the of Geomatics NRD2014 Reconciliation Report[16]. The property exclusions included things such as caravans classed as holiday parks with short term lets, telephone boxes, bandstands, playgrounds and public car parks.

Unclassified buildings and buildings awaiting classification were included as an unclassified count where they fall within a MasterMap building (have a TOPOFID) as detailed in the reconciliation report.

[16] Reconciliation Report. NRD2014. Geomatics. Environment Agency. 2015.

17 Project summary

JBA Consulting was commissioned to complete a numerical modelling study, as part of the Programme Delivery Unit (PDU) Modelling and Mapping Lot 1, to assess coastal flood risk along the north coast in the Bristol Channel.

Two existing flood inundation models were updated to cover the areas of interest:

- Woodspring Bay - Kewstoke to Clevedon (previously named Som3)
- Severn House Farm - Aust to Sharpness (previously named Som5)

The Bristol Channel north coastline is vulnerable to a range of coastal flood risk drivers including extreme still water flooding and wave overtopping.

Several modelling tools were used to understand flood risk as follows:

- Multivariate extreme value methods using the conditional approach of Heffernan & Tawn (2004) were used to assess the joint exceedance probability of different sea state combinations. Joint probability combinations of extreme wave, wind and sea level conditions were generated for a range of return periods for 8 sectors (240, 270, 300, 330, 0, 30, 60, 90) determined by wind direction.
- A 2D SWAN wave transformation model of the Severn Estuary was updated with new topographic data and recalibrated using wave data from the Wave Watch III point 573 (51.2969°N, -4.243°W) and water level data from the Class A Ilfracombe tide gauge. The model was used to transpose each joint probability offshore wave and water level combination into the nearshore.
- A wave overtopping model was used to provide mean overtopping discharges at defence sections along the coast. The EurOtop 2 ANN tool was fed with defence geometric profiles, and the transposed nearshore wave and water level joint probability combinations to provide mean overtopping discharges. The wave and water level combinations that led to the worst-case overtopping rate for each AEP was used in the design overtopping modelling.
- A 2D TUFLOW flood inundation model was used to generate flood risk outputs at the sites of interest. Two TUFLOW models were build and simulated using the latest 2018 Coastal Flood Boundary (CFB) extreme sea levels using the HPC solver.

The coastal modelling suite was used to map the flood risk for a range of design events and scenarios:

- Design scenarios included:
  - Defended
  - Undefended
  - Marsh erosion (Woodspring Bay model only)
  - Wick St Lawrence secondary defence removal (Woodspring Bay model only)
  - Kingston Seymour secondary defence removal (Woodspring Bay model only)
  - Defence breach scenarios (Severn House Farm model only)
- Present-day flood risk was modelled for the 10% 5%, 3.3%, 2%, 1.3%, 1%, 0.5% and 0.1% Annual Exceedance Probability (AEP) events. The Severn house Farm model was simulated for the 20% AEP event in addition.

- Climate change flood risk was modelled for the 0.5% AEP event based on sea-level rise guidance in United Kingdom Climate Projections 2009 (UKCP09) medium emission 95th percentile and National Planning and Policy Framework (NPPF) projected to the year 2068 and 2118. The sea level rise methodology used in assessments of future risk will depend on the purpose. The NPPF sea level rise estimates along this stretch of coast are generally higher than UKCP09 and should be used in planning decisions.

The model outputs were used to:

- Map coastal flood risk and produce a variety of outputs including gridded outputs for flood depth, level, velocity and hazard.
- Produce processed flood extents.
- Derive new Flood Map components for Flood Zone 3, Flood Zone 2 and identify the Areas Benefitting from Defences (ABDs).
- Create Flood Warning Areas and criteria/procedures for flood incident management.
- Generate incident management tools.
- Estimate the Standard of Protection (SoP) of coastal defences.

Appendices

A Defended defence schematisation

Note; Full detailed defence schematisation QA sheets are provided separately due to size as supporting documentation.

B Undefended defence schematisation

Note; Full detailed defence schematisation QA sheets are provided separately due to size as supporting documentation.

C Overtopping rates and nearshore wave conditions for each event at each toe

The overtopping rates and nearshore wave conditions for each event at each toe are provided separately due to size as supporting documentation.

D Wave and water level conditions used in sensitivity testing as baseline conditions (0.5% AEP, 2018)

Model	Inflow name	Wave height (m)	Wave period (s)	Obliquity (deg)	Water level (mAOD)
WSB	WO_1	0.818	5.004	32.821	8.276
	WO_2	0.590	4.768	11.469	8.293
	WO_3	0.609	4.576	4.623	8.307
	WO_4	0.694	4.521	6.754	8.307
	WO_5	0.664	4.525	5.285	8.311
	WO_6	0.599	4.486	5.513	8.313
	WO_7	0.337	3.296	3.273	8.382
	WO_8	0.599	4.375	11.972	8.323
	WO_9	0.436	4.177	1.944	8.323
	WO_10	0.027	1.860	26.970	8.265
	WO_11	0.178	2.241	30.606	8.459
	WO_12	0.576	4.240	26.567	8.011
	WO_13	0.466	3.115	26.704	8.442
	WO_14	1.326	4.989	16.795	7.839
	WO_15	0.745	4.174	25.370	8.492
	WO_16	0.545	3.968	37.523	8.492
	WO_17	0.392	4.348	3.284	8.503
	WO_18	0.612	4.102	0.726	8.512
	WO_19	0.930	4.987	6.049	8.192
	WO_20	0.767	4.099	13.032	8.532
	WO_21	0.966	5.004	10.878	8.221
	WO_22	0.904	5.114	0.000	7.930
	WO_23	1.283	5.107	0.553	7.930
SHF	WO_1	0.341	2.455	43.627	9.372
	WO_2	0.207	1.856	45.000	9.534
	WO_3	0.718	3.320	0.000	9.136
	WO_4	0.000	0.000	0.000	0.000

Model	Inflow name	Wave height (m)	Wave period (s)	Obliquity (deg)	Water level (mAOD)
	WO_5	0.000	0.000	0.000	0.000
	WO_6	0.000	0.000	0.000	0.000
	WO_7	0.375	2.535	45.000	9.564
	WO_8	0.342	2.368	45.000	9.610
	WO_9	0.103	1.436	45.000	9.675
	WO_10	0.288	2.025	45.000	9.703
	WO_11	0.273	2.142	0.000	9.714
	WO_12	0.213	1.752	24.758	9.734
	WO_13	0.310	2.111	45.000	9.805
	WO_14	0.113	1.425	45.000	9.914
	WO_15	0.328	2.325	15.096	9.918
	WO_16	0.166	1.598	35.414	10.005

E Sensitivity testing of the overtopping results (0.5% AEP, 2018) in m3/s/m

Woodspring Bay percentage difference from base rates

Inflow name	Base rates	Wave Height +10%	Wave Height - 10%	Wave period +1s	Wave period - 1s	Crest Freeboard +0.1m	Crest Freeboard - 0.1m	Angle of lower slope +5%	Angle of lower slope +10%	Angle of upper slope +5%	Angle of upper slope +10%	Armour Freeboard +0.1m	Armour Freeboard - 0.1m
WO_1	0.0001	-79	-85	-82	-83	-82	-83	-79	-74	-83	-84	-84	-82
WO_2	0.0000	0	0	0	0	0	0	0	0	0	0	0	0
WO_3	0.0000	-94	-97	-92	-95	-95	-95	-94	-92	-95	-95	-95	-94
WO_4	0.0000	-94	-94	-91	-92	-94	-94	-93	-91	-94	-94	-94	-93
WO_5	0.0000	-71	-83	-88	1	-77	-77	-74	-72	-77	-77	-78	-75
WO_6	0.0049	-100	-100	-99	-100	-100	-99	-100	-100	-100	-100	-100	-99
WO_7	0.0001	-70	-71	-59	247	-72	-72	-67	-61	-74	-75	-71	-72
WO_8	0.0034	-88	-90	-83	-95	-90	-90	-86	-80	-90	-90	-90	-89
WO_9	0.0398	-99	-100	-99	-100	-99	-99	-99	-99	-100	-100	-100	-99
WO_10	0.3014	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100
WO_11	0.2241	-100	-100	-100	-99	-100	-100	-100	-100	-100	-100	-100	-100
WO_12	0.0003	-96	-98	-94	-100	-98	-98	-98	-98	-98	-98	-98	-97
WO_13	0.0018	-100	-99	-98	-98	-99	-99	-99	-99	-100	-100	-99	-99
WO_14	0.0078	-90	-98	-85	-99	-95	-95	-96	-96	-95	-95	-97	-93
WO_15	0.0099	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100
WO_16	0.0001	-99	-97	-96	-99	-98	-98	-98	-98	-98	-98	-98	-98
WO_17	0.0000	0	0	0	0	0	0	0	0	0	0	0	0
WO_18	0.0017	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100
WO_19	0.0081	-99	-100	-99	-100	-100	-100	-100	-100	-100	-100	-100	-100
WO_20	0.0000	-92	-88	-84	-94	-91	-91	-89	-87	-91	-90	-91	-90
WO_21	0.0000	0	0	0	0	0	0	0	0	0	0	0	0
WO_22	0.0001	-8	-23	0	-42	-18	-18	-18	-18	-18	-18	-15	-21
WO_23	0.0002	-97	-99	-95	-99	-98	-98	-98	-99	-98	-98	-99	-98

Severn House Farm percentage difference from base rates

Inflow name	Base rates	Wave Height +10%	Wave Height - 10%	Wave period +1s	Wave period - 1s	Crest Freeboard +0.1m	Crest Freeboard - 0.1m	Angle of lower slope +5%	Angle of lower slope +10%	Angle of upper slope +5%	Angle of upper slope +10%	Armour Freeboard +0.1m	Armour Freeboard - 0.1m
WO_1	0.0069	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100
WO_2	0.0340	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100
WO_3	0.0126	-99	-100	-100	-99	-99	-99	-99	-100	-99	-99	-100	-99
WO_4	0.0000	0	0	0	0	0	0	0	0	0	0	0	0
WO_5	0.0000	0	0	0	0	0	0	0	0	0	0	0	0
WO_6	0.0000	0	0	0	0	0	0	0	0	0	0	0	0
WO_7	0.0049	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100
WO_8	0.0029	-100	-100	-99	-99	-100	-100	-100	-100	-100	-100	-100	-100
WO_9	0.2460	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100
WO_10	0.0043	-100	-100	-99	-99	-100	-100	-100	-100	-100	-100	-100	-100
WO_11	0.0048	-100	-100	-99	-98	-100	-100	-100	-100	-100	-100	-100	-100
WO_12	0.0025	-100	-99	-99	-96	-100	-100	-100	-100	-100	-100	-100	-100
WO_13	0.0078	-100	-100	-100	-98	-100	-100	-100	-100	-100	-100	-100	-100
WO_14	0.0660	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100
WO_15	0.0005	-100	-99	-99	-99	-99	-100	-100	-99	-100	-100	-99	-100
WO_16	0.1357	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100

F Design simulation model stability summary

Woodspring Bay model stability – Defended and Undefended scenarios

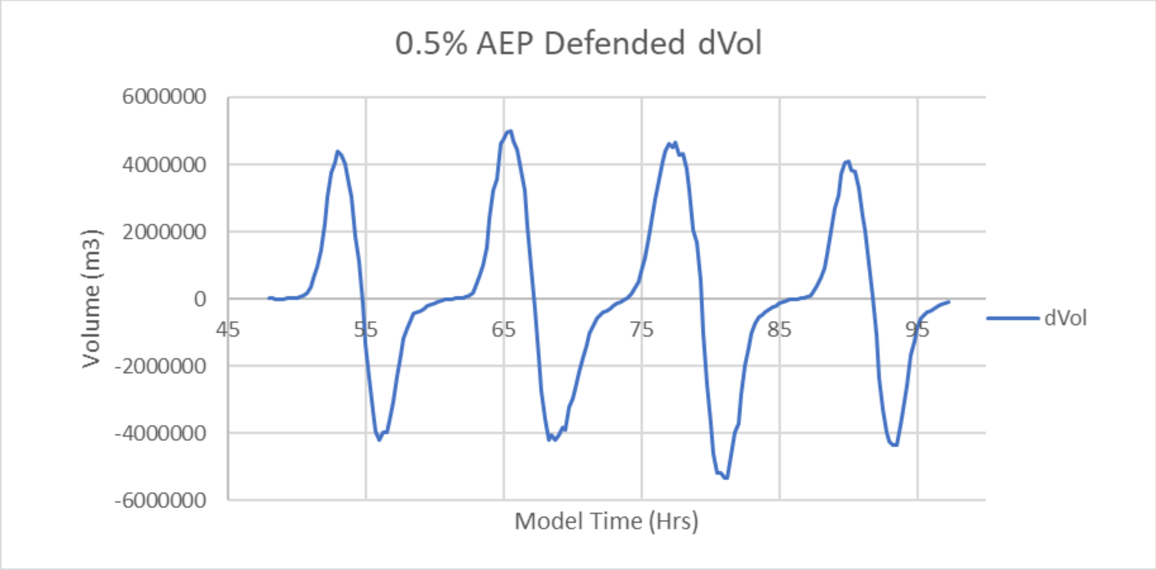
A series of good modelling practice model checks were undertaken. These included:

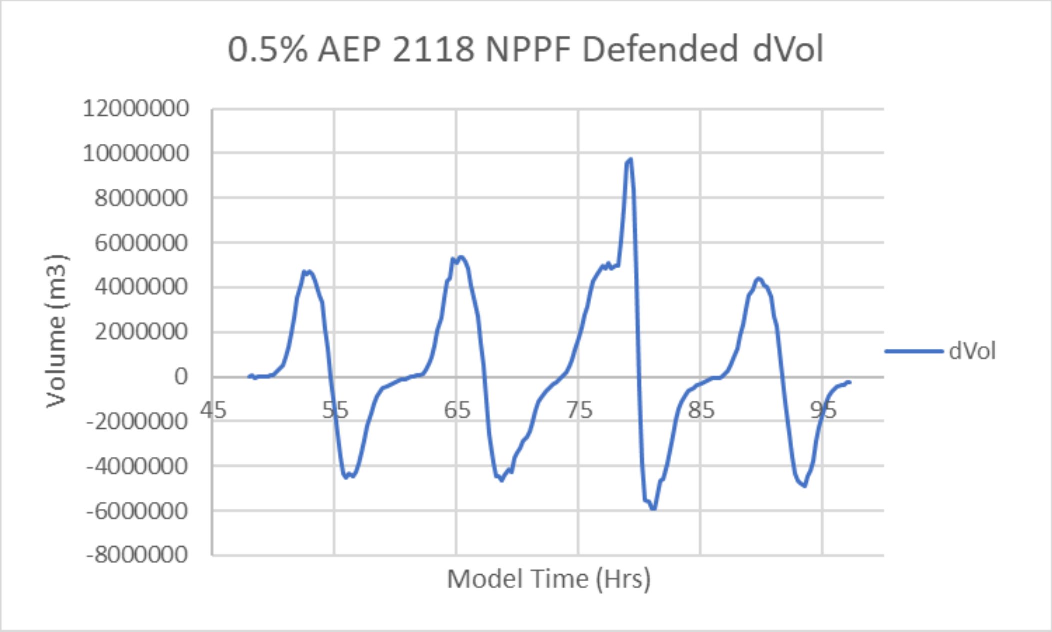
- Checking maximum grids (water level, depth, velocity and hazard) for spikes that would suggest model instability
- Assessing flow through 1D culvert units for erratic spikes and unusual flow patterns
- Assessment of time-series output points (PO points) within the 2D model domain, primarily for water levels and flows for unusual spikes suggesting instability
- Visual assessment of flow paths by plotting animations

Other stability checks and model health checks are discussed in more detail below.

dVol

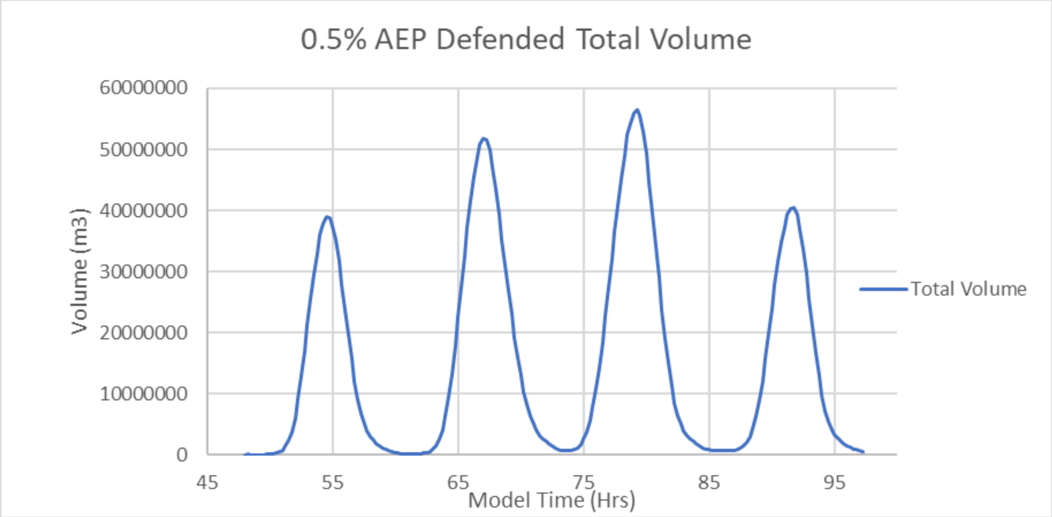
The TUFLOW change in total volume of water within the 2D domain (dVol) plots for the present-day defended 50% to 0.5% AEP events are as expected. These show a plot that represents the design tidal cycle flux of the flood and ebb tides. The 0.5% AEP dVol plot is graphically shown below. The graphs show a smooth transition between high and low tide with some small fluctuations in between which is expected in a Coastal 2D model. In the largest defended event (0.5% AEP NPPF 2118), there is a spike in the dVol at high tide. This is due to a large volume of overtopping being applied in the model during this event.





Total Volume

The total volume of water within the 2D domain for the 0.5% AEP event is shown graphically below. For all modelled events and scenarios, the total volume plots are as expected based on the flood and ebb tidal flux and small overtopping discharges.



Mass Error

Though HPC is mass conserving it is still important to review Mass Error, as it can still occur when coupling HPC with 1D elements in either the 1D/2D linking, such as for the culverts included in the model. A 'healthy' model will usually report up to ±1% mass error.

Mass error for all model simulations is 0.00/-0.00, as detailed in the Table F-1 and F-2 for defended and undefended model scenarios.

HPC design time steps

HPC remains stable by reducing its timestep. Due to the underlying solution scheme, HPC typically uses a smaller timestep than Classic. A graph of the target timestep calculated from the model stability criteria is shown below. Typically, a low timestep is classed as 1/10 of the specified 2D timestep, in this case this would be 1s/10 = 0.1s. A repeated timestep occurs when 1 of 3 controls numbers are exceeded by more than 20%. These control numbers are:

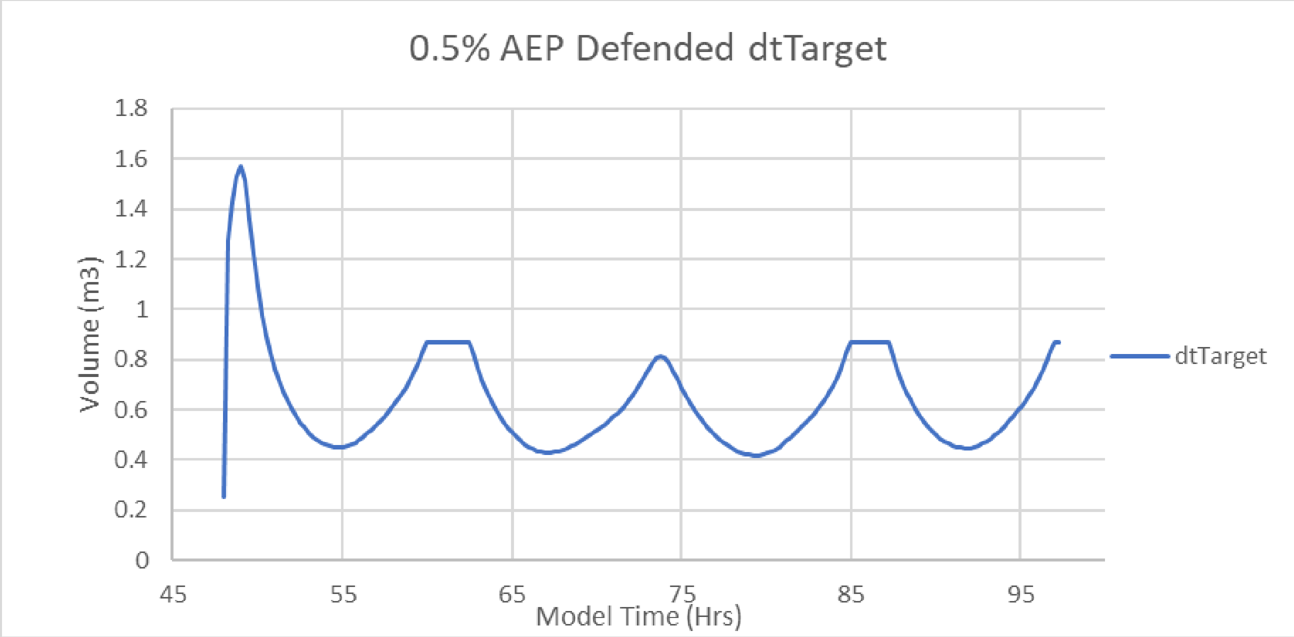
- *Courant Number (Nu)*
- *The Shallow Wave Celerity Number (Nc)*
- *Diffusion Number (Nd)*

Repeated timesteps during each simulation are detailed in Table F-1 and F-2 for defended and undefended model scenarios. Checks show that the number of repeated timesteps is generally very low and tend to occur as volume first enters the model as the shock of cells wetting up causes repeated timesteps, or around the ebb tide trough as flow passes out the model. This suggests the number of repeated timesteps is not an issue and that model health is generally good. The 0.5% AEP 2068 NPPF Undefended event has a noticeably higher number of repeating timesteps, but these occur on the low tide.

All other checks made for this simulation suggest the model is suitably stable.

Should a timestep need to be repeated more than ten times consecutively, the solution stops. The simulation will also stop if the default minimum permissible timestep of 0.1 seconds has been reached. Neither of these two conditions occur.

*Note that coastal models generally deal with much greater depths that with that of fluvial models, and therefore lower timesteps would be expected from that of fluvial models, and greater limitations by the Nu and Nc control numbers.



Simulation warnings and checks

There are multiple checks and warnings prior to simulation; they all relate to a message to say that a 2D cell was lowered to match the 1D structure invert level (based on the z flag). This is as expected.

A single warning is given during the simulation "WARNING 0255 - One or more GIS layers not closed during simulation - please notify support@tuflow.com.". TUFLOW support informs us this warning does not impact model results.

Table F-1: TUFLOW defended model log summary for Woodspring Bay

AEP	Warning messages prior to simulation	Check messages prior to simulation	Warning messages during simulation	Check messages during simulation	Model run time – CPU time (hr)	Cumulative Mass Error (%)	NaN repeated timesteps	HCH repeated timesteps
10%	31	36	1	5	5.98	0.00	0	0
5%	31	36	1	5	5.99	0.00	0	1
3.3%	31	36	1	5	6.00	0.00	0	1
2%	31	36	1	5	6.04	0.00	0	0
1.3%	31	36	1	5	6.05	0.00	0	0
1%	31	36	1	5	6.02	0.00	0	0

0.5%	31	36	1	5	6.05	0.00	0	0
0.1%	31	36	1	5	6.26	0.00	0	0
0.5%+CC NPPF 2068	31	36	1	5	6.22	0.00	0	0
0.5%+CC NPPF 2118	31	36	1	5	7.86	0.00	0	2
0.5%+CC UKCP09 2068	31	36	1	5	6.03	0.00	0	0
0.5%+CC UKCP09 2118	31	36	1	5	6.83	0.00	0	1

Table F-2: TUFLOW undefended model log summary for Woodspring Bay

AEP	Warning messages prior to simulation	Check messages prior to simulation	Warning messages during simulation	Check messages during simulation	Model run time - CPU time (hr)	Cumulative Mass Error (%)	NaN repeated timesteps	HCH repeated timesteps
10%	22	26	1	5	11.08	0.00	0	2
5%	22	26	1	5	11.34	0.00	0	2
3.3%	22	26	1	5	10.88	0.00	0	1
2%	22	26	1	5	11.07	0.00	0	2
1.3%	22	26	1	5	11.62	0.00	0	2
1%	22	26	1	5	11.49	0.00	0	2
0.5%	22	26	1	5	13.78	-0.00	0	1
0.1%	22	26	1	5	12.44	-0.00	0	4
0.5%+CC NPPF 2068	22	26	1	5	12.97	-0.00	0	10
0.5%+CC NPPF 2118	22	26	1	5	13.56	-0.00	0	0
0.5%+CC UKCP09 2068	22	26	1	5	12.45	-0.00	0	7
0.5%+CC UKCP09 2118	22	26	1	5	12.99	-0.00	0	0

Severn House Farm model stability – Defended and Undefended scenarios

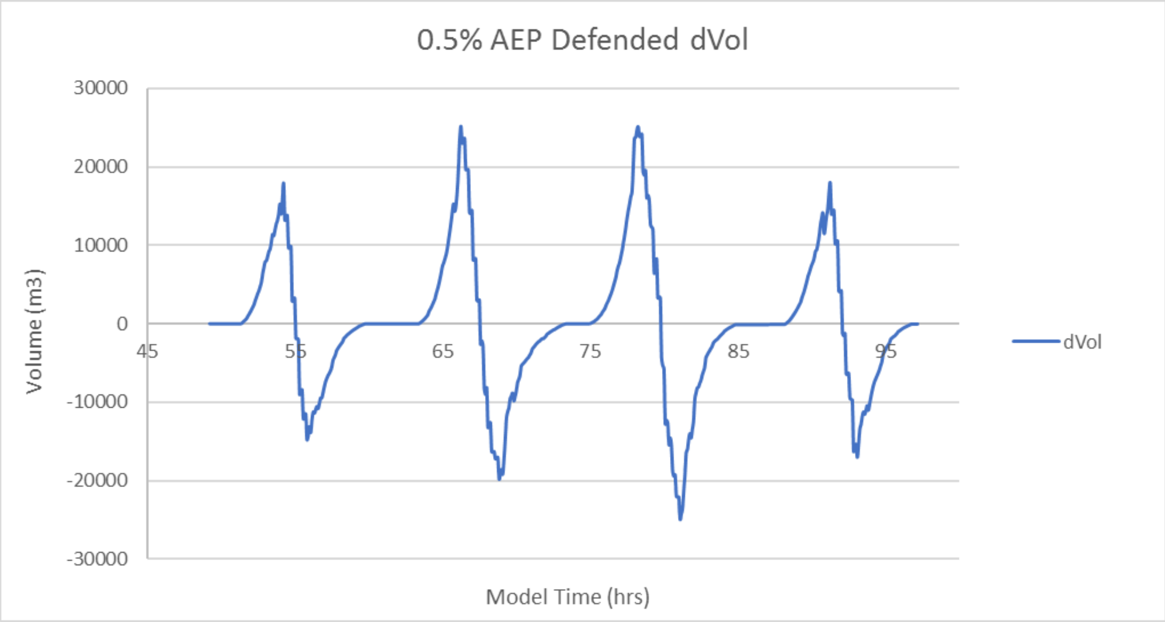
A series of good modelling practice model checks were undertaken. These included:

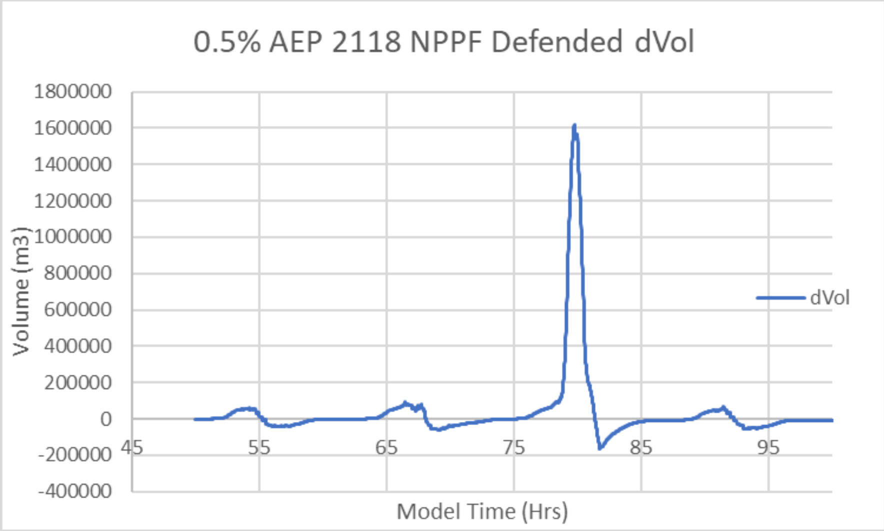
- Checking maximum grids (water level, depth, velocity and hazard) for spikes that would suggest model instability
- Assessing flow through 1D culvert units for erratic spikes and unusual flow patterns
- Assessment of time-series output points (PO points) within the 2D model domain, primarily for water levels and flows for unusual spikes suggesting instability
- Visual assessment of flow paths by plotting animations

Other stability checks and model health checks are discussed in more detail below.

dVol

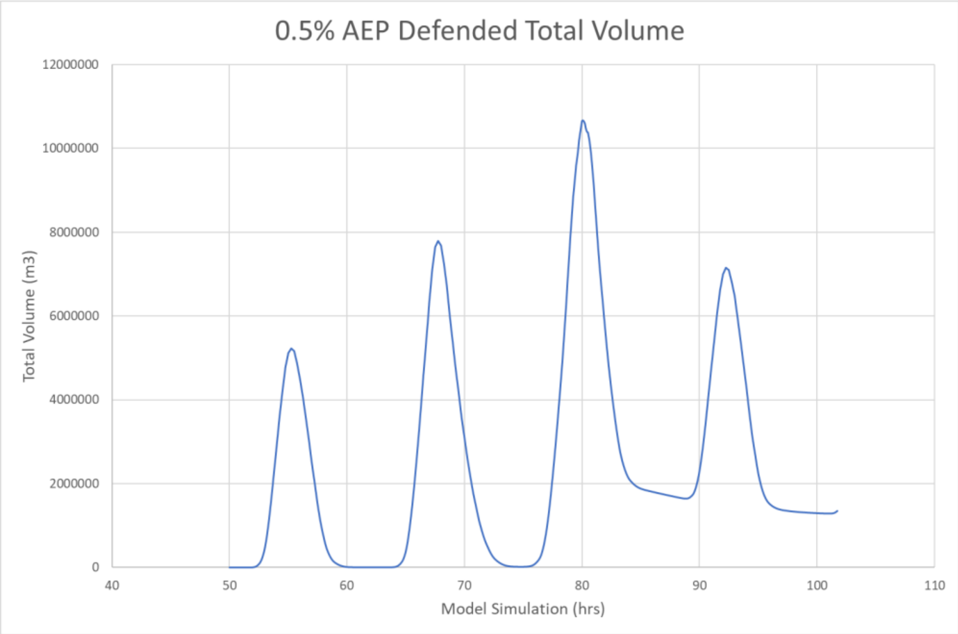
The TUFLOW change in total volume of water within the 2D domain (dVol) plots for the present-day and Climate Change defended and undefended events are as expected. These show a plot that represents the design tidal cycle flux of the flood and ebb tides. Two example dVol plots are shown below for the defended 0.5% and 0.5% AEP NPPF 2118 events. The graphs show a smooth transition between high and low tide with some small fluctuations in between which is expected in a Coastal 2D model. During the 0.5% AEP NPPF 2118 defended event, there is a large influx of water into the model attributed to the large volumes of overtopping which causes the spike in dVol.





Total Volume

The total volume of water within the 2D domain for the 0.5% AEP event is shown graphically below. For all modelled events and scenarios, the total volume plots are as expected based on the flood and ebb tidal flux and small overtopping discharges.



Mass Error

Though HPC is mass conserving it is still important to review Mass Error, as it can still occur when coupling HPC with 1D elements in either the 1D/2D linking, such as for the culverts included in the model. A 'healthy' model will usually report up to ±1% mass error.

Mass error for all model simulations is 0.00 to 0.01, as detailed in the Table G-1 and Table G-2 for defended and undefended model scenarios.

Mass error does spike at the start of the simulation, this is however expected due to the large tidal range in the model at start up. The initial spike soon disperses as the tide starts to move in and out of the model and by the peak tide the mass balance has settled down.

HPC design time steps

HPC remains stable by reducing its timestep. Due to the underlying solution scheme, HPC typically uses a smaller timestep than Classic. A graph of the target timestep calculated from the model stability criteria is shown below. Typically, a low timestep is classed as 1/10 of the specified 2D timestep, in this case this would be 1s/10 = 0.1s. A repeated timestep occurs when 1 of 3 controls numbers are exceeded by more than 20%. These control numbers are:

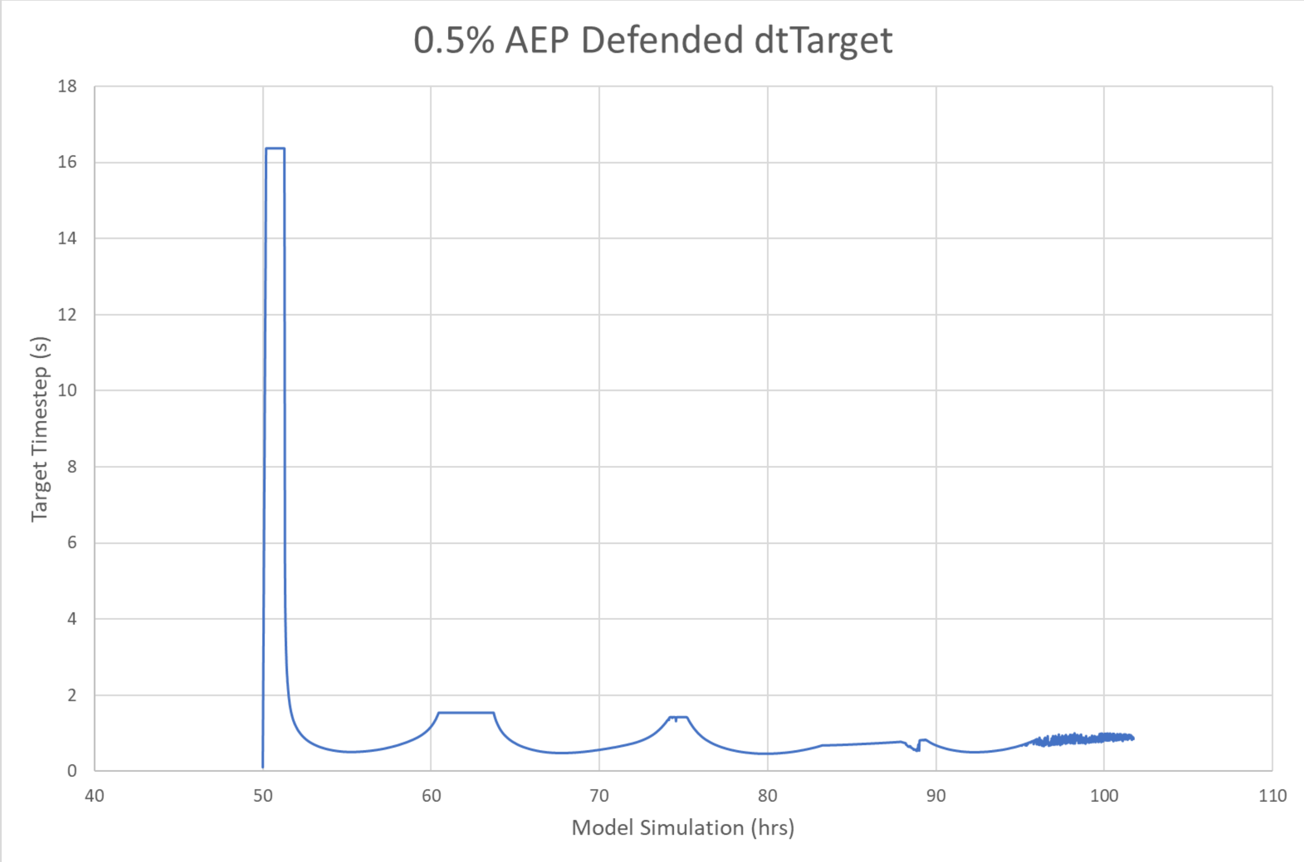
- *Courant Number (Nu)*
- *The Shallow Wave Celerity Number (Nc)*
- *Diffusion Number (Nd)*

Repeated timesteps during each simulation are detailed in the tables below for defended and undefended model scenarios. Checks show that the number of repeated timesteps is generally very low and tend to occur as volume first enters the model as the shock of cells wetting up causes repeated timesteps, or around the ebb tide trough as flow passes out the model. This suggests the number of repeated timesteps is not an issue and that model health is generally good. The 20% and 10% AEP Undefended present day event has a noticeably higher number of repeating timesteps; these are being limited by The Shallow Wave Celerity Number (Nc). A check on these show they occur between 73 and 75 hours in the model simulations, essentially low water, so should not impact on the maximum modelled results.

All other checks made for these simulations suggest the model is suitably stable.

Should a timestep need to be repeated more than ten times consecutively, the solution stops. The simulation will also stop if the default minimum permissible timestep of 0.1 seconds has been reached. Neither of these two conditions occur.

*Note that coastal models generally deal with much greater depths that with that of fluvial models, and therefore lower timesteps would be expected from that of fluvial models, and greater limitations by the Nu and Nc control numbers.



Simulation warnings and checks

A single warning is given during the simulation "WARNING 0255 - One or more GIS layers not closed during simulation - please notify support@tuflow.com.". TUFLOW support informs us this warning does not impact model results.

There are multiple checks prior to simulation; they all relate to a message to say that a 2D cell was lowered to match the 1D structure invert level (based on the z flag). This is as expected.

Table G-1: TUFLOW defended model log summary for Severn House Farm

AEP	Warning messages prior to simulation	Check messages prior to simulation	Warning messages during simulation	Check messages during simulation	Model run time – CPU time (hr)	Cumulative Mass Error (%)	NaN repeated timesteps	HCH repeated timesteps
20%	6	40	1	6	3.19	0.00	0	12
10%	6	40	1	6	3.03	0.00	0	7
5%	6	40	1	6	3.05	0.00	0	2
3.3%	6	40	1	6	3.06	0.00	0	2
2%	6	40	1	6	3.00	0.00	0	2
1.3%	6	40	1	6	3.08	0.00	0	2
1%	6	40	1	6	3.23	0.00	0	12
0.5%	6	40	1	6	4.97	0.01	0	11
0.1%	6	40	1	6	4.19	0.01	0	3
0.5%+CC UKCP09 2068	6	40	1	6	4.24	0.01	0	1
0.5%+CC UKCP09 2118	6	40	1	6	6.52	0.00	0	14
0.5%+CC NPPF 2068	6	40	1	6	3.92	0.00	0	0
0.5%+CC NPPF 2118	6	40	1	6	4.48	0.01	0	0

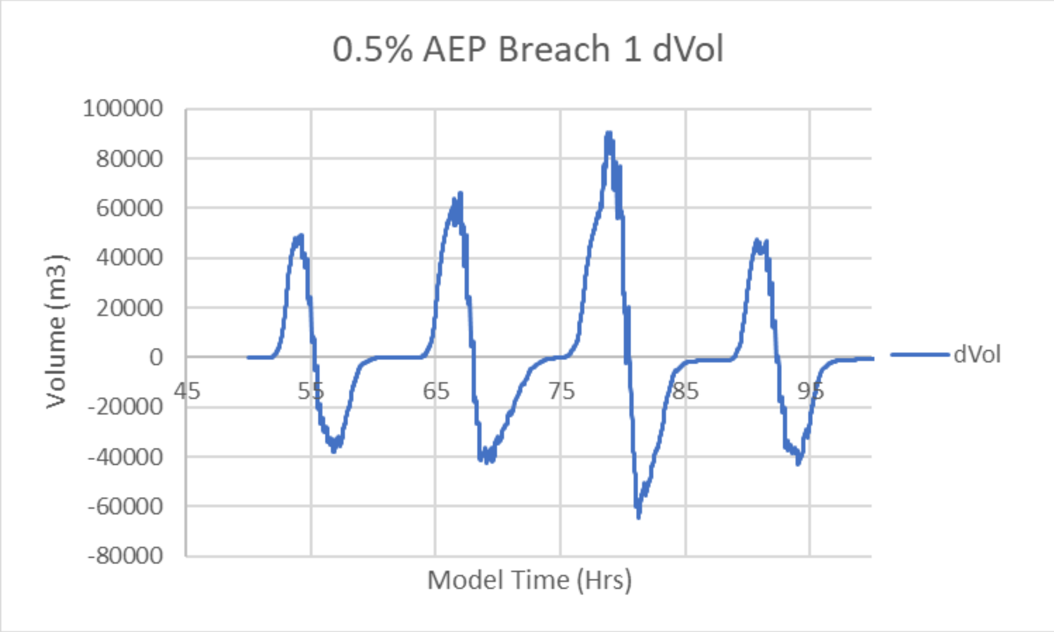
Table G-2: TUFLOW undefended model log summary for Severn House Farm

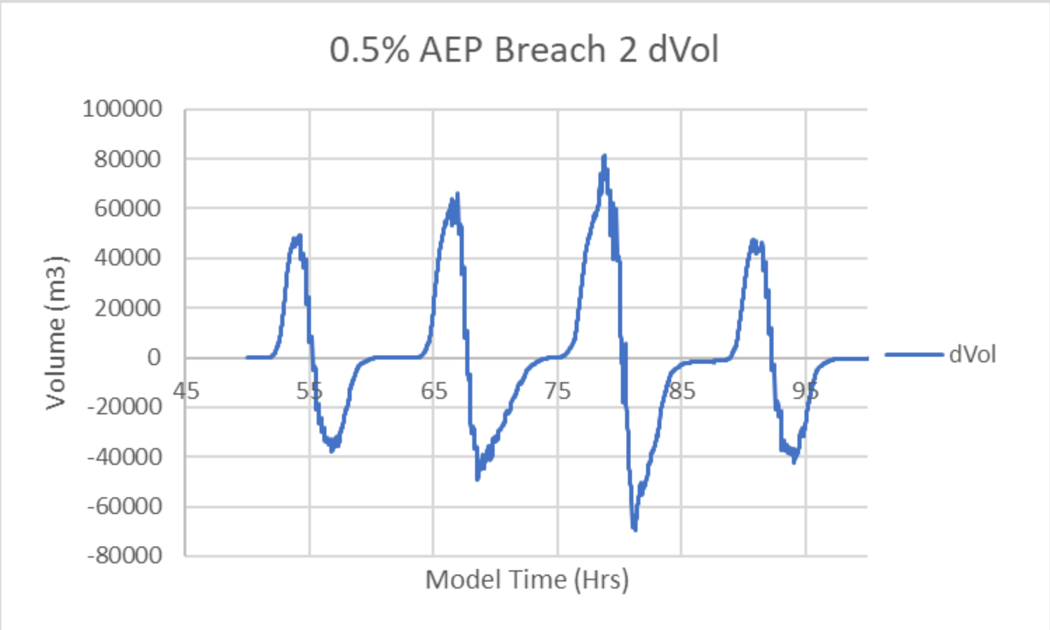
AEP	Warning messages prior to simulation	Check messages prior to simulation	Warning messages during simulation	Check messages during simulation	Model run time - CPU time (hr)	Cumulative Mass Error (%)	NaN repeated timesteps	HCH repeated timesteps
20%	5	26	1	6	4.24	0.01	0	97
10%	5	26	1	6	4.38	0.01	0	85
5%	5	26	1	6	4.33	0.01	0	1
3.3%	5	26	1	6	4.27	0.01	0	2
2%	5	26	1	6	4.21	0.00	0	0
1.3%	5	26	1	6	4.46	0.00	0	0
1%	5	26	1	6	4.48	0.00	0	0

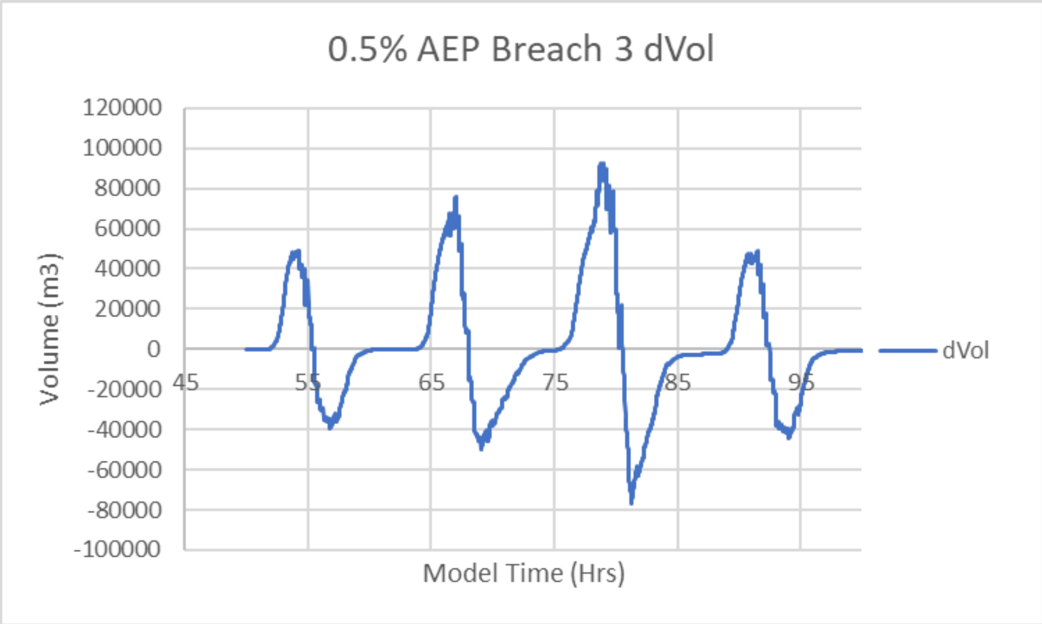
	0.5%	5	26	1	6	5.63	0.01	0	3
	0.1%	5	26	1	6	4.90	0.01	0	1
	0.5%+CC NPPF 2068	5	26	1	6	5.16	0.01	0	0
	0.5%+CC NPPF 2118	5	26	1	6	5.60	0.00	0	0
	0.5%+CC UKCP09 2068	5	26	1	6	5.20	0.01	0	50
	0.5%+CC UKCP09 2118	5	26	1	6	5.01	0.00	0	1

H Design simulation breach model stability summary

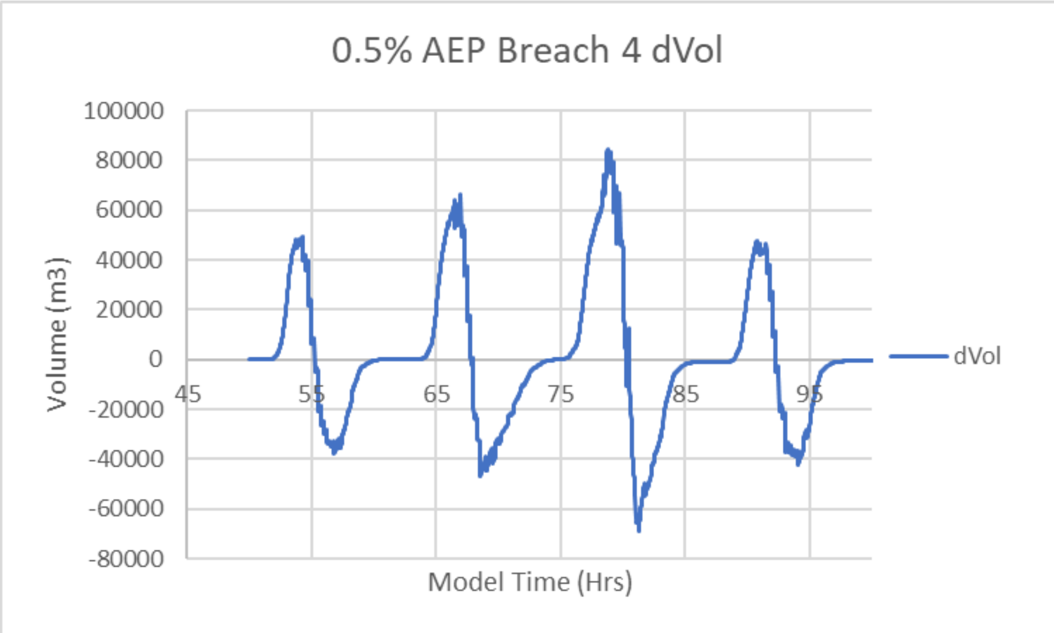
Severn House Farm model stability – Breach scenarios
A series of good modelling practice model checks were undertaken. These included: • Checking maximum grids (water level, depth, velocity and hazard) for spikes that would suggest model instability • Assessing flow through 1D culvert units for erratic spikes and unusual flow patterns • Assessment of time-series output points (PO points) within the 2D model domain, primarily for water levels and flows for unusual spikes suggesting instability • Visual assessment of flow paths by plotting animations Other stability checks and model health checks are discussed in more detail below. **dVol** The TUFLOW change in total volume of water within the 2D domain (dVol) plots for the breach scenarios are as expected. These show a plot that represents the design tidal cycle flux of the flood and ebb tides. Example dVol plots are shown below which show the present day 0.5% AEP event for each of the 4 breach scenarios. The graphs show a smooth transition between high and low tide with some small fluctuations in between which is expected in a Coastal 2D model.


0.5% AEP Breach 1 dVol
Volume (m3)
100000
80000
60000
40000
20000
0
-20000
-40000
-60000
-80000
45
55
65
75
85
95
Model Time (Hrs)
dVol


0.5% AEP Breach 2 dVol
Volume (m3)
100000
80000
60000
40000
20000
0
-20000
-40000
-60000
-80000
45
55
65
75
85
95
Model Time (Hrs)
dVol



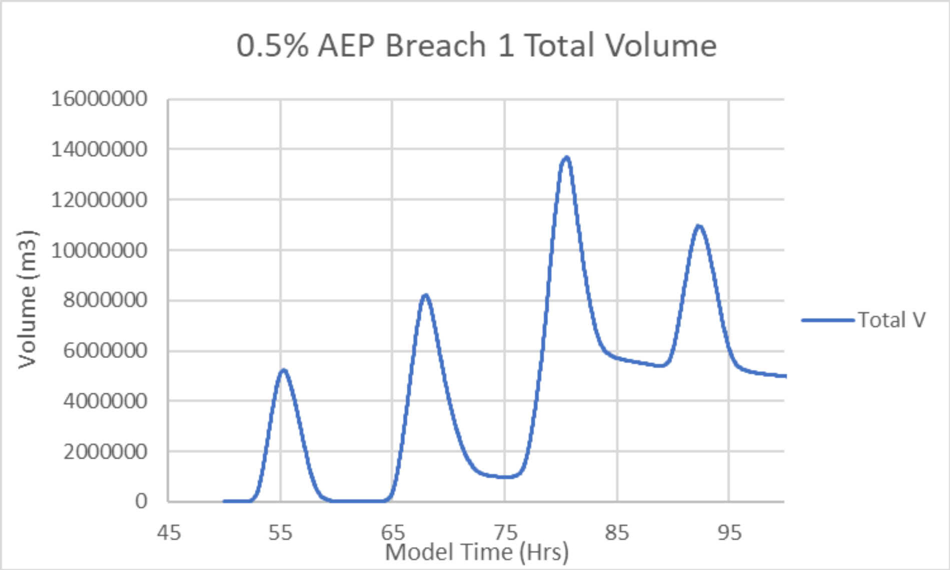
0.5% AEP Breach 3 dVol
120000
100000
80000
60000
40000
20000
0
-20000
-40000
-60000
-80000
-100000
45
55
65
75
85
95
Volume (m3)
Model Time (Hrs)
dVol

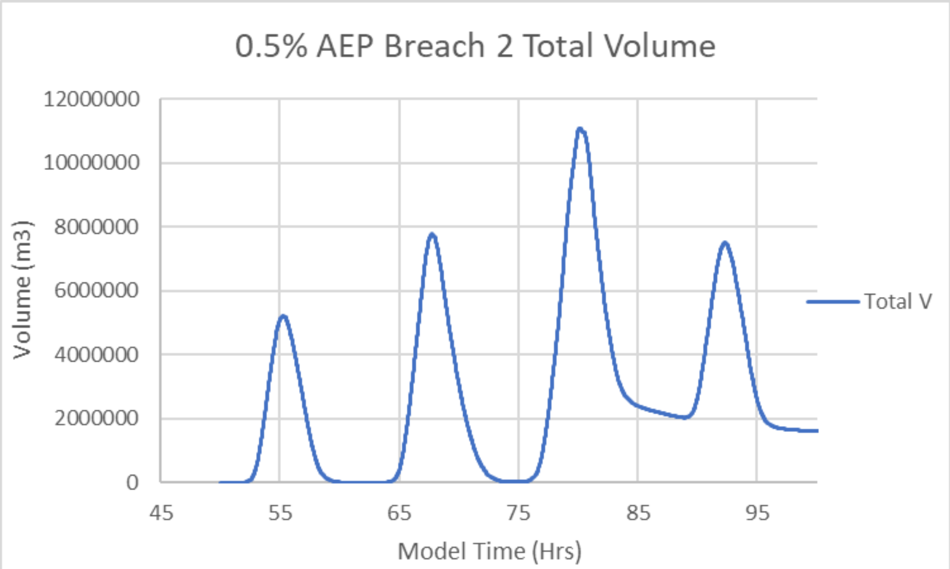


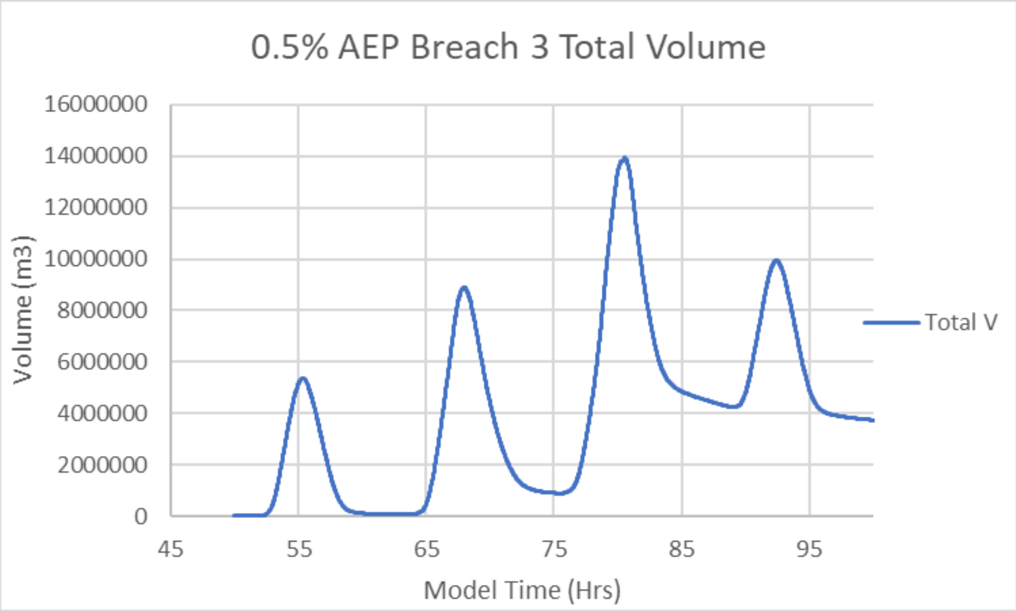
0.5% AEP Breach 4 dVol
100000
80000
60000
40000
20000
0
-20000
-40000
-60000
-80000
45
55
65
75
85
95
Volume (m3)
Model Time (Hrs)
dVol

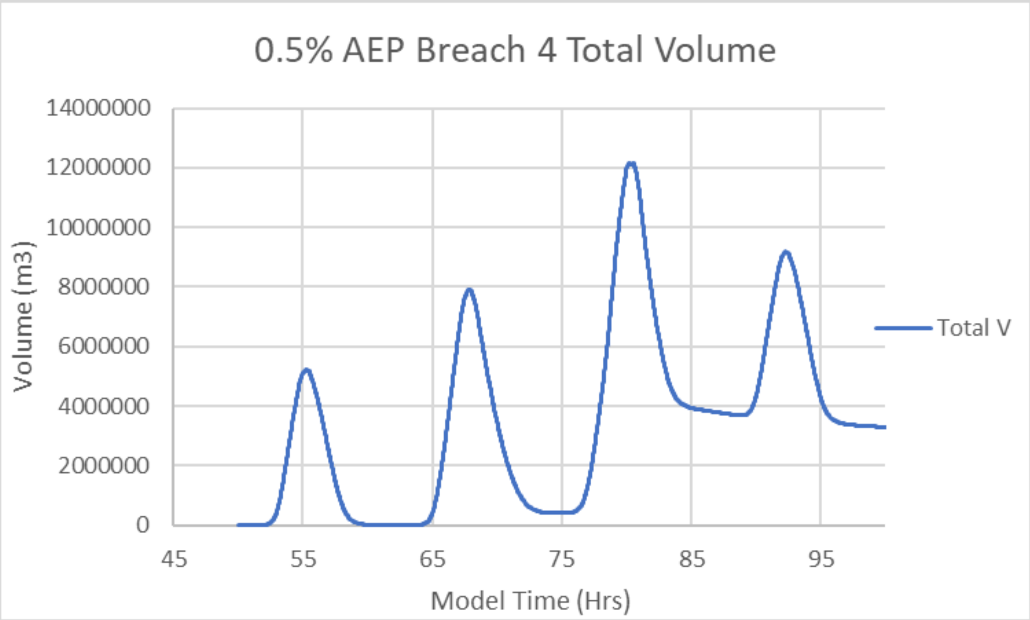
Total Volume

The total volume of water within the 2D domain for the 0.5% AEP breach events is shown graphically below. For all modelled events and breach scenarios, the total volume plots are as expected based on the flood and ebb tidal flux and small overtopping discharges.






0.5% AEP Breach 3 Total Volume
16000000
14000000
12000000
10000000
8000000
6000000
4000000
2000000
0
Volume (m3)
45
55
65
75
85
95
Model Time (Hrs)
Total V


0.5% AEP Breach 4 Total Volume
14000000
12000000
10000000
8000000
6000000
4000000
2000000
0
Volume (m3)
45
55
65
75
85
95
Model Time (Hrs)
Total V

Mass Error

Though HPC is mass conserving it is still important to review Mass Error, as it can still occur when coupling HPC with 1D elements in either the 1D/2D linking, such as for the culverts included in the model. A 'healthy' model will usually report up to ±1% mass error.

Mass error for all model simulations is 0.00 to 0.01, as detailed in Table H-1 for the 4 breach model scenarios.

Mass error does spike at the start of the simulation, this is however expected due to the large tidal range in the model at start up. The initial spike soon disperses as the tide starts to move in and out of the model and by the peak tide the mass balance has settled down.

HPC design time steps

HPC remains stable by reducing its timestep. Due to the underlying solution scheme, HPC typically uses a smaller timestep than Classic. A graph of the target timestep calculated from the model stability criteria is shown below. Typically, a low timestep is classed as 1/10 of the specified 2D timestep, in this case this would be 1s/10 = 0.1s. A repeated timestep occurs when 1 of 3 controls numbers are exceeded by more than 20%. These control numbers are:

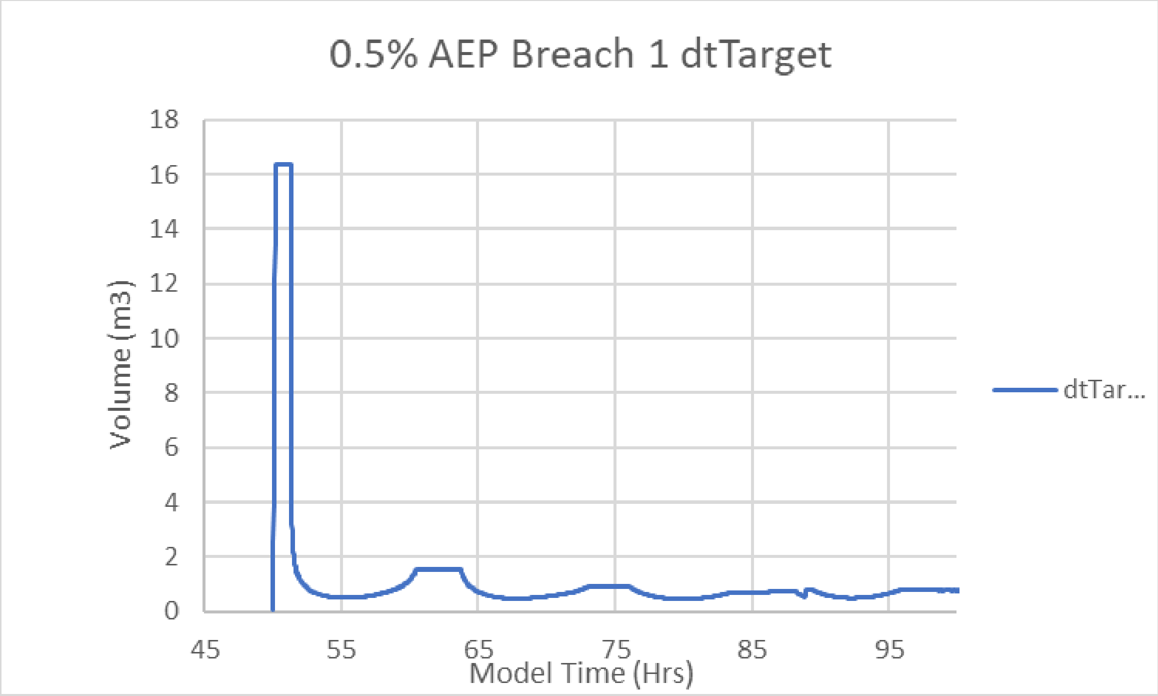
- *Courant Number (Nu)*
- *The Shallow Wave Celerity Number (Nc)*
- *Diffusion Number (Nd)*

Repeated timesteps during each simulation are detailed in the tables below for breach model scenarios. Checks show that the number of repeated timesteps is generally low and tend to occur as volume first enters the model as the shock of cells wetting up causes repeated timesteps, or around the ebb tide trough as flow passes out the model. This suggests the number of repeated timesteps is not an issue and that model health is generally good. Breach 3 scenario shows the greatest number of repeat timesteps. A check on these show they occur between 60 and 64 hours in the model simulations, essentially low water, so should not impact on the maximum modelled results.

All other checks made for these simulations suggest the model is suitably stable.

Should a timestep need to be repeated more than ten times consecutively, the solution stops. The simulation will also stop if the default minimum permissible timestep of 0.1 seconds has been reached. Neither of these two conditions occur.

*Note that coastal models generally deal with much greater depths that with that of fluvial models, and therefore lower timesteps would be expected from that of fluvial models, and greater limitations by the Nu and Nc control numbers.



Simulation warnings and checks

A single warning is given during the simulation "WARNING 0255 - One or more GIS layers not closed during simulation - please notify support@tuflow.com.". TUFLOW support informs us this warning does not impact model results.

There are multiple checks prior to simulation; they all relate to a message to say that a 2D cell was lowered to match the 1D structure invert level (based on the z flag). This is as expected.

Table H-1: TUFLOW breach model log summary for Severn House Farm

AEP	Warning messages prior to simulation	Check messages prior to simulation	Warning messages during simulation	Check messages during simulation	Model run time – CPU time (hr)	Cumulative Mass Error (%)	NaN repeated timesteps	HCH repeated timesteps
1.3%	6	40	1	6	3.53	0.00	0	5
0.5%	6	40	1	6	5.07	0.00	0	3
0.5%+CC NPPF 2068	6	40	1	6	4.35	0.01	0	0

AEP	Warning messages prior to simulation	Check messages prior to simulation	Warning messages during simulation	Check messages during simulation	Model run time – CPU time (hr)	Cumulative Mass Error (%)	NaN repeated timesteps	HCH repeated timesteps
1.3%	6	40	1	6	3.25	0.00	0	2
0.5%	6	40	1	6	5.28	0.01	0	10
0.5%+CC NPPF 2068	6	40	1	6	4.26	0.01	0	0

AEP	Warning messages prior to simulation	Check messages prior to simulation	Warning messages during simulation	Check messages during simulation	Model run time – CPU time (hr)	Cumulative Mass Error (%)	NaN repeated timesteps	HCH repeated timesteps
1.3%	6	39	1	6	3.34	0.00	0	140
0.5%	6	39	1	6	5.30	0.00	0	41
0.5%+CC NPPF 2068	6	39	1	6	4.25	0.01	0	179

AEP	Warning messages prior to simulation	Check messages prior to simulation	Warning messages during simulation	Check messages during simulation	Model run time – CPU time (hr)	Cumulative Mass Error (%)	NaN repeated timesteps	HCH repeated timesteps
1.3%	6	40	1	6	3.25	0.00	0	2
0.5%	6	40	1	6	5.20	0.00	0	3
0.5%+CC NPPF 2068	6	40	1	6	4.28	0.01	0	0