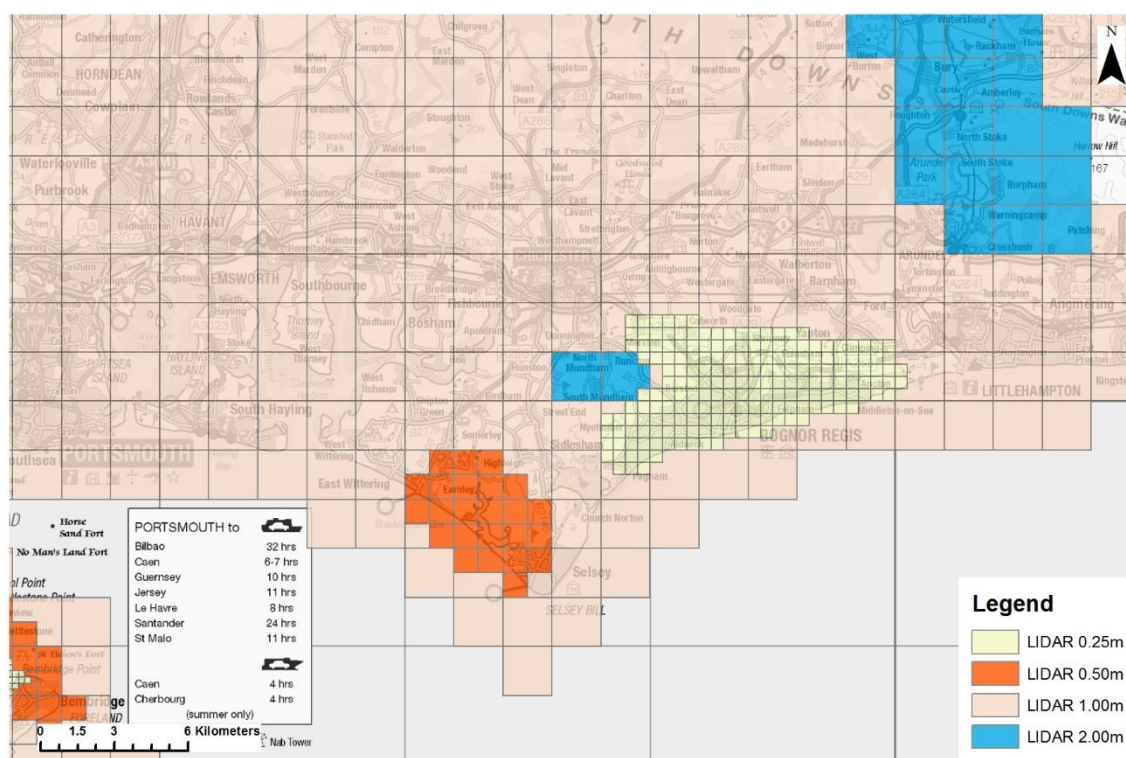


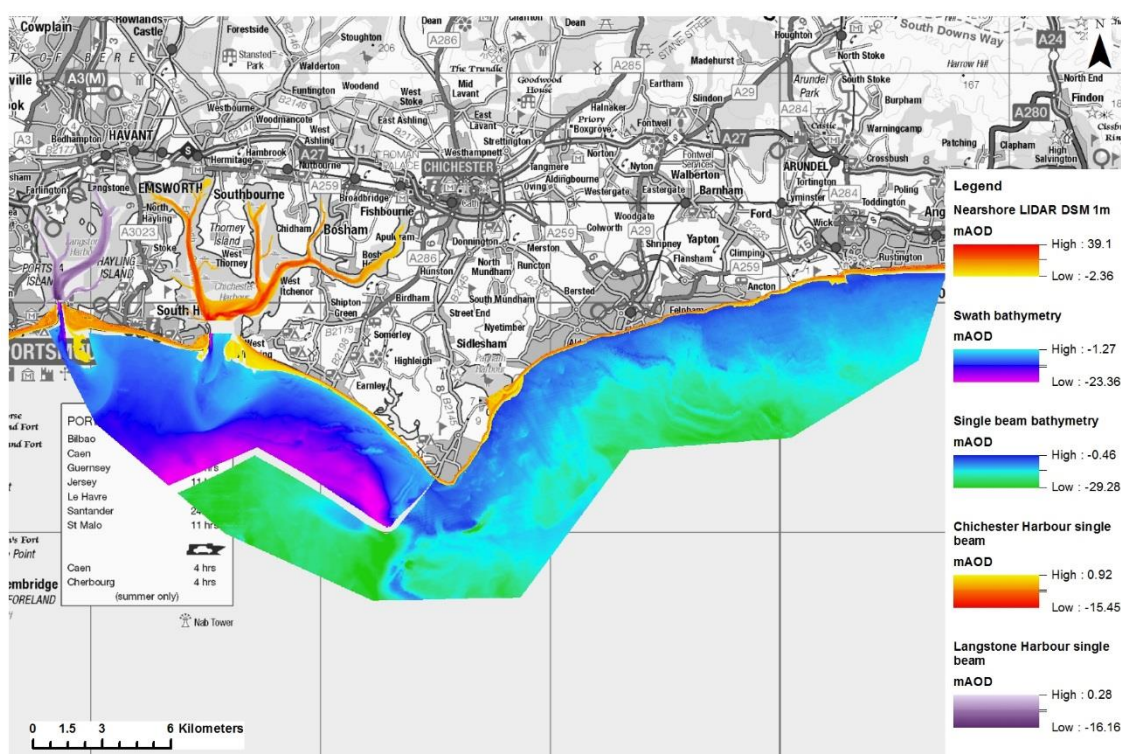


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Figure 4-1: Availability of Light Detection and Ranging data

Bathymetry data

TUFLOW coastal inundation models are generally more stable if their seaward boundaries are located some distance offshore. Whilst LIDAR data used in the construction of the DTM includes some representation of the mud flats and harbour areas in the catchment, this is limited to the nearshore area. To extend the DTM into deeper water, it was merged with bathymetric surveys coastline provided by the Channel Coastal Observatory and the UK Hydrographic Office. All of the data were freely available and are shown in Figure 4.2. The data were arranged so that the detailed LIDAR was used in the nearshore and the swath bathymetry was used where available in preference to the single beam data.



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Figure 4-2: Bathymetry data

Land use data

The rate and extent to which floodwater will flow across a floodplain is controlled partly by the roughness, which varies as a function of land cover type (for example, a woodland will offer more resistance to floodwater flow than short grassland). It was therefore necessary to attribute the terrain data used in the modelling for this study with roughness estimates. This attribution was undertaken using OS MasterMap data; these data contain a highly detailed topography layer where individual features detail land objects and uses. This dataset was used to generate a polygon dataset of different land uses in the study area. Each of these different land uses was then assigned a roughness value, based on the commonly used Manning's n parameter. The assignment of the roughness values was based on standard values given in Chow (1959)⁵ and professional judgement. As discussed in the Model Development Report, these roughness polygon datasets were read into TUFLOW during each model simulation. The DTM data over which the roughness polygons lie are assigned the relevant roughness value to best represent the manner in which floodwater will interact with land objects and varying terrain.

4.2.2 Flood defence data

The study area is well defended in terms of coastal flooding. The large majority of defences are shingle beaches and shingle beaches with raised walls or wave return walls. Infrastructure, including roads and railway lines, also play an important role in limiting the passage of floodwaters. It was therefore important to account for the local defences affording this protection were in the modelling. The resolution of the model grid at 8m is not high enough to properly resolve the flood defences directly from the DTM. To allow the flood defences to be accurately represented, a separate dataset based on higher resolution information, to represent their location and elevation, was generated. The location of flood defences and infrastructure included in each TUFLOW model within the catchment can be found in the Model Development Report that supplements this report.

The final form of the defence dataset derived for the study consists of a point dataset, linked by a line dataset. Each point in the dataset contains an elevation, taken either from survey data, Asset Information Management System (AIMS) defence data or sampled from the LIDAR data. The line

⁵ Chow, V.T. (1959). Open-Channel Hydraulics. McGraw-Hill: Auckland.
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dataset is generally used to link the points and allow TUFLOW to interpolate a continuous defence line within the model.

Defence elevation data

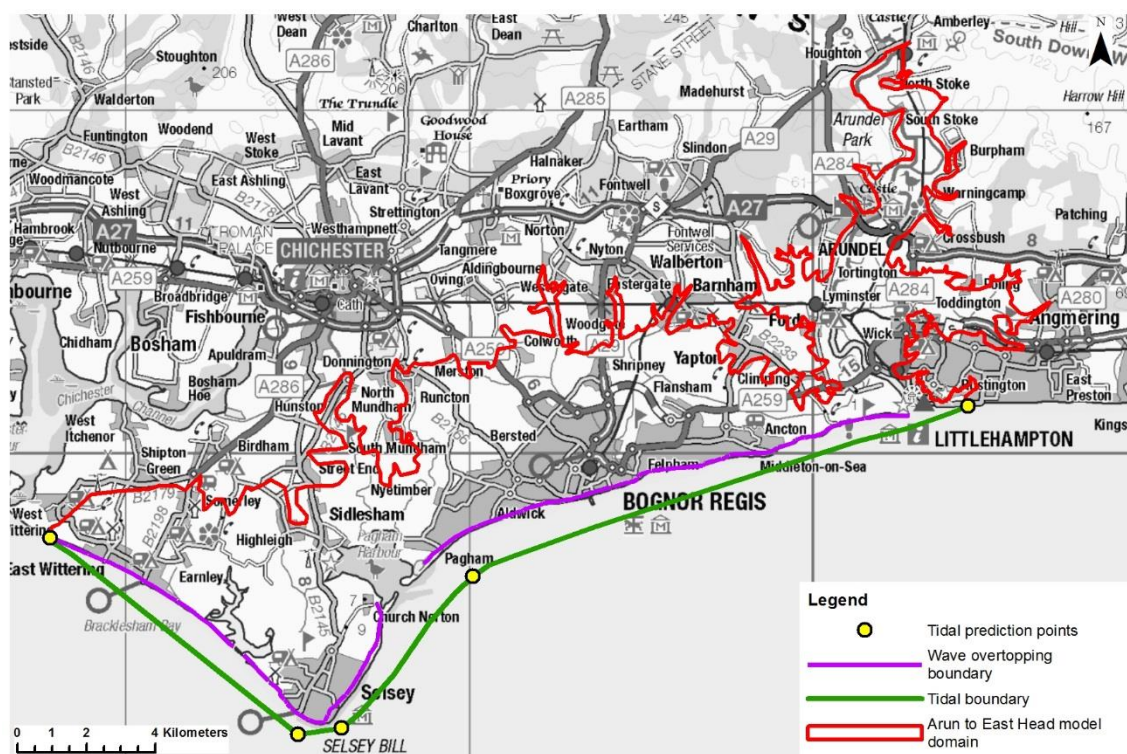
The basis of the defence dataset created for this study was the existing defence data from the Chichester Harbour models, the surveyed crest heights from the beach survey data from the Channel Coastal Observatory (CCO) and levels from high resolution LIDAR data. All defence elevations were verified against crest data in AIMS.

The surveyed data, AIMS data, and the points extracted from LIDAR were then combined and joined by a defence line to create a final defence dataset.

4.2.3 Inflow boundary data

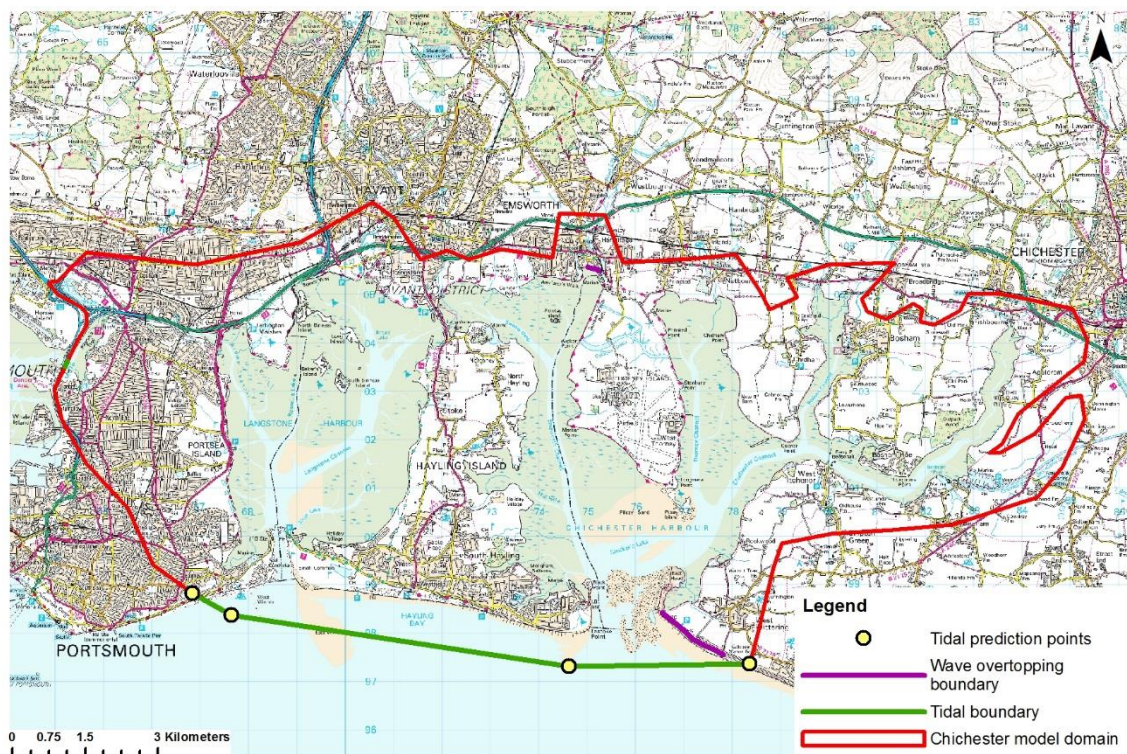
Figures 4.3 and 4.4 indicate the location and type of hydraulic boundaries required for the numerical modelling for the Arun to East Head and Chichester Harbour models. These boundaries include:

- (1) A still water boundary, located offshore, which allows propagation of the tide and surge into the model domain and into the estuary; and
- (2) Wave overtopping boundaries along the coastal frontage, which inject wave water into the model at the location of flood defences.



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Figure 4-3: Arun to East Head model domain



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Figure 4-4: Chichester Harbour model domain

Offshore still water boundary

Modelling the entrance of the tide and surge wave into the model domain required the derivation of design tidal-graphs based on the CFB data points along the Sussex coastline. A design tidal-graph is a time-series that quantifies how sea-levels are expected to change through time during an extreme event. It is these design tidal-graphs that are used to drive the still water component of a flood inundation model at its offshore boundaries.

Derivation of the design tidal-graphs required three principal sources of information:

- (1) Extreme still water sea-level estimates for the events of interest (20%, 5%, 3.33%, 1.33%, 1%, 0.5%, 0.5% (2070 UKCPO9), 0.5% (2115 UKCPO9), 0.5% (2070 NPPF), 0.5% (2115 NPPF), 0.2% and 0.1% AEP events);
- (2) A design surge shape; and
- (3) A design astronomical tide.

The extreme still water sea-level estimates were taken from the latest coastal extreme guidance for the UK⁶. The astronomical tides were based on data from the closest output points using the Total Tide software and the design surge profiles were based on data from the Dover gauge for sites east of Selsey and Portsmouth for sites to the west.

The design tidal-graphs were applied at points along the model boundaries to model offshore still water level boundaries. Methods available in TUFLOW were used to interpolate the levels between each tidal-graph point. The Highest Astronomical Tide (HAT), surge, and CFB data used to create the design tidal graphs for each model is detailed in the Model Development Report. Five design tidal-graph locations were used to model the offshore boundary of the Arun to East Head model and two were used for the smaller Chichester model, as detailed in Table 4-1.

Site CFB Chainage	Extreme Still water level for the year 2014 (mAOD)						
	20	10	3.33	1.33	1	0.5	0.1
ESL4566	3.63	3.70	3.82	3.92	3.95	4.04	4.24
ESL4582	3.33	3.40	3.51	3.62	3.65	3.73	3.92
ESL4590	3.17	3.24	3.36	3.46	3.49	3.57	3.75
ESL4592	3.14	3.21	3.33	3.43	3.46	3.54	3.72
ESL4602	2.98	3.06	3.17	3.27	3.30	3.37	3.55
ESL4612	2.83	2.90	3.02	3.11	3.14	3.21	3.38

Table 4-1: Extreme sea-level data use in the derivation of design tidal-graphs

Climate change

In addition to present day extreme still water level events, 0.5% AEP climate change scenarios for the years 2070 and 2115 were modelled. The water level rise for climate change was based on the latest UKCPO9 sea-level change guidance⁷ using the medium emission 95th percentile scenario and on the sea-level rise estimates from the NPPF. The sea-level rise due to climate change from 2014 to 2070 is estimated to be 0.39m from UKCPO9 and 0.52m using the NPPF values. For 2115 these figures rise to 0.78m and 1.13m for the UKCPO9 and NPPF estimates respectively.

Wave overtopping

Although still water levels often provide the background conditions resulting in a flood event, most of the flooding in open coastal areas occurs through the overtopping of defences due to wave action. Along the Sussex coastline there are several communities that are exposed to swell and wind waves from the south and south-west and wind waves from the south-east, therefore, the accurate representation of the effects of wave overtopping is crucial. If defences are overtopped the impact on the flood extents may be significant.

⁶ Defra. SEPA. The Scottish Government. Environment Agency (2011). Coastal flood boundary conditions for UK mainland and islands. Project: SC060064/TR2: Design sea levels.

⁷ <http://ukclimateprojections.defra.gov.uk/>
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The behaviour of waves in the nearshore and surf zones is highly complex and the subject of detailed research. It is unnecessary to incorporate details of individual wave processes into a flood inundation model but rather to represent worst case conditions at each individual defence. Therefore, a number of assumptions are made to represent wave overtopping at the model boundary for the appropriate design conditions.

The most important assumption is that wave conditions, and therefore wave set-up, 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 at the same time. 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 Flood and Coastal Risk Management Modelling Guidance recommends modelling wave action over a 12-24 hour period, as the waves will then diminish as the storm moves and the wind changes direction. It was assumed that the storm continues with constant wind speeds and direction for the entire progression of the tidal curve, concurrent with the wave action. If water levels at the toe of the defence fall below the water depth physically required to support the incident wave height then the waves are assumed to be 'depth limited' and the waves are reduced to the maximum possible height within the available water depth.

The volume of overtopping is quantified by the parameter 'q', which represents the mean overtopping discharge in $\text{m}^3/\text{s}/\text{m}$. The mean overtopping discharge was calculated for this study using the methods outlined in the European Overtopping Manual (EurOtop). This manual includes methods and guidelines on prediction of wave overtopping at various structures.

For this study the Neural Networks methodology was used. The Neural Network tool was developed by the European CLASH programme and uses a large database of results from physical modelling tests to derive a solution based on a complex defence profile. In each case defence profiles were schematised in order to describe the components of the profile using the parameters required by the analysis tools.

The Neural Network approach requires a set of input parameters for the calculations. These are:

- the still water level at the structure toe (including wave set-up)
- the incident wave conditions at the toe of the defence to be overtopped, and
- the defence profile shape.

Still water levels are provided by the design tidal-graph described previously in this chapter. The other sources of data required for the calculations were obtained as follows.

Wave conditions

Wave conditions at the toe of each defence were required for use in the wave overtopping calculations.

To provide 'worst-case' nearshore wave conditions for use in the wave overtopping calculations joint probability analysis was used to create a series of scenarios representing different water levels, wave heights, wave periods, and wave directions that account for a specific joint probability AEP event. These joint probability scenarios were run through the wave overtopping calculation tools in EurOtop to determine which set of parameters results in the worst case wave overtopping. The worst case combination at each defence was then used in the full tide wave overtopping calculations undertaken for the project.

4.2.4 Flooding from other sources

The TUFLOW models constructed for this study are purely tidal with no assessment of the fluvial, surface water or groundwater flood risk.

5 Technical method and implementation

5.1 General methodology

As described in Chapter 3, coastal modelling is undertaken using a series of models to simulate the different coastal processes of wave transformation, wave overtopping and flood inundation. This chapter details the numerical modelling and describes how the output of each model provides the boundary conditions for the subsequent modelling stages.

5.2 Wave transformation modelling

Wave transformation modelling is required to transform the offshore waves into the nearshore. The model simulates the transformation of wave conditions supplied at the ocean boundary. As wave energy propagates over increasingly shallower bathymetry towards the line of coastal defences, the impact of friction, refraction and shoaling become more important than in the open ocean environment. These processes alter the angle of wave approach and limit characteristics such as the height and period of the wave climate. The growth of waves due to winds is also modelled within the study area.

5.2.1 Wave modelling software

The modelling package used for development of the wave transformation model was SWAN. SWAN is a third generation wave model incorporating complex physics for the description of nearshore processes. It is an open source package used widely for research and commercial application, developed by internationally recognised experts at the Delft University of Technology⁸. The model is capable of simulating all of the following nearshore wave transformation processes:

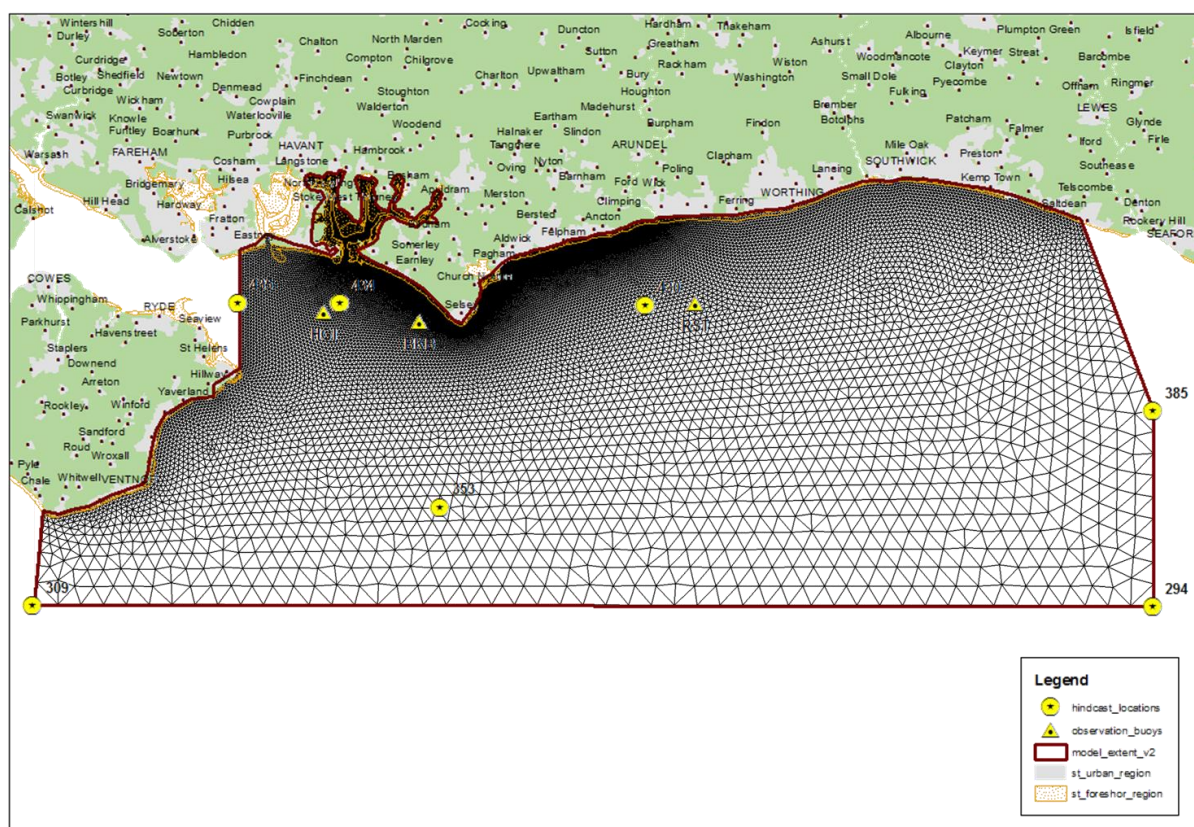
- Wave growth due to wind
- Wave – wave interactions (quadruplets and triads)
- Wave breaking
- Bed friction
- White-capping
- Diffraction
- Refraction
- Reflection.

SWAN is able to calculate steady state wave conditions for specific inputs of wave height, period and direction at the offshore boundary, and wind speed and direction applied across the model domain surface. Water levels can be set to account for tidal variations.

5.2.2 Model domain

The study requires the simulation of nearshore wave parameters for 83 defence locations along the coastline from Chichester Harbour to Littlehampton. The study area is shown in Figure 5.1. This domain was selected based on the availability of offshore data to drive the model boundary. Offshore hindcast wave data were available through Centre for Environment Fisheries and Aquaculture Science (CEFAS) from the UK Met Office Wavewatch III model at the locations plotted in Figure 5.1 (indicated by yellow circles). Nearshore observations of wave conditions in the area are available from the CCO operated Waverider buoys at Hayling (HGI), Bracklesham (BKB) and Rustington (RSK) at approximate water depths of 10.0m, 10.4m and 9.9m Chart Datum (CD), respectively.

⁸ SWAN User Manual, SWAN Cycle III version 40.81, Delft University of Technology, 2010
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Figure 5-1: Wave transformation model extent covering the coastline from East Head at West Wittering, Bracklesham Bay, to Selsey, Bognor Regis and up to Littlehampton

5.2.3 Terrain and bathymetry data

To construct the wave model, seabed bathymetry and ground level data were required. These data constitute the largest dataset required for the wave model and were used to represent the surface over which waves propagate and interact. The ground level data, described in Chapter 4 was combined to produce a seamless bathymetric/topographic grid. These datasets were used in order of priority according to their resolution i.e. where the LIDAR data overlapped with the bathymetry data on the beaches the LIDAR data were used in preference. Prioritising the datasets in this way means that the best quality data were used where they were available. The LIDAR (1m and 2m resolution) were used in the foreshore areas, intertidal and depending on the calibre of the data to approximately the 1m elevation contour. The swath bathymetry is the next highest resolution and covers much of the coastline west of Selsey to the western boundary. Singlebeam data were available in two resolutions. The higher resolution was used in the nearshore areas from Selsey to beyond Littlehampton and the coarser singlebeam data were used offshore. Gaps were left between the different datasets to allow interpolation and ensure a smooth transition between the differing resolution data, so that this does not have a detrimental effect on the model results.

5.2.4 Wave model mesh development

The grid on which the model calculations were performed was constructed as an unstructured mesh using irregular triangular elements. The points of the triangles are the model grid nodes at which computations of wave parameters were performed. Such a grid allows for high resolution representation in the nearshore, where wave behaviour can change rapidly over short distances, and low resolution offshore where changes are more gradual. The use of an unstructured grid increases the computational efficiency of the model. At the coastline and in the swash zone, the spatial resolution of the final model mesh was generally 25m whereas at the offshore boundary, where depths were greatest (over -60mODN), the spatial resolution was 2,000m.

Figure 5-2 illustrates the final merged bathymetry data and the resolution of the unstructured model mesh graded to bathymetry. Generally, as the bathymetry shallows the mesh increases in

resolution (to approximately 10m). Further, the mesh is resolved around key areas of interest such as the toe of the defences, coastal structures (groynes, piers and breakwaters) and very shallow areas such as sand bars. This increase in resolution provides more detail to the model mesh and enables the model to better resolve shallow wave processes. The final bathymetry is created by extracting an elevation at each node from the bathymetry tiles described in the previous section.

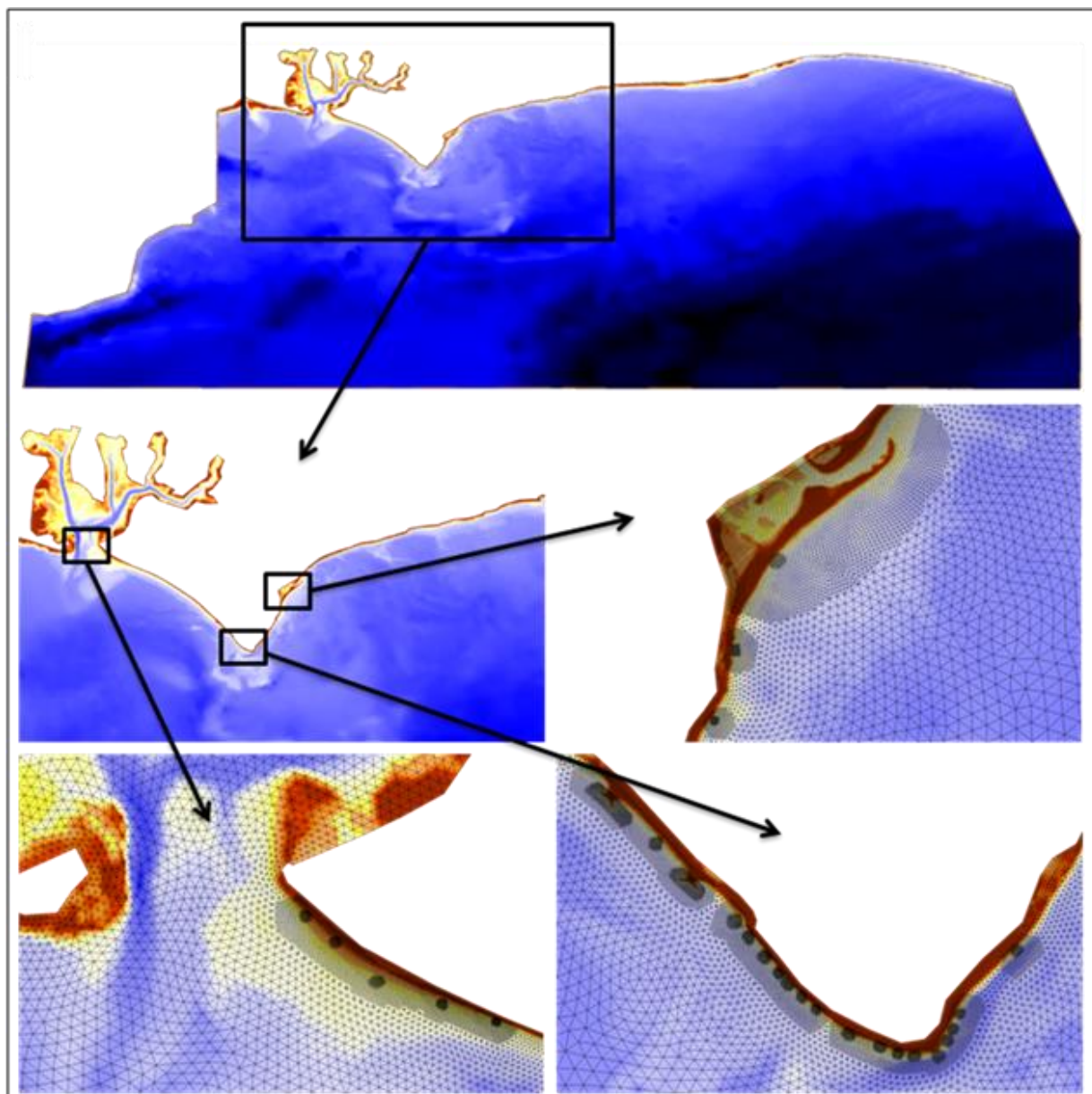


Figure 5-2: Final model bathymetry and unstructured model mesh

5.2.6 Sea-level boundary data

The wave transformation model must be given boundary information to represent the surface of the sea. This can be supplied either as a single level across the model grid or as a varying grid across the model domain. For the ensemble simulations a single water level was used for each simulation as every combination of conditions was simulated at all locations, whereas for the AEP simulations varying water level grids were created to represent the individual AEP conditions at all locations along the coast.

5.2.7 Wave boundary data

The wave transformation model required an offshore wave boundary (height, period, and direction) to allow deep water waves from the Atlantic Ocean to propagate into the model. Total sea waves and swell waves were considered as separate forcing scenarios. The term total sea wave refers to the total wave energy contained in the wave spectrum including both locally generated waves and swell waves. Swell waves are long period waves generated by storms far away from the site of interest. Offshore waves from three directions of wave exposure were considered: 135° south-west (SW), 180° south (S) and 225° south-east (SE). These wave parameters were used to derive an offshore energy spectrum used to drive the model. A JONSWAP spectrum was used with the characteristic period set to the peak period.

The WWIII wave data provide more detailed hindcast model data than were previously available from the UK Waters hindcast. The WWIII includes four-way partitioning of the spectra such that, as well as wind waves, up to three separate swell events may be distinguished during each time interval. Previously, the analysis had simply distinguished wind waves from swell resulting in an averaging of different swell events arriving at the same time. Hence, well-developed, very low height and long period swell waves travelling from far offshore were averaged with perhaps more locally generated newly forming swell waves characterised by larger wave heights and shorter periods.

The offshore and Solent boundaries for the model required the assignment of appropriate boundary wave conditions for propagation into the model domain. Wave conditions were generated through extremes analysis of the WWIII data. Four of the WWIII points were selected to provide the wave data along the model boundary, these were points 294, 309, 385 and 435, as shown in Figure 5.3. The hindcast data from the Met Office wave model at each of these locations provides data from 1980 to 2014. Point 309 represents the southwest boundary and the south boundary is variable towards the east. Point 294 represents the eastern extent of the variable south boundary to provide the deepest wave conditions. Point 385 represents the east boundary. Point 435 represents conditions in The Solent, the stretch of water between the mainland and the Isle of Wight.

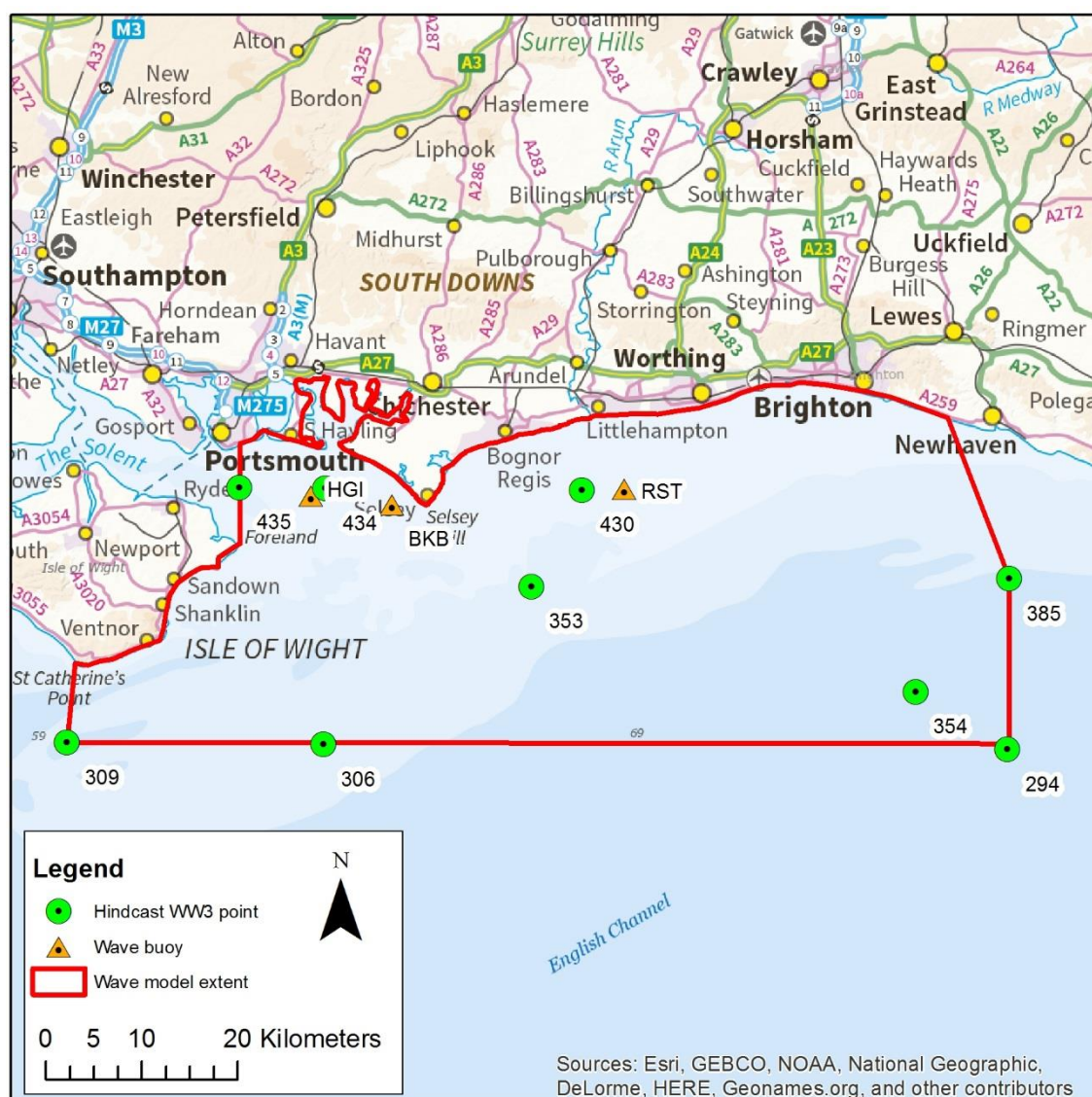


Figure 5-3: Location of Met Office hindcast wave data points used for comparison of offshore conditions

Extreme value analysis

Revised extreme wave heights using the new WWIII data were generated using an extreme value marginal analysis Peaks over Threshold (PoT) approach⁹. This involves defining a high threshold, fitting a Generalised Pareto Distribution (GPD) to all the observations that exceed the threshold, and estimating the rate of threshold exceedance. The PoT method requires a long time series of complete data for the analysis. WWIII hindcast wave data are of suitable length and completeness for the PoT analysis.

Since locations around the coastline do not have equal exposure to waves from all directions, wave direction must be considered as a covariate in the PoT analysis. Thus, the AEP of wave heights derived for a particular direction is a function of both the probability of the wave height occurring and the probability of the wave travelling from a particular direction. The data were therefore split into the SW, S and SE direction sectors and a probability distribution of wave heights was generated for each sector.

The estimated extreme total-sea wave heights and extreme swell wave heights are presented by direction sector in Table 5-2. Note, swell waves were only derived for the SW sector since there were insufficient extreme wave height data available from the S and SE direction sectors to derive a distribution. Swell wave heights from the SW sector are much higher than from the S and SE and so represent greater flood risk to West Sussex coast. The S and SE swell waves were therefore not modelled as they do not represent the worst case flood risk. All three partitions of

⁹ Coles, S. (2001) An Introduction to Statistical Modeling of Extreme Values, Springer-Verlag, ISBN 1852334592.
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the swell waves were interrogated to find the largest swell events for the extremes analysis as these give the worst case flooding. For the swell waves, there is only a relationship between wave heights and periods for waves between 0.5 and 2.5m in height, beyond this an average period of 10.25s was used.

Table 5-1: Significant wave height (m) by direction sector, total sea and swell waves

	Total Sea Waves (m)			Swell Waves (m)	Swell Waves Period (Tp)
AEP event (%)	South-west (WW3 point 309)	South (WW3 point 306)	South-east (WW3 point 294)	South-west (WW3 point 309)	South-west (WW3 point 309)
100.00	5.16	4.01	2.01	2.10	14.12
50.00	5.75	4.45	2.35	2.35	14.78
20.00	6.47	4.94	2.76	2.66	10.25
10.00	6.96	5.25	3.04	2.86	10.25
5.00	7.42	5.52	3.30	3.06	10.25
3.33	7.67	5.66	3.44	3.16	10.25
2.00	7.97	5.83	3.61	3.29	10.25
1.33	8.20	5.95	3.74	3.38	10.25
1.00	8.35	6.02	3.83	3.44	10.25
0.66	8.56	6.13	3.94	3.53	10.25
0.50	8.70	6.19	4.02	3.59	10.25
0.40	8.81	6.24	4.08	3.63	10.25
0.25	9.02	6.34	4.20	3.72	10.25
0.20	9.12	6.38	4.26	3.76	10.25
0.10	9.41	6.51	4.42	3.88	10.25

5.2.8 Joint probability of sea-level and wave heights

For each of the AEP simulations, a joint probability analysis was undertaken based on sea-level and offshore wave height. Joint probability analysis was necessary to derive possible combinations of storm forcing conditions that correspond to each modelled AEP event.

Joint probability was undertaken using the Department for Environment Food and Rural Affairs (Defra) desk-based method¹⁰. This method was used to evaluate the joint probability of the marginal sea-level extremes from the CFB data and the marginal wave heights given in Table 5-2 (total sea and swell waves). This joint probability analysis was undertaken for each wave type and each direction sector separately, resulting in a wide range of possible combinations of water level, wave height and direction that each equate to an overall AEP. The joint probability combinations of conditions are detailed for the present day scenarios in Appendix A.

5.2.9 Wind boundary data

The final storm forcing data required for the wave transformation model were wind speed and direction data. These data were required to ensure realistic wave generation and propagation within the model domain. Wind conditions were generated again using the hindcast data from WWIII.

It was not possible to undertake a three-way joint probability analysis that included sea-level, wave height and wind speeds using the Defra methods described in Section 5.3, therefore, extreme analysis of wind data could not be incorporated in the calculation of event storm forcing conditions. Instead, an "associated" wind speed was derived for each of the extreme wave heights given in Table 5-2. This assumes high dependence between wave height and wind and is based on regression analysis of the offshore total sea wave height and wind speed data contained in the WWIII dataset to derive a relationship. Relationships were developed for each of the three

¹⁰ Use of Joint Probability Methods in Flood Management: A Guide to Best Practice, Defra and the Environment Agency, March 2005, including associated spreadsheet.
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direction sectors. Figure 5-4 shows the fitted relationship between the wind speeds and wave heights for the south-west direction sector.

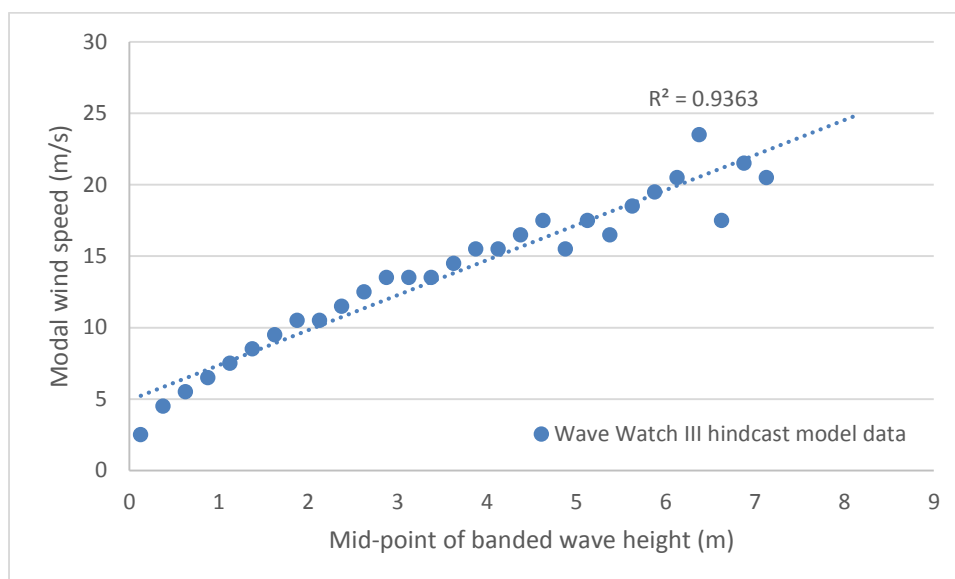


Figure 5-4: Wave height and wind speed relationship for the south-west

5.2.10 Wind and swell waves

Swell waves may travel large distances before they reach the coastline and so have no association with the prevailing wind conditions. The four-way partitioning of new WWIII wave spectra data has produced more detailed information on swell waves than was previously available from UK Waters. The four-way partitioning of wave spectra allows swell waves of varied origins to be distinguished from one another. Hence, very long period, small waves from distant locations may be distinguished from larger swell waves arriving due to storm activity in the North Atlantic.

It is the larger waves that are of most interest to modelling flood risk since these larger, more locally developed waves are more likely to occur coincidently with extreme sea-levels and high wind speeds, each generated by the same storm arriving from the North Atlantic.

There is no clear correlation between swell waves and wind speed, therefore, an average coincident wind speed of 10.5m/s was applied for all swell events.

The "associated" wind speed for each of the modelled total-sea and swell waves were derived and applied in the wave transformation model using the newly defined relationships. To evaluate how wind conditions, affect flood risk for each AEP event, worst-case modelling for each of the combinations of water level and wave height were run for winds originating from the three different wind direction sectors (i.e. 135°, 180° and 225°).

5.2.11 Wave period boundary data

For each wave height it was also necessary to derive an appropriate wave period to apply at the boundary of the model. Wave period is known to have a significant effect on wave overtopping discharge and a detailed analysis was therefore carried out to derive the associated wave period for each wave height. As with wind speed, the associated wave period for each offshore total sea wave height was derived by undertaking a regression analysis of the offshore total sea wave height and wave period contained within the Wave Watch III hindcast dataset for each direction sector. The relationship derived for the south-east sector is shown in Figure 5-5.

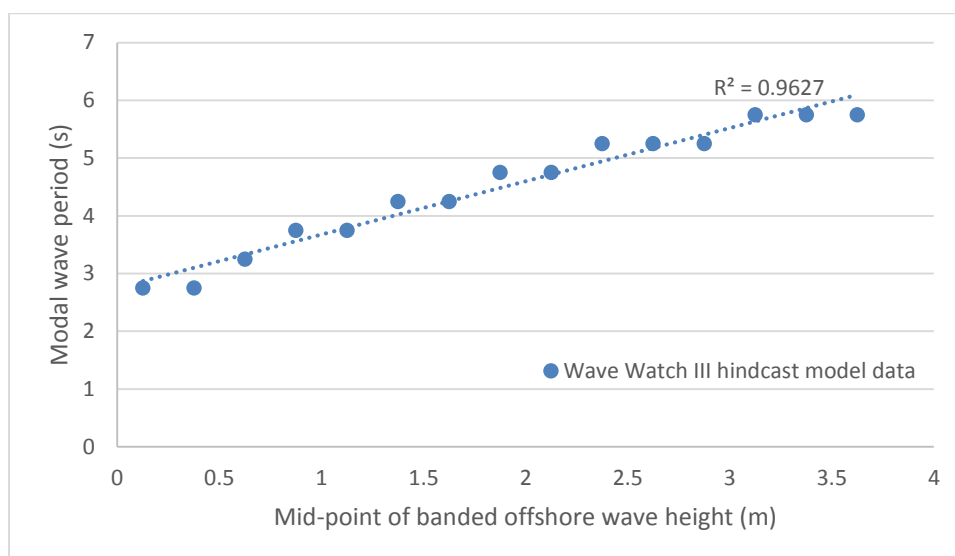


Figure 5-5: Wave Watch III hindcast south-east wave regression relationship between wave period (y) and offshore total sea wave height (x)

5.2.12 Swell wave period

Although a relationship exists between swell wave period and wave height, the relationship is not represented as clearly by a linear trend as that of total sea waves. Regression analysis to derive the relationship for swell waves between offshore wave height and period was only possible for south westerly swell wave heights between 0.5 and 2.5m, as shown on Figure 5-6. For wave heights above 2.5m an average period of 10.25s was used.

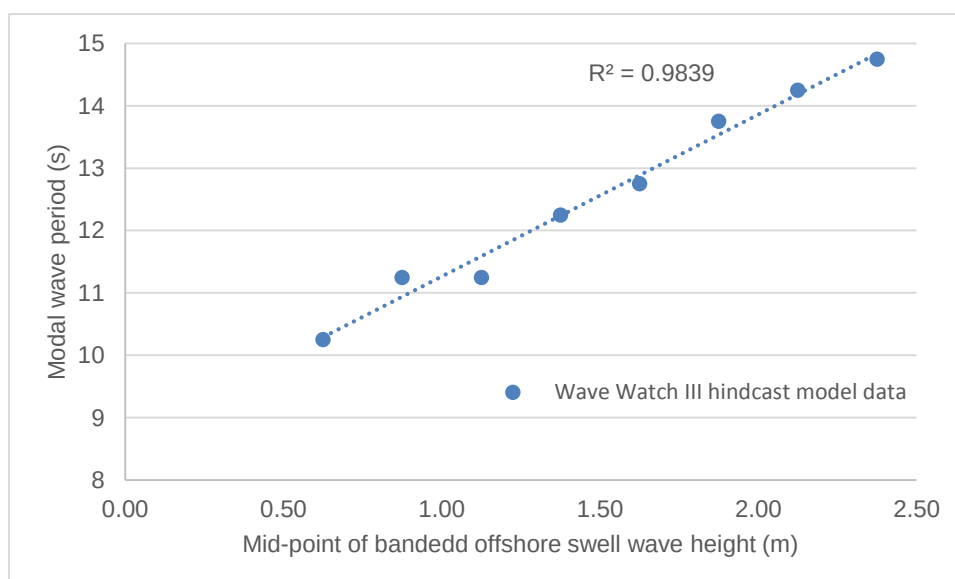


Figure 5-6: Wave Watch III hindcast south-west wave regression relationship between wave period (y) and offshore swell wave height (x)

To derive the associated wave period for swell waves, the data were segregated into group according to wave height and period and modal wave periods selected. These were applied to the offshore swell wave heights for which they are valid, based on the Met Office hindcast data, for each direction sector and favouring the longer wave periods to give the worst-case condition.

The above process resulted in 516 combinations of sea-level, wave and wind properties that each equate to the overall AEP event modelled.

5.3 Wave overtopping modelling

Following the completion of the wave transformation joint probability simulations the next step is to calculate the associated wave overtopping discharges relevant to each defence section of

interest. As discussed previously, wave overtopping calculations require a number of input conditions, which may be categorised as sea-state conditions and defence profile shape.

5.3.1 Derivation of Wave Overtopping Inputs

5.3.2 Sea-state Conditions

Wave overtopping calculations are inherently complex, resource intensive and defence specific. The recommended approach for the estimation of wave overtopping over complex coastal defence structures is through one of three empirical methods specified in the EurOtop. These methods require wave properties at the toe of coastal defences, which are calculated using the wave transformation model providing the wave height, period and direction in the nearshore coastal zone.

5.3.3 Defence Profile Shape

In addition to the nearshore wave data the overtopping calculations also require detailed information on the defence structures, including details on the defence type, construction material, crest, berm and toe elevation data and geometric properties such as the slope angles of the various defence facets. A single profile location was then chosen to represent each defence section in the wave overtopping calculations. However, given that wave overtopping discharge is expected to change significantly according to the shape of the defence profile, it is important that more than one defence profile shape is considered for each representative location to show the potential range of overtopping that could occur. The profiles were compiled with beach survey data from the CCO, LIDAR data and AIMS data sets.

Given the significant amount of information available to the study, there were a number of choices for the profile shapes that could be assessed. Where several beach profiles had been surveyed along one section of defence, the more recent profiles from 2012-2014 were favoured and a typical profile was selected from the CCO data to represent the relevant section of beach. This was cross-checked and refined using a combination of available data, such as LIDAR and AIMS defence data, as well as site visit notes and photographs. In total 84 separate flood defences were schematised along the length of the study reach.

5.4 Calculation of Wave Overtopping

5.4.1 Overtopping Model

All wave overtopping calculations are to be undertaken using the methods outlined in the EurOtop. This manual includes methods and guidelines on prediction of wave overtopping at various structure types. For this study, Neural Networks, was utilised as this provides the most suitable methodology to assess combined defences which include seawall structures.

Neural Network overtopping calculations require a set of 15 specific defence inputs. The parameters are obtained through manually schematising profiles of the defence structures. The defence schematisations were created using LIDAR, defence survey data and structural plans. Table 5-2 describes the parameters used and how they are calculated; a graphical illustration in Figure 5-7 summarises the 15 parameters in relation to the profile. The first schematising process involved breaking sections of coastline down based on defence type, materials, changes in crest height elevation and changes to the direction towards which the defence faces. Once the profiles were established the input parameters were calculated and used to run the Neural Network Overtopping models.

Table 5-2: Neural Network input parameters

Variable	Method of calculation
Angle of wave attack, $\beta(^{\circ})$	The angle of wave attack is the dominant direction of the oncoming wave field. For the purposes of this study the worst case scenario of head on waves were used.
Water depth in front of structure, h (m)	The water depth at the base of the structure, obtained from relevant extreme sea water level AEPs.
Significant wave height at the toe, $H_{m0,toe}$ (m)	The significant wave height is obtained from the wind driven wave conditions discussed previously. The wave heights are depth limited by the water level at the toe.

Variable	Method of calculation
Wave period, T_{m-10} (s)	The wave period used is derived from the wind driven wave conditions which gives a peak period value, T_p , however Neural Network inputs require a spectral mean wave period, T_{m-10} .
Water depth at the Toe, h_t (m), Water depth at Berm, h_b (m)	This is the depth of water from the top of the toe level and is obtained by taking the difference between the two elevations. If the toe level would be zero, $h_t = h$. The berm level must be within the range of $1.5 \times$ the wave height plus or minus the water level at the toe.
Width of toe/berm/armour, B_t, B, G_c (m)	The toe width is accounted for when there is a formal toe construction and is measured on the top. The berm width measures the horizontal difference between the start and end points of the berm. The armour width, also measured horizontally is the distance between the armour crest and the overall crest location.
Roughness coefficient, λ / break in roughness (m)	The roughness coefficient indicates the permeability of the structure. Higher permeability will lead to more wave energy dissipation and therefore lower overtopping discharge. Different material types are given specific values and their relevant locations. Multiple roughness values can be used for more complex defences.
Angles of down/up slope Berm slope $\cot \alpha_d$, $\cot \alpha_u$, $\tan \alpha_b$	The up and down slope parameters define the general slope of the structure. They are the cotangents of the mean gradient around them berm level. The value is obtained by taking the slope from the berm start to the toe (down slope) and the berm end to the armour crest (up slope). The berm itself can also be sloped and is found as the gradient of its start and end values.
Crest/Armour freeboard , R_c , A_c (m)	The freeboard describes the distance between the crest or armour level and the still water level. At this point the waves could be stopped by the structure.

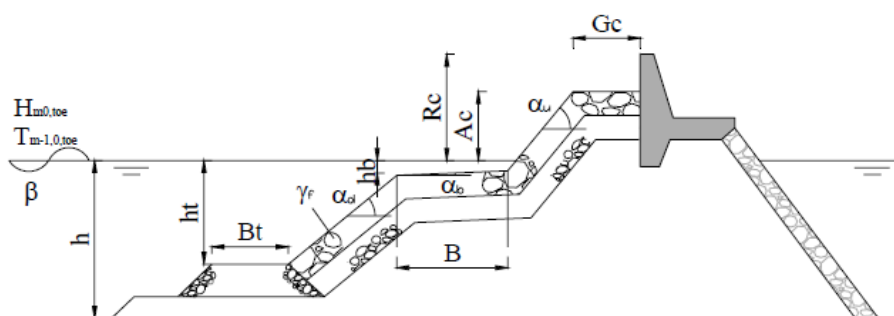


Figure 5-7: Parameters used (and the descriptions) in Neural Network schematisation for wave overtopping modelling¹¹

There are several limitations with regards to the ranges of values within the overtopping model. The berm and crest widths and structure slopes all can significantly reduce overtopping when increased and cause the model to fail when exceeding Neural Networks tools ranges. To overcome these, the profile must be re-schematised to bring the values within range. The roughness coefficient also has a large importance and significantly affects the overall overtopping values. Therefore, detailed analysis of material type at each defence is highly important.

5.4.2 Overtopping Simulation

Once each of the wave overtopping simulations were computed for each of the 84 defences, the discharges were integrated within the flood inundation model by digitising inflow lines just behind the crest of each section of defence. The timing of the injection of wave water was synchronised with the still water boundaries ensuring that the two types of flood water interact appropriately where required.

5.5 Flood Inundation modelling

The final stage of the modelling process was the flood inundation modelling. Inundation modelling for the study was undertaken using TUFLOW (Two-dimensional Unsteady Flow) v2013-12-AB. TUFLOW is a fully hydrodynamic flood model suited to modelling the progression of flood waves across a 2D floodplain. This modelling framework has been used extensively in coastal flooding

¹¹ Coeveld E.M, van Gent M.R.A, and Pozueta B, 2005. Crest level assessment of coastal structures by full-scale monitoring, Neural Network predictions and hazard analysis on permissible overtopping. CLASH

studies worldwide. TUFLOW has an upper limit in terms of the number of grid cells that it is able to compute during a simulation. The consequence of this is that the size of the floodplain and the resolution of the modelling to be achieved must be considered carefully in order that the most appropriate use of grid cells is made.

5.5.1 Model approach

Existing TUFLOW flood models were available covering the coast from the River Arun to Pagham and covering Chichester Harbour. These models were updated to incorporate new extreme sea-level data from the CFB Data study. The tidal boundaries were re-positioned offshore to allow the tides to propagate more realistically into areas such as the Emsworth and Chichester channels. The models were also updated to take account of new defences, such as the breakwaters and embankments built as part of the Medmerry realignment scheme.

The models were updated to incorporate new LIDAR and bathymetry data described in Chapter 4 and the new wave overtopping inflows.

5.6 Model outputs

The base outputs required from each of the flood inundation model simulations were:

- Maximum water level grids
- Maximum depth grids
- Maximum velocity grids
- Hazard rating grids.

5.6.1 Flood outlines

Remove dry islands

Flood outlines were generated from the raw model depth grids. These raw outlines contained dry islands which are areas of elevated topography that are surrounded by water but which are not themselves flooded. In keeping with the standard practice on Environment Agency Flood Map projects, dry islands with less than 250m² were removed. This manipulation was carried out using a GIS routine developed by JBA.

Wave overtopping gaps filled

Wave overtopping volumes are input to TUFLOW via a model boundary located on the landward side of the coastal defence. Assuming no still water flooding occurs then the defence itself will be left dry when in reality it will be wet. These gaps in the flood outline were filled manually.

Final revised Flood Zones

A new Flood Zone 2 outline derived from the 0.1% AEP scenario and a new Flood Zone 3 outline derived from the 0.5% AEP scenario were produced as digital deliverables.

5.6.2 Creation of areas benefiting from defences

Once the final Flood Zone 3 and 0.5% AEP residual flood risk area (RFRA) outline was completed, generation of the ABD polygon was a subtraction of the two. The ABD area is the region between the periphery of the RFRA and the periphery of Flood Zone 3. This is the area that benefits from the presence of flood defences. Following subtraction of Flood Zone 3 and the RFRA, it was necessary to make a number of minor amendments to the remaining ABD areas. This included removing small ABD polygons in areas where there is no benefit in showing an ABD (i.e. in very rural areas or on embankment tops; polygons with areas less than 200m² were removed) and a number of additional manual modifications that were required to tidy and improve the final ABD area. The Environment Agency may wish to filter the remaining ABD area further before uploading to the Flood Map.

5.6.3 Hazard rating grids

Flood Hazard Rating grids were generated as part of this study. 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 Defra's hazard rating thresholds¹². A debris factor of 0.5 was used for depths greater than 0.25m and a debris factor of 1.0 was used for depths less than 0.25m. These debris factors correspond to those recommended for urban floodplains in Defra's Supplementary Note. This 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.

Hazard grids were generated for all modelled events and scenarios as part of this study. Following the completion of the modelling within this study, the Estuary modelling standards (CH2MHill, 2015) were released and these recommend a grid resolution of less than 6m to be able to accurately model flow velocities and hazards. Users of the hazard outputs should be aware that the model resolution of 8m is beyond the 6m recommended within the new guidance.

5.6.4 Onset of flooding

Onset and duration of flooding grids were extracted from the model event simulations which detail the duration of inundation (duration grid) and the time of first inundation above a depth of 0.01m (onset grid). Both onset and duration grids were extracted for the 0.5% AEP event as part of this study.

5.6.5 Flood inundation animations

The model grids were used to generate a series of flood inundation animations to enable understanding of flood flow pathways.

¹² Defra (2008) Supplementary Note on Flood Hazard Ratings and Thresholds for Development Planning and Control Purposes.

6 Model proving

6.1 Introduction

This Chapter covers the model proving, calibration, validation and sensitivity testing that was performed on the wave transformation, overtopping and flood inundation models.

6.2 Wave transformation model calibration

The wave model was initially calibrated against 10 storm events using hindcast model data to force the model at the offshore boundaries and observational wave buoy data at three locations. Comparing hindcast model data to observed results enables a better understanding of how well the model is performing and under which set-up conditions the most accurate simulations are achieved. In order for the model to perform well under the largest of storms events observational data were obtained from three Waverider Buoys operated by the CCO. The observations range from 2004-2012 for the Hayling Island (HGI) and Rustington buoy (RST), and from 2008-2010 for the Bracklesham buoy (BKB).

The wave data were analysed by wave height, period and direction for each of the three buoys. The HGI buoy observations are characterised by waves with a mean height of 0.66m, the mean peak period is 7.67s and mean direction of 189°. This buoy is in the most westerly location and observations of the wave climate here are likely to be impacted by refraction and sheltering around the Isle of Wight, therefore the wave direction is more westerly than at the other two locations.

The BKB observations are characterised by waves with a mean height of 0.69m, the mean peak period is 7.74s, and mean direction is 208°. This buoy has a more south-westerly wave climate as it is not as sheltered as the HGI buoy, the greater mean wave period and heights are due to this exposure to southwest from open Atlantic Ocean swell conditions.

The RST observations are characterised by waves with a mean height of 0.80m, the mean period is 6.58, and mean direction of 195°. This buoy represents much of the more easterly wave climate from the North Sea, as well as the Atlantic and locally driven wave conditions. The RST buoy is the most exposed and therefore wave energy is not as attenuated as with the other two buoys. The shorter mean period reflects the North Sea (easterly) component in the wave climate at this location, where the period is typically shorter than the Atlantic (westerly).

Wave roses for each of the wave buoys are presented in Figures 6-1, 6-2 and 6-3.

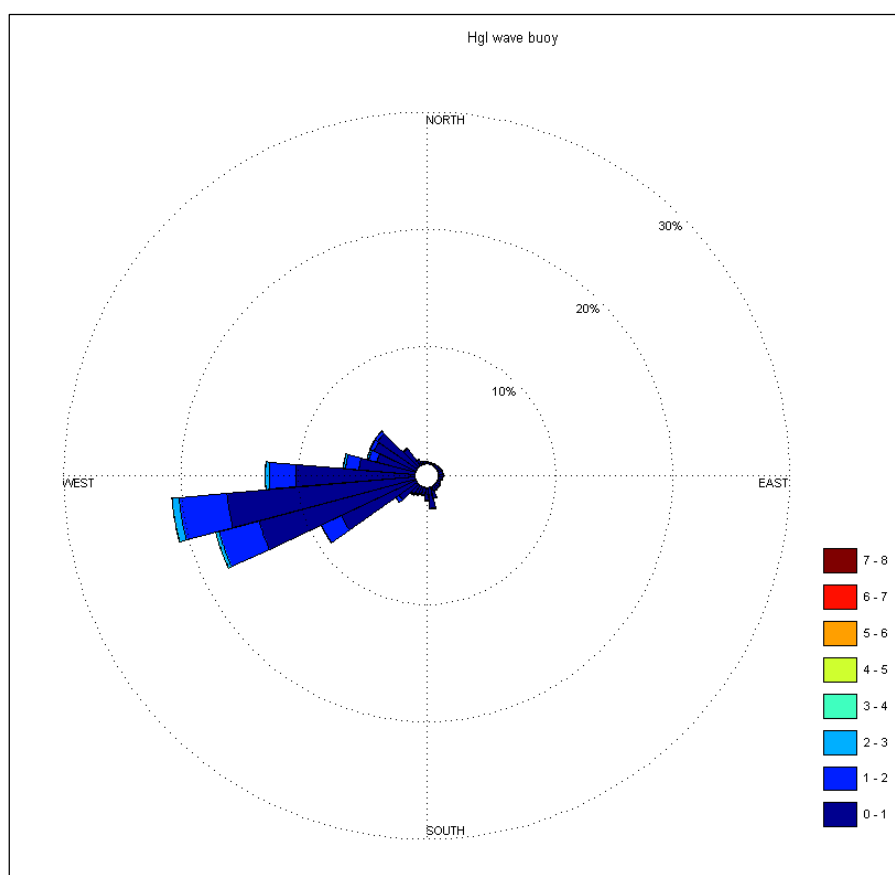


Figure 6-1: Wave direction histogram illustrating observations at Hayling Island (HGI) waverider buoy. Filled bins are wave heights (m)

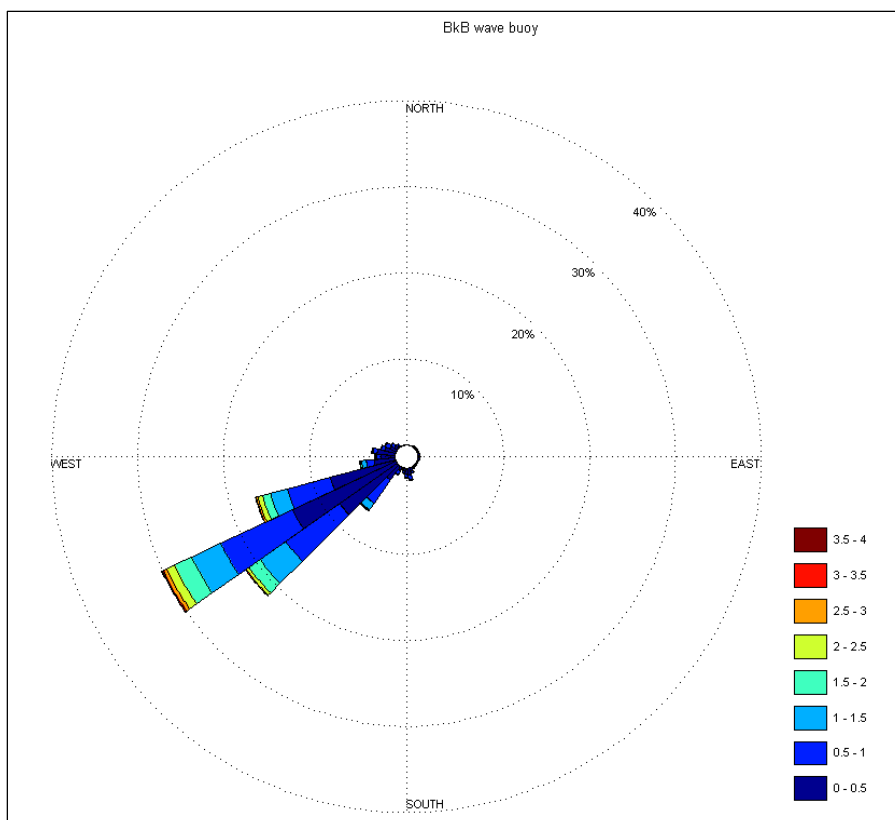


Figure 6-2: Wave direction histogram illustrating observations at Bracklesham Bay (BKB) waverider buoy. Filled bins are wave heights (m)

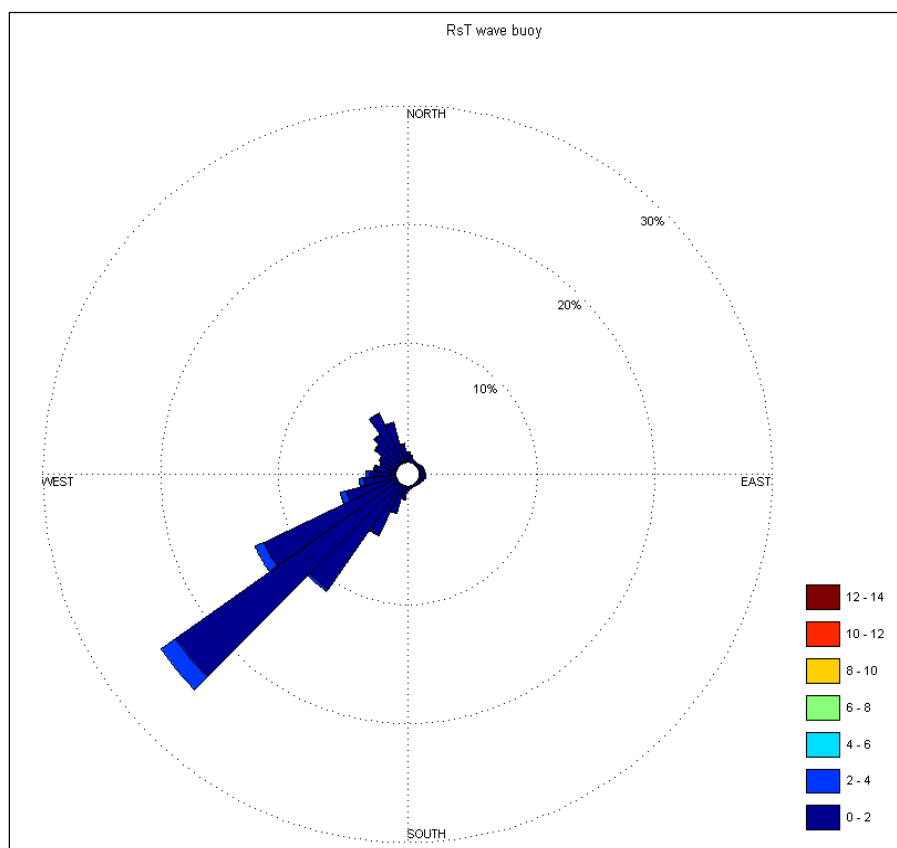


Figure 6-3: Wave direction histogram illustrating observations at Rustington (RST) waverider buoy. Filled bins are wave heights (m)

The 10 largest storm events were selected from these observations for the calibration. The calibration events and associated wave conditions are shown in Table 6-1. The wave model was calibrated by simulating these ten large storm events and driven by boundary conditions supplied by the Met Office WWIII hindcast model data. These events are driven by offshore waves from directions between 169° and 214° . Half of the storm events were not in the range of the BKB wave buoy recordings and so were not used in the calibration with this buoy. The storms that were (events 1,2,3,7 and 8) are reported.

Table 6-1: The 10 largest storm events as recorded by the three CCO wave buoys included in this report

Event no.	Date and Time (GMT)	Peak significant wave height (m)	Peak period (s)	Mean period (s)	Peak direction (Degrees from N)
1	23-Nov-2009 13:00:00	3.83	10.5	6.7	203
2	14-Nov-2009 19:30:00	3.70	13.3	7.4	204
3	25-Nov-2009 03:00:00	3.62	8.3	6.3	212
4	23-Jun-2004 15:00:00	3.90	11.1	6.6	214
5	02-Dec-2005 17:00:00	3.53	8.3	6.3	169
6	13-Dec-2011 01:00:00	3.77	9.1	6.3	187
7	08-Nov-2010 11:00:00	3.86	8.3	6.3	180
8	13-Dec-2008 10:00:00	3.64	7.7	6.0	169
9	18-Jan-2007 13:00:00	3.58	10.0	6.3	204
10	03-Dec-2006 08:00:00	3.42	8.3	6.0	184

Model performance was assessed by comparing the simulated and recorded wave heights at the wave buoy. The storm events listed were simulated using the wave model for nine different model setups. Each setup consisted of a unique permutation, formed by selecting one of three separate physics schemes (Janssen, Komen, Westhuijsen) and one of three separate bed friction schemes (Collins, Madsen, JONSWAP). Default friction coefficients were used for each of these schemes (cfw=0.015, kn=0.05, cfjon=0.067 respectively). Calibration was performed by selecting the wave

model setup that most accurately simulated the observed wave heights at the buoy. The model boundary wave conditions were selected from the hindcast wave model data for the same storm event; seven different times were run for each event (i.e. 3 hours either side of the storm) to ensure the storm was captured from the hindcast data.

From experience gained in calculating wave overtopping in recent studies, the parameter wave height was considered to be the most important in terms of the calibration process, given that it is the most important factor controlling wave overtopping. The peak wave height observed during each of the ten storm events along with the peak modelled wave height (the maximum was selected from the three hours nearest the observed storm), from each of the nine model set-ups were reviewed to find the best performing model set-up across the three observation sites. All three extraction locations must be taken into careful consideration; the model might accurately predict the event at one location but under estimate it at another. For some events the model is under or over predicting the wave height, particularly events 7 and 8. Given the relative good performance of the wave model overall, it is possible that the reduction in performance for these two events is due to inaccuracies in the boundary conditions, or inaccuracies in the observations. It should be noted that the boundary conditions for the wave model are themselves modelled values, from the WWIII hindcast data. Therefore, any inaccuracies in the hindcast model are carried through to the nearshore wave transformation model.

The Root Mean Square Error (RMSE)¹³ was calculated for wave height for all nine model setups and was used to assess the magnitude of the error for each model set-up. All model set-ups exhibit a RMSE range from 0.47 to 0.74m. The best performing set-up uses the Komen physics and JONSWAP (Joint North Sea Wave Project) bed friction schemes. This set-up is abbreviated to KOM-JON. Further analysis comparing the RMSE enables a better examination of the deviation of this model set-up from the observed values. Table 6-2 summarises the RMSE and associated error extracted from the SWAN model at the location of the three waverider buoys from the KOM-JON set-up for the 10 calibration storm events. As a comparison, the calibration results are presented alongside the Met Office WWIII model. The SWAN model has the lowest mean error at 13% of Hs compared to the WWIII model that has a mean error of 20%.

Table 6-2: Met office WWIII wave model compared to the SWAN model significant wave height results from the KOM-JON simulations; the final setup decision based on the RMSE and lowest % error value achieved (mean = 0.13)

Waverider Buoy	Calculation	Met Office WWIII model Hs (m)	Peak sig. wave height (m)	Mean period (s)	Peak direction (Degrees)
HGI	RMSE	0.55	0.41	1.15	8.04
	Average parameter	2.82	3.27	6.02	185
	% error	20	13	19	4
BKB	RMSE	1.04	0.55	0.96	9.82
	Average parameter	2.82	3.30	6.42	196
	% error	37	17	15	5
RST	RMSE	0.37	0.43	0.62	6.74
	Average parameter	3.79	4.04	6.30	205
	% error	10	11	10	3

It should be noted that the performance of the wave model in the shallow areas of the nearshore is most important, as this is where the model outputs will be used in wave overtopping calculations. In the nearshore region the effects of depth limitation on the waves becomes much more pronounced and the range in the predicted wave heights from the nine mode set-ups is much smaller than the range in predicted wave heights in the deeper water at the wave buoys. However,

¹³ This is a measure of the model error over a number of simulations. The use of the square of the error is preferred to use of the error. This is best shown by example: if model A has an error of +1, and model B has an error of -1, the mean error is 0, i.e. no model error. However, the RMSE reports the true error of 1.

there are no observations of wave conditions in these shallow areas to calibrate the nearshore performance of the model.

6.2.1 Calibration: Winter 2014 Storms

The initial model calibration took place in 2013 prior to the winter storms of 2013/14. Following the significant storms of that winter there was a desire to re-run the calibration to validate the model performance against these events. The wave buoy data were examined for the period between 30/01/14 to 17/02/14 as this period included four of the largest storms of the 2014 winter. The CCO wave buoy at Rustington was chosen to highlight the extremes in wave height, period and direction observed during this time and the data is summarised in Table 6-3.

Table 6-3: The four largest storm events from 2014 winter as recorded by the Rustington CCO wave buoys included in this report

Event no.	Date and Time (GMT)	Peak significant wave height (m)	Peak period (m)	Peak direction (Degrees from N)
1	31-Jan-2014 23:00:00	3.93	7.7	204
2	05-Feb-2014 17:00:00	4.72	8.3	194
3	12-Feb-2014 16:00:00	4.18	9.1	197
4	14-Feb-2014 23:00:00	4.97	11.1	215

The performance at the Hayling Island and Rustington wave buoys against the four February storms is summarised in Table 6-4. No data was available for comparison at the Bracklesham buoy.

Table 6-4: Validation of wave model against winter 2014 storms

Waverider Buoy	Date	Observed wave height Hs (m)	Model wave height (m)	Difference (m)	Observed wave period (s)	Model wave period (s)	Difference (s)
HGI	01/02/2014	2.76	3.18	0.42	5.5	6.7	1.2
	05/02/2014	4.13	4.41	0.28	7.1	8.6	1.5
	09/02/2014	3.50	4.11	0.61	6.3	7.6	1.3
	14/02/2014	4.07	4.82	0.75	6.9	8.4	1.5
	RMSE			0.55			1.4
	% error			16			22
BKB	n/a						
RST	01/02/2014	3.93	3.50	-0.43	6.2	7.1	0.9
	05/02/2014	4.72	5.14	0.42	7.1	8.8	1.7
	09/02/2014	4.18	4.60	0.42	6.8	7.8	1.0
	14/02/2014	4.97	5.22	0.25	7.7	8.6	0.9
	RMSE			0.39			1.1
	% error			10			17

Overall the model performance against the 2014 storms was similar to the calibration against the largest events from the gauged records prior to 2014. The overall RMSE against the 14 events was 0.46m, 0.42m and 0.54m at the Hayling Island, Rustington and Bracklesham, RST and BKB respectively which equated to a percentage error of 13%, 10% and 15% for the significant wave height at the three locations. This is considered to be a good performance given that the model is forced at the offshore boundaries by other model data and it was noted that the WWIII outputs also struggled to match the extremes of the observed data at the locations of the wave buoys.

6.2.2 Model simulations

Following the model calibration the wave transformation model was used to pre-compute the potential outcomes of the many combinations of the key storm driving variables. These pre-computed simulations were used to develop the automated look-up tables which can be interrogated to return the nearshore wave conditions. To be effective, it was important that the

pre-computed simulations undertaken cover the likely range of conditions that could occur, extending from the onset of overtopping to major events. A range of water levels, offshore wave heights, offshore wave periods, offshore wave directions and wind speeds and directions were simulated, as presented in Table 6.5. This represented a total of 31,752 ensembles. This included: 9 sea-levels, ranging from approximately the Mean High Water level to above the 0.1% AEP extreme sea-level; wind speeds ranging from 1 to 33m/s, and wind directions from 3 sectors at 45 degree intervals (from 135 to 225 degrees).

Table 6-5: Model simulations undertaken using the wave transformation model

Sea-Level (mAOD)	Offshore wave height (m)	Offshore wave period (s)	Offshore wave direction (Degrees)	Wind speed (m/s)	Wind direction (Degrees)
2.50	0	0	135	0	90
2.75	1	4	180	3	135
3.00	2	8	225	8	180
3.25	4	12		14	225
3.50	6	16		21	
3.75	8	20		28	
4.00	10			33	
4.25					
4.50					
Note: all combinations of the above variables were simulated in the wave transformation model ensembles representing 31,752 discrete events					

In addition to the ensemble simulations the wave transformation model was also used to simulate the 516 joint probability scenarios to represent the conditions required for the AEP event simulations.

6.2.3 Model outputs

For each ensemble simulation, nearshore wave characteristics were extracted from the model at the assumed toe of each flood defence. This follows standard practice for wave overtopping modelling. At each defence toe location the following nearshore wave characteristics were output:

- significant wave height, H_s
- spectral wave period, $T_{m-1.0}$
- peak wave direction, Dir.

This information has then been used for the wave overtopping modelling, as described in the following section.

6.3 Wave overtopping model validation

There were no available data to quantitatively validate the wave overtopping calculations. Instead the validation was performed through an analysis of the frequency of the wave overtopping estimates when calculated using available hindcast model data and the performance was also checked through the calibration of the wave transformation model, wave overtopping discharges and flood inundation extents from the December 2013 and February 2014 storms. The first stage of this analysis was to calculate the wave overtopping discharges for the 31,752 ensemble simulations completed using the wave transformation model. This enabled the creation of look-up tables from which the forecast offshore conditions could be related to the predicted nearshore wave overtopping discharges.

6.3.1 Frequency analysis of wave overtopping

Wave overtopping modelling is inherently uncertain and very subtle changes in the schematisation of a wave overtopping model can have a significant impact on predicted overtopping discharges. Given the absence of quantitative data upon which to formally calibrate wave overtopping models, a more creative approach must be taken to ensure that the calculations are as reliable as possible.

For this study, the performance of the wave overtopping models was evaluated using a long-term performance method undertaken within specialty software developed by JBA and Plan B called *ForeCoast® Flood*.

Hindcast model data from the Met Office WWIII model were used to provide time-series of wave heights, wave periods, wave directions and wind speed and direction for a 34-year period spanning from 1980-2014. Hindcast surge model data from the National Oceanography Centre (NOC) CS3X surge model was stitched together with surge data from the European 2G wave model (1990-2000) and the UK Waters Wave model (2000-2008) to provide a continuous time-series of surge predictions from 1990-2014. Astronomical tidal predictions from the Newhaven and Portsmouth Class A tide gauges were extracted for the same period as the surge data and then combined with the surge model data to produced continuous time-series of total sea-levels from 1993 to 2012.

Using the time-series of sea-level, wind and wave data and the look-up tables generated from the ensemble modelling, *ForeCoast®* calculates a continuous time-series of wave overtopping discharges for the hindcast period at each of the schematised defences. It should be noted that the wave overtopping profiles were schematised based on recently surveyed beach profiles and over the 20-year period there would have been a lot of variation in the beach shape and the subsequent overtopping rates. Despite this uncertainty, the time-series of wave overtopping discharges enables a comparison of the wave overtopping rates against major flood events in the period and allows an analysis of the frequency and predicted seasonality of wave overtopping to allow sensibility checks on the performance of the wave overtopping models.

Optimisation of the wave overtopping models was undertaken using an iterative process. Rather than reporting the frequency of predicted overtopping rates for all 84 defences Table 6-6 summarises the frequency of overtopping at representative defence sections for each community along the coast. First, the representative defence sections in each community¹⁴ were assessed using *ForeCoast®* to determine how many times they would have triggered based on published "tolerable discharge" overtopping rates from the Eurotop Manual (2007) for the safety of unaware pedestrians (0.03L/s/m, considered a precautionary limit), for aware pedestrians (0.1L/s/m), and for the lower and upper limit for trained staff (1 and 10L/s/m respectively). To give further detail the trained staff category was broken down into a lower limit, a mid-range and an upper limit. This equates to fixed wave overtopping thresholds of 0.03, 0.1, 1, 2 and 10L/s/m (0.00003, 0.0001 and 0.001, 0.002 and 0.01m³/s/m). The results of this analysis can be seen in Table 6-6.

Table 6-6: Number of overtopping discharges that exceed published EurOtop thresholds for pedestrians and trained staff

Location	EurOtop Unaware Pedestrians 0.03L/s/m (number and annual frequency)	EurOtop Aware Pedestrians 0.1L/s/m (number and annual frequency)	EurOtop Trained staff lower limit 1L/s/m (number and annual frequency)	Mid range 2L/s/m (number and annual frequency)	EurOtop Trained staff upper limit 10L/s/m (number and annual frequency)
Climping	16 (0.8)	1 (0.05)	0 (0)	0 (0)	0 (0)
Elmer	3 (0.15)	0 (0.00)	0 (0)	0 (0)	0 (0)
Middleton	24 (1.2)	0 (0.00)	0 (0)	0 (0)	0 (0)
Felpham	190 (9.5)	29 (1.50)	0 (0)	0 (0)	0 (0)
Bognor	1025 (52)	128 (6.50)	0 (0)	0 (0)	0 (0)
Aldwick	1232 (61)	149 (7.50)	0 (0)	0 (0)	0 (0)
Pagham	953 (47)	163 (8.00)	0 (0)	0 (0)	0 (0)
East Selsey	1324 (66)	608 (30.00)	0 (0)	0 (0)	0 (0)
West Selsey	538 (27)	21 (1.00)	0 (0)	0 (0)	0 (0)
West Sands Caravan Park	365 (18)	15 (0.75)	0 (0)	0 (0)	0 (0)
Bracklesham	22 (1)	1 (0.05)	0 (0)	0 (0)	0 (0)
East Wittering	9 (0.5)	1 (0.05)	0 (0)	0 (0)	0 (0)
Notes: Number relates to the number of exceedances over the 20-year period and the annual frequency is the average number of exceedances per year over the same period					

¹⁴ Representative defence sections were chosen as those that experience significant overtopping and these tended to be the defences with the lowest crest.

Wave overtopping is inherently uncertain and in particular the prediction of the onset of overtopping and low discharge overtopping is highly uncertain. Validation for overtopping is rare and the tools used to calculate overtopping are largely based on the results of laboratory experiments rather than field data where additional complexities of an individual foreshore, beach, defence, angle of waves, wind speed and many other factors all combine to increase the complexity. Confidence in the occurrence of overtopping increases as the magnitude of the event increases.

The model results reflect this uncertainty and indicate that although there were many occasions when small amounts of splash overtopping were predicted, there were no defences where significant wave overtopping exceeding 1-10L/s/m were predicted. This is in line with the recent flood history as there are very few records of wave overtopping or flooding along the coast; the few records of flooding that were reported in the study area and surrounding sites, are documented below:

- 02/11/1967 overtopping in Ferring, Littlehampton and West Sands Caravan Park
- 14/10/1976 flooding on promenade at the Mill Pond Emsworth
- 14/12/1981 Hayling Island flooded
- 09/04/1985 A27 flooded due to high tides and strong winds
- 26/08/1992 overtopping at Ferring, Littlehampton, overtopping at Lancing, Worthing
- 18/12/1997 overtopping at Climping
- 02/01/1998 overtopping was reported at Selsey/Bracklesham
- 29/01/2001 flooding of roads from high tides at Slipper Mill Pond, Emsworth
- 03/11/2005 overtopping/ failure of defences at Bracklesham/Medmerry, overtopping on Hayling Island frontage. Flooding at Slipper Mill Pond, Emsworth
- 10/03/2008 storm surge caused widespread flooding around Selsey, damage to 500 caravans at Medmerry and road flooded at Hermitage Bridge Emsworth.

For the events where wave overtopping was reported these were checked in the wave overtopping hindcast time-series to check whether wave overtopping was predicted for those events in the correct locations. For each of the reported events the ForeCoast® module predicted overtopping at the correct locations, with overtopping rates as follows:

- 18/12/1997 overtopping at Climping of 0.02L/s/m predicted.
- 02/01/1998 overtopping at Selsey of 0.20L/s/m predicted
- 29/01/2001 still water flood event
- 03/11/2005 overtopping at Bracklesham of 0.09L/s/m predicted
- 10/03/2008 overtopping at Selsey of 0.005L/s/m predicted.

The more recent winter storms were out of the range of the hindcast data that were available at the time of the model checking. As data became available later in the project, verification of the wave overtopping models was completed using individual simulations of the wave transformation, wave overtopping and flood inundation models using event data from the December 2013 and February 2014 events. This is described in the following section.

6.3.2 Calibration of wave overtopping models using flood inundation model

A further check was completed on the performance of the wave overtopping models by using the hindcast model data to force the wave transformation model with conditions from the 5 December 2013, 3 January 2014 and 14 February 2014 storm events. The resultant nearshore wave data was used to calculate wave overtopping discharge rates over the duration of the storm events for all water levels within the tidal time-series. Within the SWAN model the nearshore wave conditions were calculated to coincide with the time of the peak tide and surge. The wave overtopping timeseries were then calculated using the same wave conditions along with the varying water levels from the tidal graph. The calculated wave overtopping was then input into the flood inundation model along with the boundary conditions of still-water levels for those events and the resulting flood extents were compared to the reports of flooding.

During the three events, although conditions were severe and amongst the highest recorded levels from the nearby tide gauges, there were no reports of flooding to properties along the coastal frontage. The first calculations of the wave overtopping predicted significant wave overtopping rates of up to 80L/s/m which resulted in significant flooding when simulated within the flood

inundation model. To reduce this flooding in line with expectations from the event, an iterative process of re-schematising the defences to check that appropriate surveyed profiles were being used and making alterations to the profile parameters. For many of the shingle beaches an appropriate crest width had not been schematised within the profiles. A wide crest on a shingle beach is porous and allows water to infiltrate rather than overtop to cause flood risk behind the defence. Many of the defences were tweaked to adjust the roughness, increase the crest and berm widths and the associated up and down slope angles until the wave overtopping was reduced to acceptable levels. It was important to keep the overtopping profiles within range of the acceptable structures within the Neural Network CLASH database to enable overtopping predictions to be made to populate the overtopping look-up tables. If the profile goes out of range, no overtopping is reported, however severe the conditions. The profiles were iteratively amended to reduce the amount of overtopping. Despite several iterations of the profile schematisations, the final profiles still predicted wave overtopping that resulted in flooding behind the defences in the TUFLOW models for the two calibration events.

Within the TUFLOW models, no infiltration or drainage is accounted for in the model, therefore all wave overtopping contributes to flooding. To remove splash wave overtopping which would be unlikely to contribute to flooding and to allow for a small amount of infiltration in the TUFLOW models, any predicted wave overtopping less than 1.5L/s/m were removed. This reduced the overtopping to produce calibration extents that matched the observed conditions from the three winter storms.

6.4 Flood inundation model calibration

As discussed above the flood inundation models were calibrated against the two severe storms of 6 December 2013 and 14 February 2014 and they were also tested against the event from the 3 January 2014. These events were chosen as they were large events with corresponding data available from other nearby gauges at Pagham, and Cambermet.

To compare the performance of the model to the observed data, model output points were added at the locations of the recorded data. For the Arun to East Head model the tidal boundary was created from the observed data records at Littlehampton with distance-weighting applied to the levels along the coast between Littlehampton and Portsmouth. The performance of the model was compared with the observed data at the Pagham gauge.

For the Chichester Harbour model the tidal boundaries were again created from the observed data from the Littlehampton and Portsmouth gauges and the model performance was compared at the Cambermet gauge.

6.4.1 Arun to East Head model

The performance of the Arun to East Head model against the observed data at Pagham was good. Given the uncertainties associated with the reliance on distance weighting to create the tidal boundary and the uncertainties associated with Pagham gauge the RMSE of 0.14m is within the desired calibration accuracy of $\pm 150\text{mm}$. The Pagham gauge data appear to have a number of datum shifts over the available period and only the data from 2011 to present appear consistent with the data recorded at the Littlehampton gauge, even within this period there are a few small steps in the data. This was noted in the update to the sea-level estimates for the Southern Harbours and Estuaries study. Within the flood inundation model the output point at the Pagham gauge had to be located downstream of the Pagham Rife structure as there were no fluvial inputs within the model and there were no wet cells for any of the runs upstream of the structure.

The model performance was good for the January and February 2014 events, with the model predicting levels within 50mm of the observed data. For the December event the model under predicts against the observed data. The levels at the Cambermet gauge are well represented in the Chichester model so the under-prediction at Pagham may be due to fluvial flows which are not accounted for in the flood inundation model. Table 6-7 summarises the model performance against the gauged data and Figure 6-4 plots the modelled and observed data for the January 2014 event.

Table 6-7: Summary of model performance against observed data at the Pagham Harbour gauge

Date of event	Pagham observed water level (mAOD)	Modelled water level (mAOD)	Difference (m)
06/12/2013	3.67	3.44	-0.23
03/01/2014	3.18	3.13	-0.05
14/02/2014	3.18	3.14	0.04
RMSE			0.14

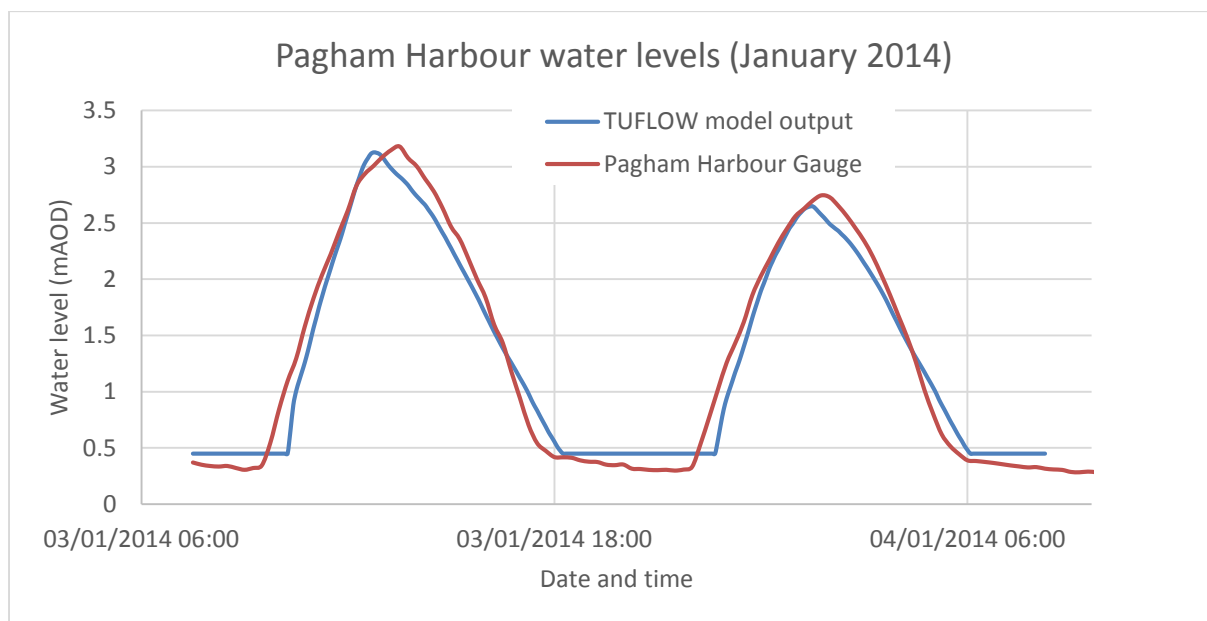


Figure 6-4: Model and observed data from the January 2014 event

6.4.2 Chichester Harbour model

The performance of the Chichester Harbour model against the observed data at the Cambermet tide gauge was good. The model over-predicted against the gauged data during the December 2013 event but was within 60mm for the other two events and the overall RMSE of 0.10m is within the desired calibration accuracy of $\pm 150\text{mm}$. The impact of fluvial inputs were not as important at this gauge and the main uncertainties were associated with the tidal boundary conditions, which were derived through distance weightings of the levels recorded at Littlehampton and Portsmouth. Table 6-8 summarises the model performance against the gauged data and Figure 6-5 plots the modelled and observed data for the January 2014 event.

Table 6-8: Summary of model performance against observed data at the Cambermet gauge

Date of event	Cambermet observed water level (mAOD)	Modelled water level (mAOD)	Difference (m)
06/12/2013	3.12	3.28	0.16
03/01/2014	3.08	3.14	0.06
14/02/2014	3.16	3.12	-0.04
RMSE			0.10

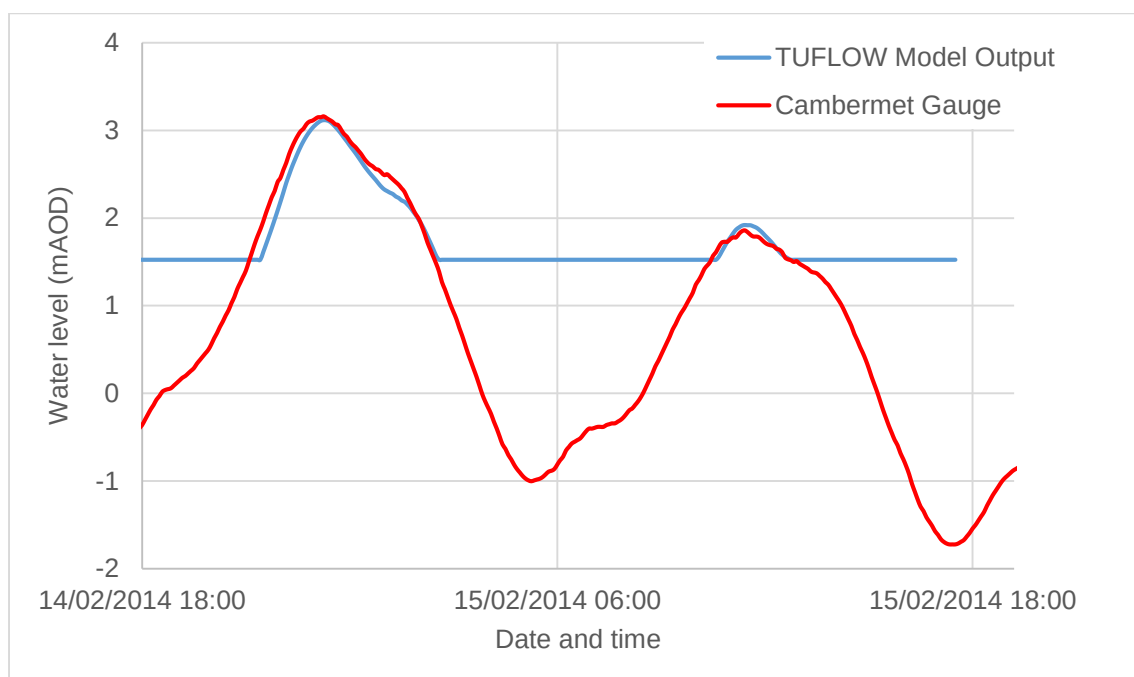


Figure 6-5: Model and observed data from the February 2014 event

6.5 Sensitivity testing

The flood inundation models were run through a series of sensitivity tests to help understand the uncertainty in the model set-up and boundary conditions. The sensitivity tests performed were a test on the model roughness by increasing and decreasing the design modelled roughness values by $\pm 20\%$; a test on the representation of buildings by modelling as stubby buildings raised 0.3m above ground level and as porous buildings by allowing water across the building footprint at ground level but restricting the flow through the building by 90% to account for the physical blockages caused by walls; and a final test was undertaken on the downstream boundary to increase water levels by adding in the uncertainty in the CFBD data by adding the confidence intervals for the 0.5% AEP event.

6.5.1 Sensitivity to model roughness

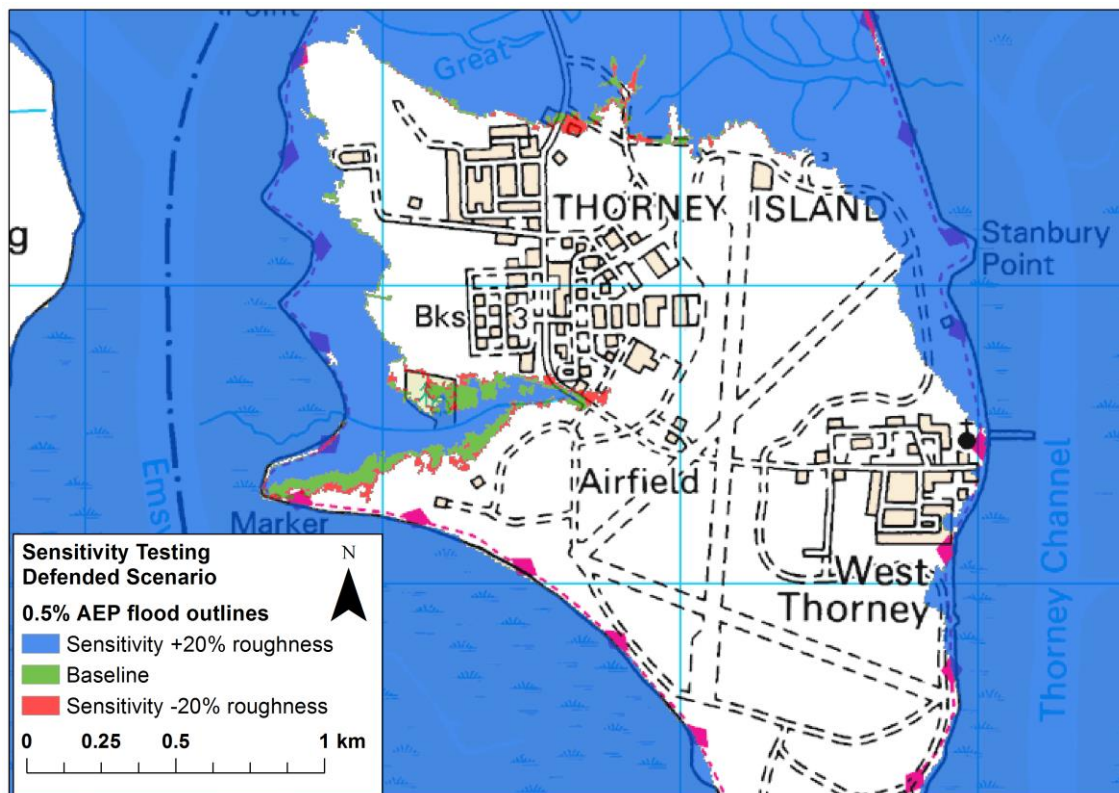
Overall there was very little change to the flood extents when an increase or decrease to the model roughness values were applied. The area with the most noticeable change in extent in the Chichester Harbour model was on Thorney Island where depths of flooding on the floodplain changed by $\pm 0.30\text{m}$ with an increase in roughness resulting in a slightly smaller flooded area and a decrease in roughness increasing the flood extents as shown in Figure 6-6.

For the Arun to East Head model the most noticeable differences were around Pagham harbour with increased flood depths of 0.30m and more extensive flooding for the reduced roughness simulation, see Figure 6-7. Along the open coast there were no noticeable differences in the flood extents.

6.5.2 Sensitivity to representation of buildings

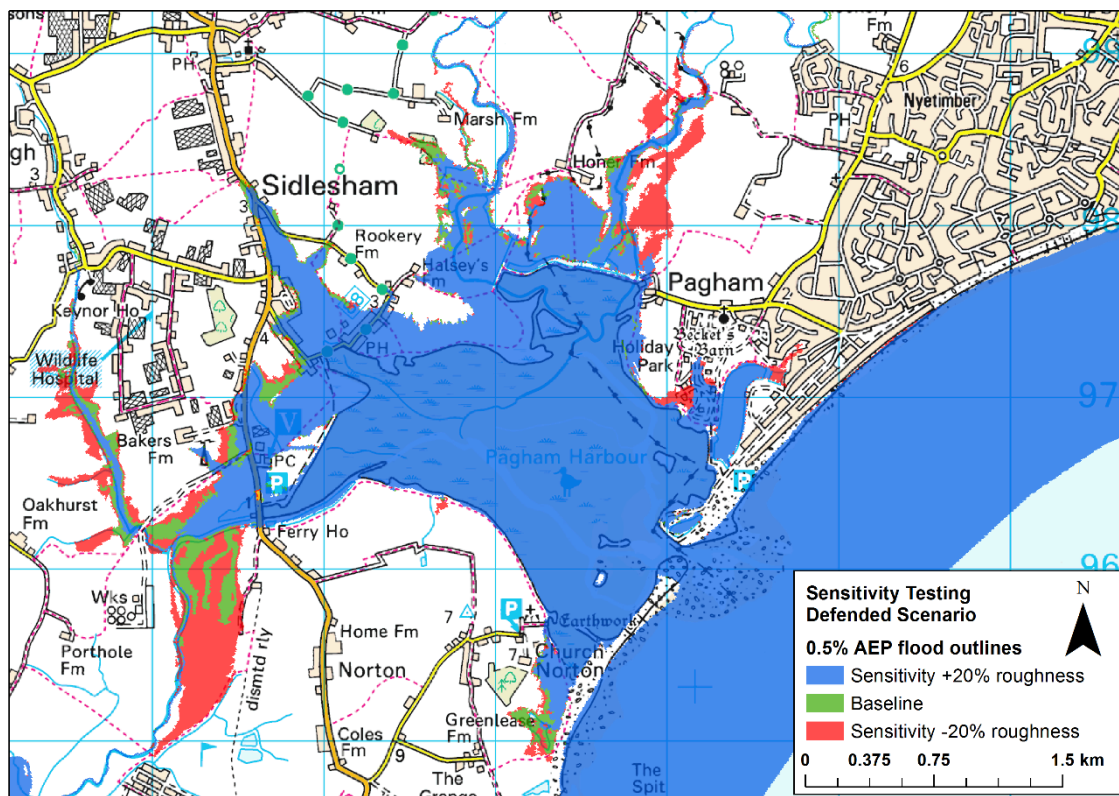
In the design simulations the buildings were represented by areas of increased roughness. For sensitivity testing two alternative approaches were simulated. In the first the building footprints were raised by 0.3m to represent flooding commencing when a threshold is reached in combination with increased roughness, this is referred to as stubby buildings. A separate simulation with buildings represented as flow constrictions to reduce flow by 90% across the model cells containing buildings, this is referred to as the porous buildings approach. Again the flood extents were not noticeably changed. The porous buildings produced flood extents very similar to the design simulations whereas the stubby buildings reduced the flooding to the buildings where flood depths were less than 0.30m. The results for Emsworth are shown in Figure 6-8, note that the design extent (based on increased roughness over the building footprint) is almost identical to the porous buildings and no differences can be distinguished in this area.

For the Arun to East Head model the sensitivity to building representation resulted in a small change in flood depths of $\pm 0.01\text{m}$ and resulted in no noticeable change in flood extents. As the differences were so small, the design approach was to use the increased roughness over the building footprint.



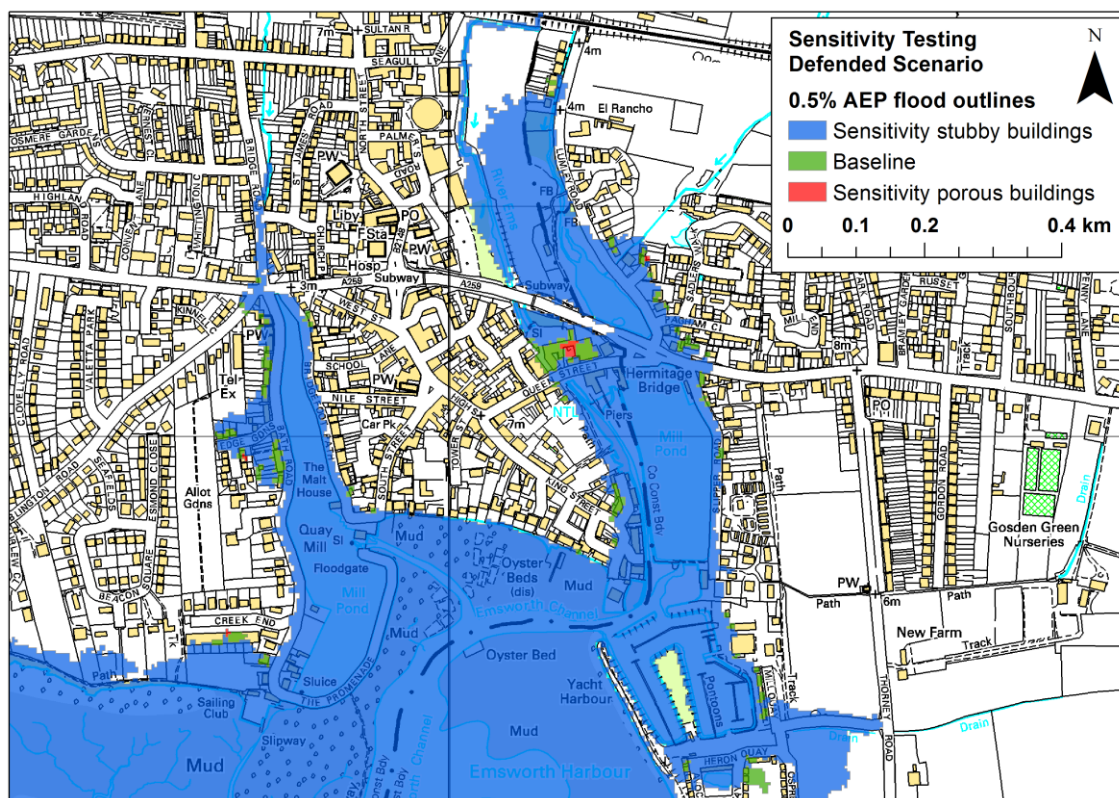
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Figure 6-6: Sensitivity to model roughness in the Chichester Harbour model



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Figure 6-7: Sensitivity to model roughness in the Arun to East Head model



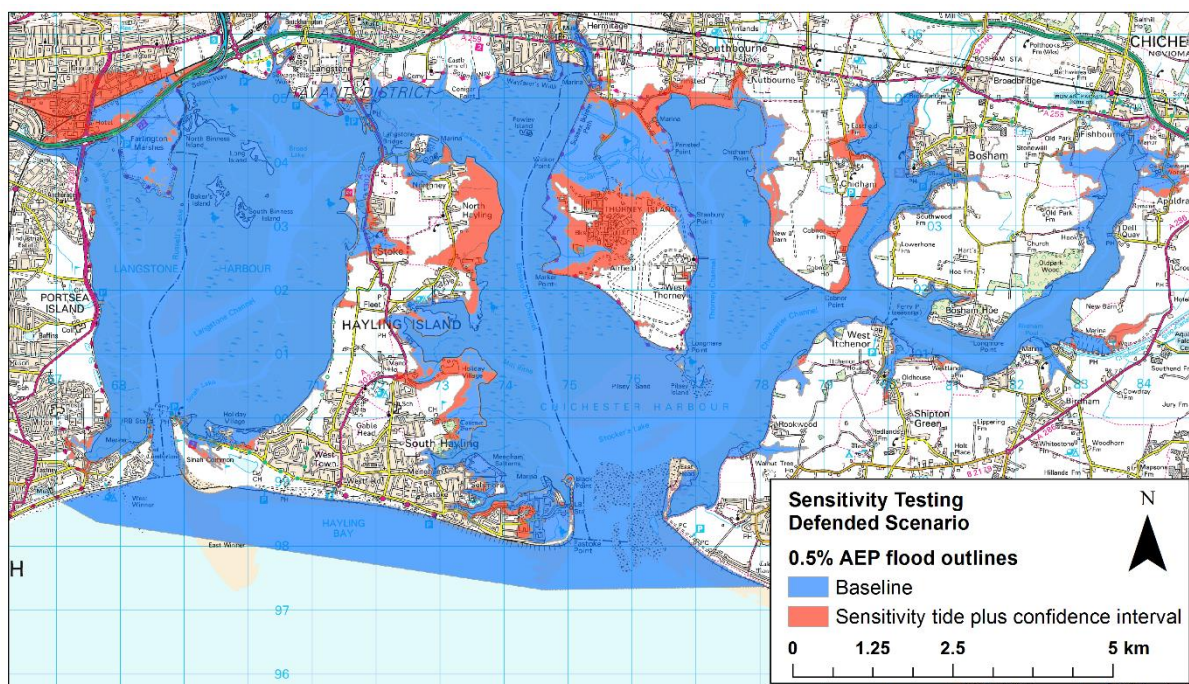
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Figure 6-8: Sensitivity to building representation within Emsworth

6.5.3 Sensitivity to downstream boundary

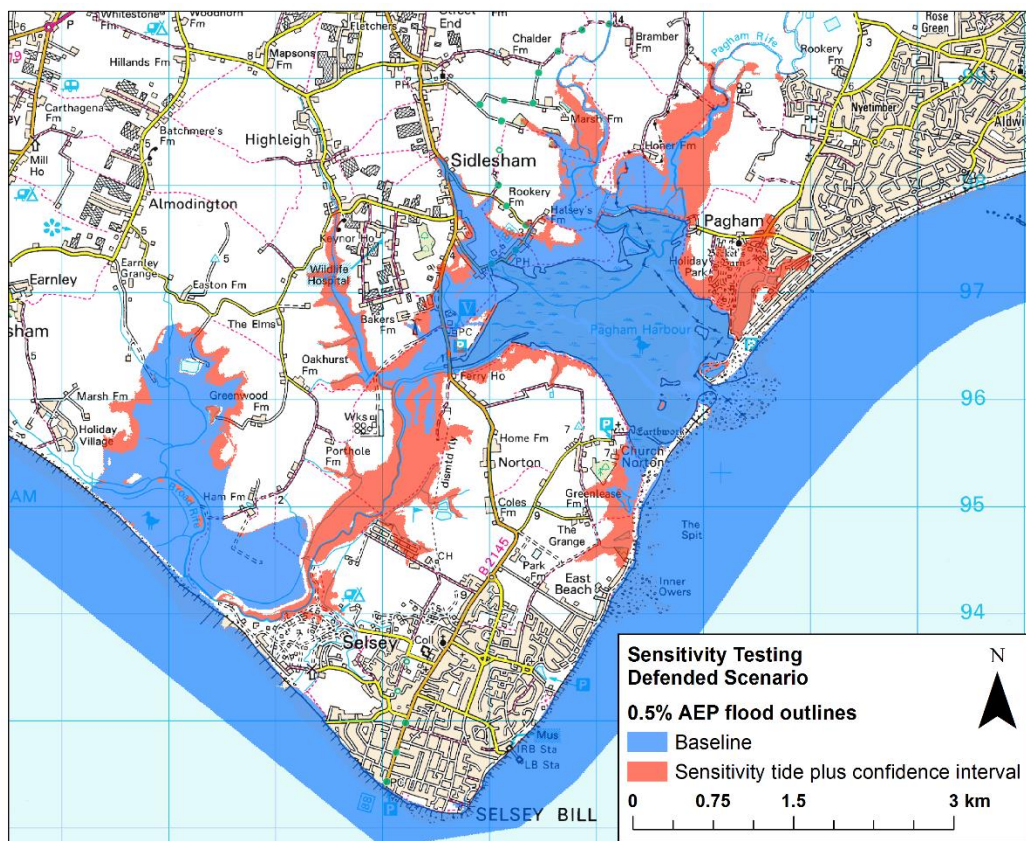
A final sensitivity test performed on the flood inundation model was to test the sensitivity to the downstream boundary. This was tested by adding on the confidence interval values representing the uncertainty in the CFBD sea-levels for the 0.5% AEP event. For the Chichester Harbour model the increased levels along the downstream boundary lead to increased flooding on Hayling Island, Thorney Island and on land to the south of Southbourne, as shown in Figure 6-9.

The Arun to East Head model is also sensitive to changes in the downstream boundary. The increase in levels from the confidence intervals of 0.3m increases the peak level of the 0.5% AEP tide beyond the peak of the 0.1% AEP tide level. Increased flooding is seen in all areas at risk of still water flooding, particularly along the River Arun and at Pagham harbour, as shown in Figure 6-10.



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Figure 6-9: Sensitivity to downstream boundary for the Chichester Harbour model



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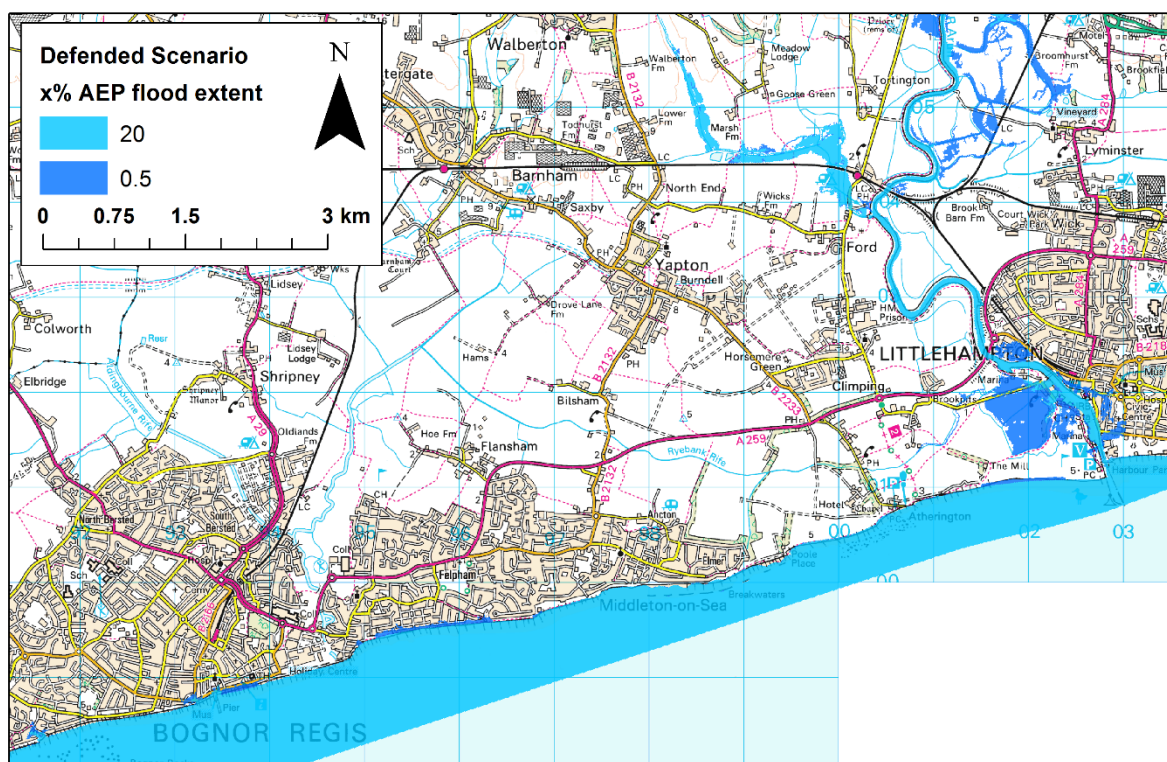
Figure 6-10: Sensitivity to downstream boundary for the Arun to East Head model

7 Results

This chapter discusses the model results and summarises the flood risk in the main urban areas. The study was separated into two models and as such this chapter is divided into two main sections, River Arun to East Head and Chichester Harbour.

7.1 River Arun to East Head model

This model extended from Littlehampton in the east to Berrybarn Lane at West Wittering. The flood extents vary significantly depending on the modelled AEP event. Figure 7-1 shows a comparison of the 20 and 0.5% AEP defended flood extents for the communities of Bognor Regis and Littlehampton. There are clear differences in flood extents between the 20 and 0.5% AEP events, of which the most noticeable are around the lower regions of the River Arun and the coastal communities of Felpham. It can be seen that during the 20% AEP event the defence network mitigates the flood risk with coastal communities protected and flow remains largely in bank on the River Arun. There is however, still water flooding at Rope Walk on the west bank of the Arun during both events. Elsewhere there is minimal flooding during the 20% AEP event, with overtopping present in Bognor Regis by the pier, but limited to the road and at depths of approximately 0.10m.

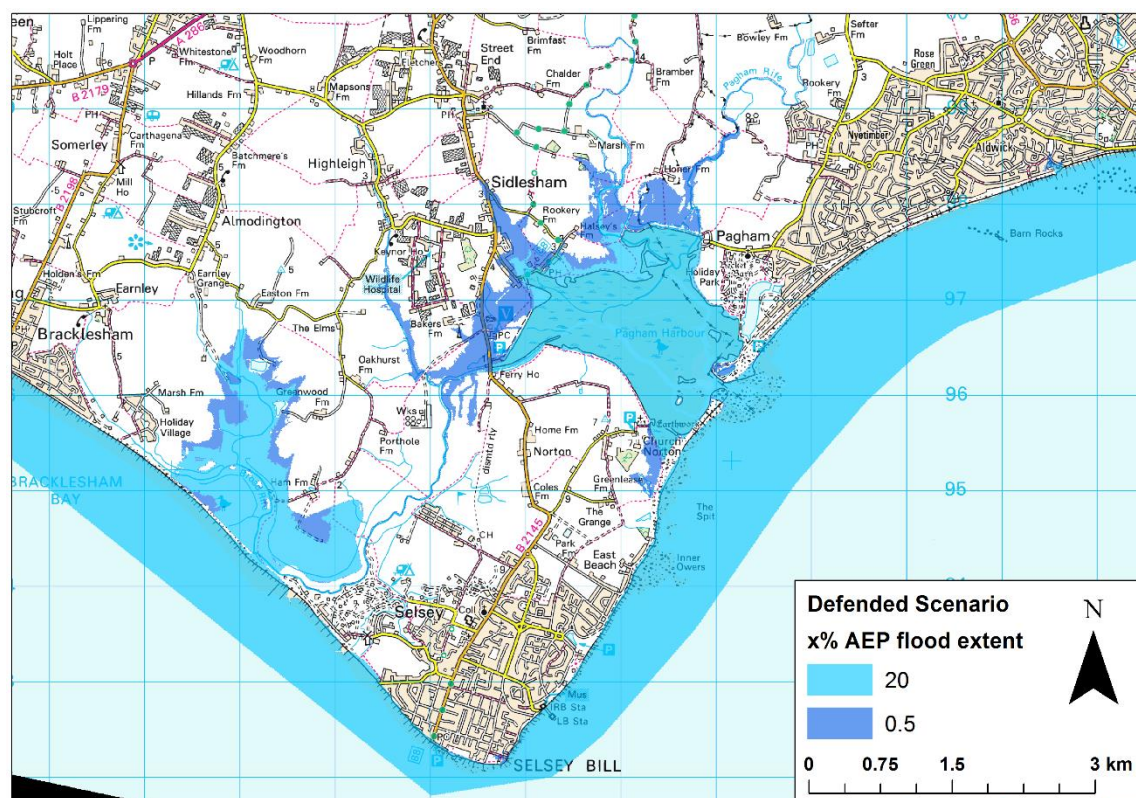


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Figure 7-1: Bognor Regis and Littlehampton flood extents for the 5 and 0.5% AEP defended scenarios

In Aldwick, wave overtopping results in flooding at Dark Lane with depths of 0.05-0.60m. For the 0.5% AEP event the flood depths increase to a maximum of 0.81m, with further properties flooded. Additionally, the flooding around Bognor Regis pier extends further along The Esplanade in both directions, causing flooding of properties to the west of the pier including Connaught House and Tuptim Siam restaurant, with depths of up to 0.50m on the road. The 0.5% AEP event also causes extensive flooding in Felpham, with significant overtopping at Canning Road, and from Blakes Road to Sea Drive. The depths vary along this community from 0.02m to 1.20m at Jacken Close. There is more extensive still water flooding up the River Arun in the 0.5% AEP event, with both west and east banks experiencing flooding. All of Rope Walk is inundated and the industrial estate north of Littlehampton Marina is flooded. On the east bank, flooding extends to properties off South Terrace and on Clifton Road and Bayford Road, as well as further north on River Road.

Figure 7-2 shows a comparison of two events, the 5 and 0.5% AEP defended, for Pagham Harbour, Selsey and Wittering. As with Figure 7.1 the coastal defence network mitigates the flood risk for much of the region, with the 20% AEP event resulting in significantly reduced flooding when compared with the 0.5% AEP event. There is no coastal wave overtopping in this area for the 20% AEP event, with flood inundation limited to Pagham Harbour and Medmerry. The community of Selsey is not affected by the 20% AEP event. Properties are affected in Pagham Harbour due to flooding in the 20% AEP event, with the properties on Mill Lane near the Crab and Lobster Inn affected. For the 0.5% AEP event, the coastal defence network continues to protect much of the area, with only one coastal community affected. The area flooded is south east Selsey, including three properties and the recreation ground, although depths are limited to 0.01-0.10m. The 0.5% AEP event also causes a greater extent in Medmerry although the properties to the east of the realignment scheme remain protected. There is also additional flooding in Pagham Harbour, with part of Church Farm Holiday Village flooded and properties in and around Sidlesham inundated.



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Figure 7-2: Selsey and Pagham Harbour flood extents for the 5 and 0.5% AEP defended scenarios

7.1.1 Undefended model results

The undefended model simulations show increased flooding for all AEP events when compared with the defended equivalents. During the 0.5% AEP event, there is a significant increase in flooding throughout the model. East Wittering remains largely protected, with flooding limited to the first line of properties nearest the coast. Sussex Beach Holiday Village is entirely inundated during the undefended simulation together with the farm cottages around the holiday village. West Selsey is also affected, with flooding from Pagham and Medmerry having an impact on the area. During the 0.5% AEP undefended event, Bognor Regis flooding is largely limited to the areas already affected by the 0.5% AEP defended event and some additional areas along the seafront. Areas closest to the rivers are worst affected, including near the Aldingbourne Rife and Elmer. A comparison of the number of properties flooded during the undefended and defended simulations is provided later in this chapter in Table 7-1.

The 5% AEP event has a larger flood extent than the 20% AEP event in Littlehampton West bank along Rope Walk, with additional properties affected by still water flooding. There is also further flooding near Bognor Regis pier due to wave overtopping, with ten extra properties inundated and more flooding at Dark Lane in Aldwick, including an electricity sub-station. Likewise, further properties are flooded around Pagham Harbour. Elsewhere, no additional coastal defences were overtopped.

Overtopping and inundation of properties does not occur in Middleton until the 0.1% AEP event, when houses on the coastal side of Sea Way Road are flooded. In Felpham there is overtopping of Bognor Regis promenade during the 1% AEP event which causes flooding at houses on Jacken Close and Limmer Lane. The promenade along this defence is breached from Jacken Close west to Blakes Road during the 1% AEP event. With increasing AEP events, the flood extent at Dark Lane in Aldwick increases, spreading to more properties and High Trees Road during the 0.2% AEP event. In Selsey, there is limited overtopping of defences. Selsey Bill is the first area to be affected, with properties inundated by the 1.33% AEP event. Elsewhere, flooding is restricted to East Selsey, near Kingsway and the museum, during the 0.1% AEP model simulation.

7.2 The impact of climate change

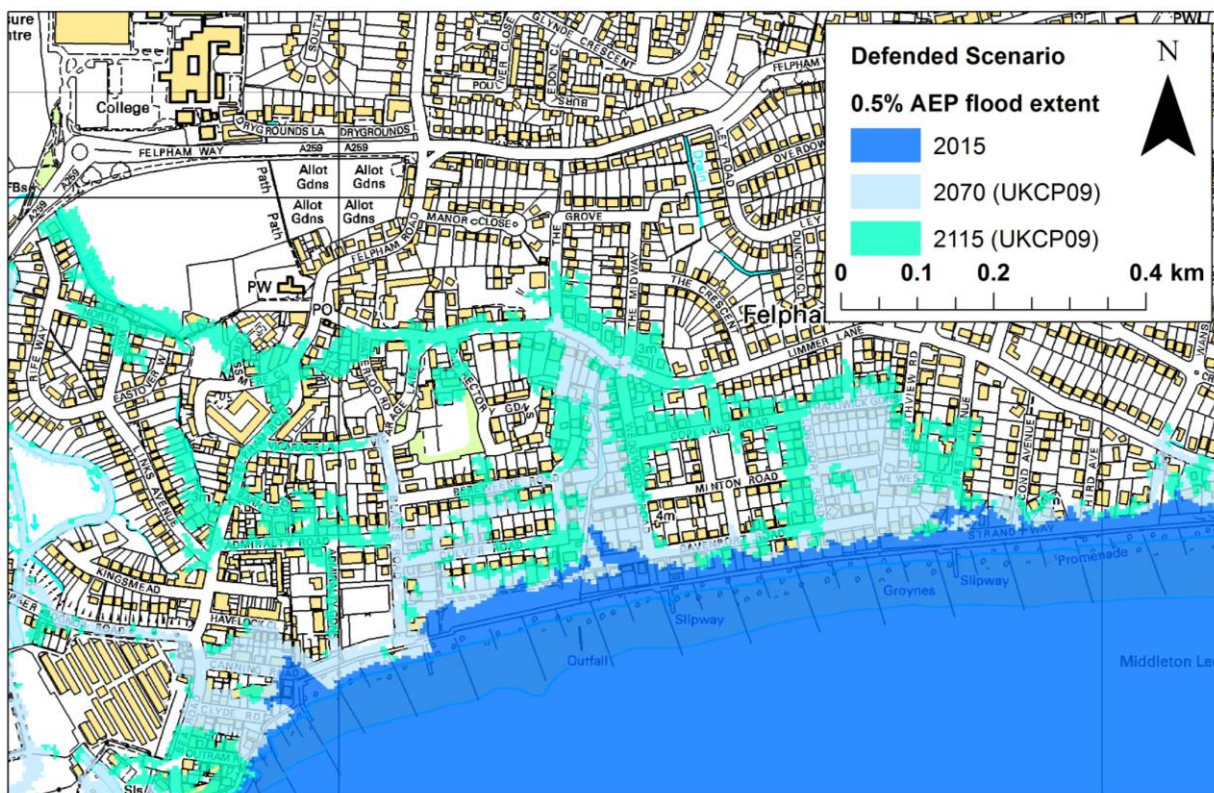
In July 2011, new climate change guidance was released based on the UKCP09 report¹⁵. In addition to present day extreme still water level events, 0.5% AEP climate change scenarios for the years 2070 and 2115 were also modelled based on the 95th percentile medium emissions scenario. The sea-level rise for climate change (from 2014 to 2070) was 0.39m whilst from 2014 to 2115 sea-level rise in the area was 0.78m.

The flood outlines for the years 2070 and 2115 flood similar areas to their present day simulation counterparts, albeit more extensive, with additional flooding in Littlehampton, Bognor Regis, Selsey and Wittering, particularly during the 2115 event. An example that is fairly consistent with the impact that climate change has throughout Arun to East Head can be seen at Felpham. Figure 7-3 shows a comparison of the 0.5% AEP defended outline against the 0.5% AEP defended climate change outlines for both 2070 and 2115. Clearly, sea-level rise due to climate change results in a much larger flood extent in the defended scenario at Felpham. For 2115, the flood outline extends over 350m further inland towards Felpham Way, resulting in many more residential properties flooding.

Littlehampton West Bank has increased flooding extents for the 3.33%, 1.33% and 1% AEP events without any additional properties flooding. The Golf Club floods during the 0.5% AEP plus climate change (2070 UKCP09) event and Climping semi-retirement park floods during the 0.5% AEP plus climate change (2115 UKCP09).

For both 2070 and 2115, climate change for the model was also calculated based on the NPPF guidance. In contrast to the UKCP09 guidance, an increase of 0.52m and 1.13m was calculated for 2070 and 2115 respectively. As would be expected, with this larger increase in still water level, there is more widespread flooding based on NPPF guidance. Much of the coastline is affected, particularly during the 2115 NPPF event. The community of East Wittering is heavily inundated and the coastal properties at Selsey are flooded. Bognor Regis, Felpham and Middleton-on-Sea are also flooded, with breaching occurring at most of the coastal defences. The River Arun flood extents are also large, with flooding extending west to Climping and completely inundating the caravan park. For the largest of the modelled climate change events, over 7,000 properties were shown to be at flood risk.

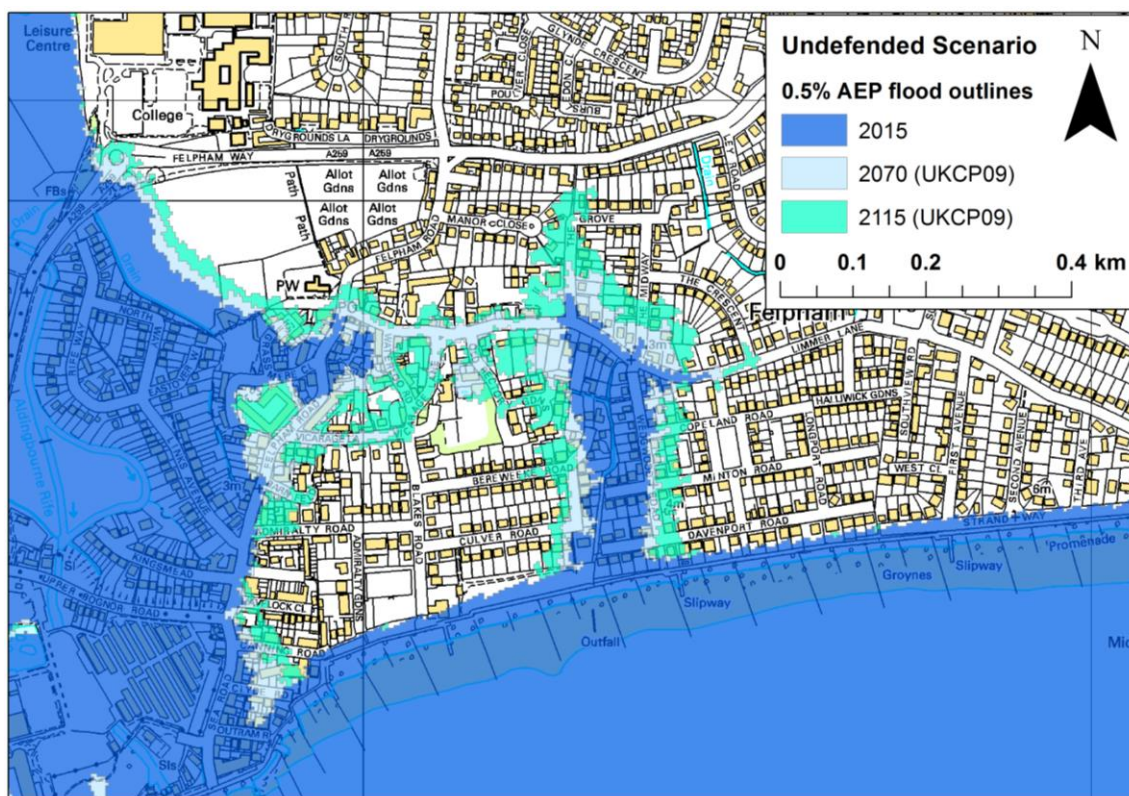
¹⁵ <http://ukclimateprojections.defra.gov.uk/>
2014s0753_Emsworth to Littlehampton_Coastal final report_v2.docx



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Figure 7-3: 0.5% AEP defended and defended climate change comparison at Felpham (UKCP09)

Figure 7-4 shows a comparison of the 0.5% AEP undefended outline against the climate change undefended outlines for both 2070 and 2115 (UKCP09). Much like the defended comparison, the climate change flood extent is much larger. Compared with the defended outline in Figure 7-3 there is reduced inundation in east Felpham due to the lack of defences allowing the wave overtopping to drain back into the sea. Flooding to the west nearer Bognor Regis is significantly increased. During the undefended simulations, this increased extent causes flooding on the A259, Felpham Way, which was not inundated during the defended model scenarios.



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Figure 7-4: 0.5% AEP undefended and undefended climate change comparison at Felpham (UKCP09)

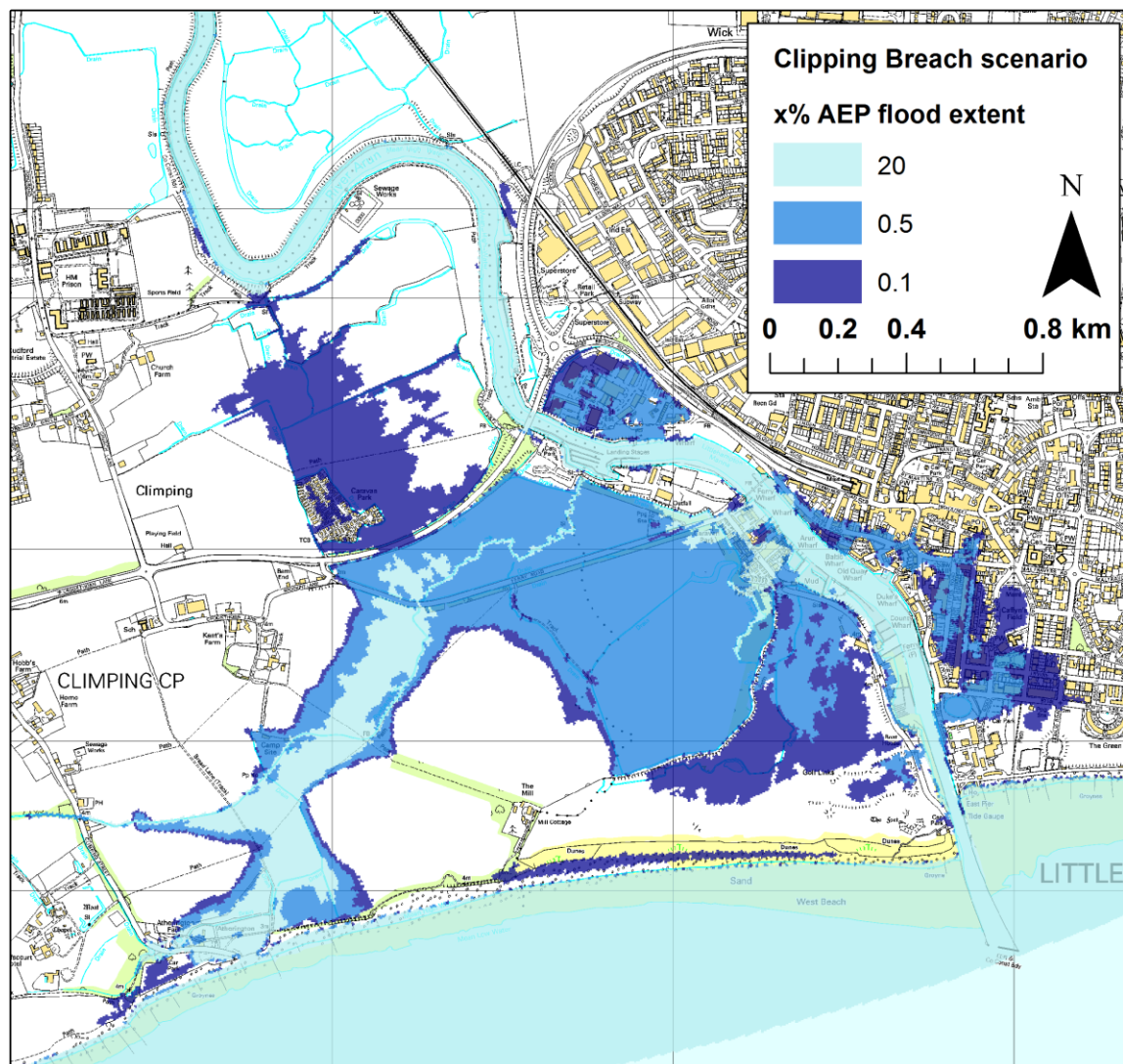
7.3 Breach scenarios

For the Arun to East Head model a number of model simulations were completed with breaches through the coastal defences. The defences were breached in four locations; the locations and breach parameters used were as follows:

- Climping - 50m wide breach through the hard defences near the car park. The breach occurs one hour before high tide and fails over a half an hour period.
- Felpham - 50m wide breach through the hard defences near the pumping station. The breach occurs one hour before high tide and fails over a half an hour period.
- Elmer - 50m wide breach through the hard defences near the outfall of Elmer Rife. The breach occurs one hour before high tide and fails over a half an hour period.
- Pagham - 40m breach through the spit located directly in front of the harbour mouth. The breach is open from the start of the simulation and is eroded down to -1.80mAOD on the coast and interpolating to -0.90mAOD on the harbour side to match the elevation of the channel through the harbour mouth. The material from the breach is used to block the channel between the spit and the coast to an elevation of 5.3mAOD.

The flood risk from each of the breaches was assessed in isolation for the 20, 0.5 and 0.1% AEP events. To reduce the number of model simulations the Pagham and Climping breaches were run within the same model as there is no connectivity between the flood risk areas from the defence breaches.

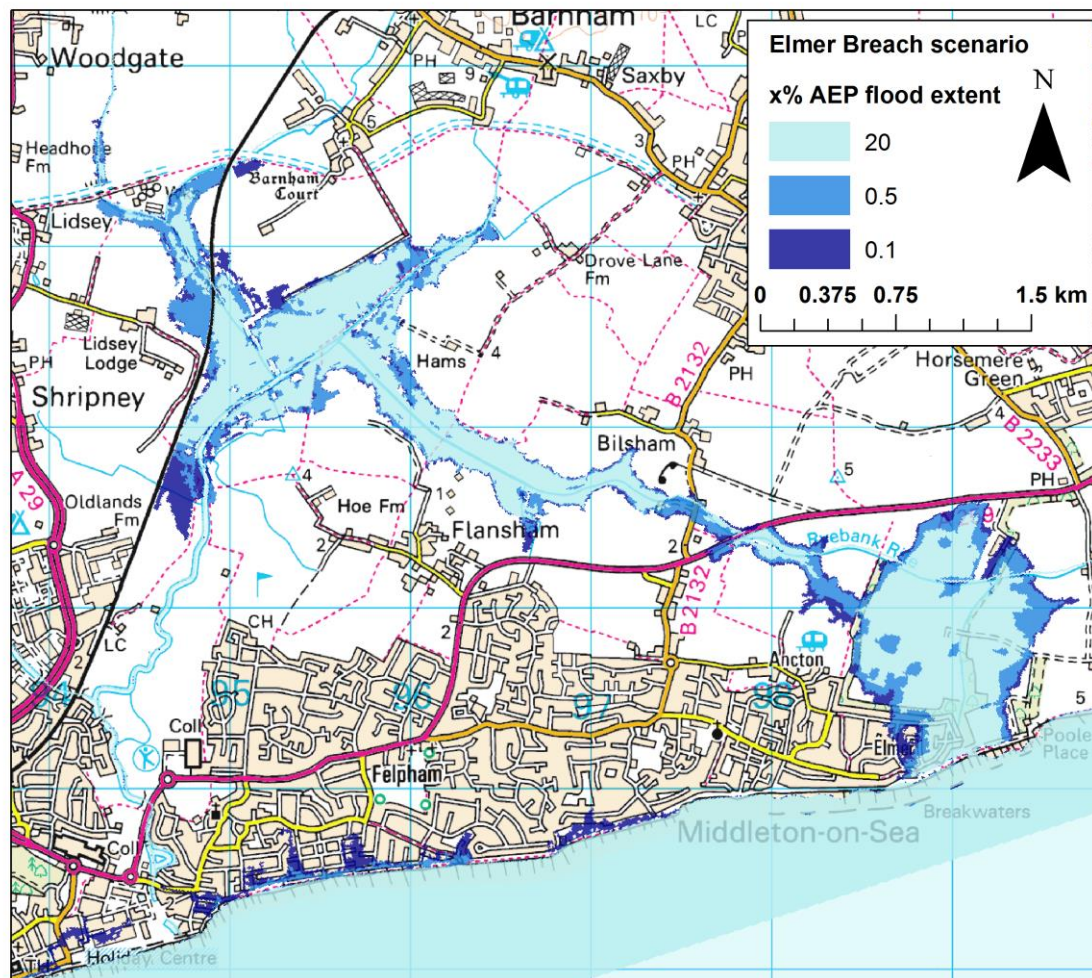
The breach at Climping was initially located on the western side of the car park but due to the high ground behind the defence this only resulted in limited flooding. The breach location was adjusted to tie into lower ground behind the defence near the entrance to the car park and this resulted in more extensive flooding which spread north-east to join the flooded areas from the Arun west bank. Figure 7-5 shows the flood risk from the Climping breach for the 20, 0.5 and 0.1% AEP events.



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Figure 7-5: Flood extents from a breach at Climping

The breach at Elmer initially floods a large area behind the defence including over 100 properties. The flood water then spreads north-west along the channel of Ryebank Rife extending 6km inland during the 0.1% AEP AEP event. Figure 7-6 shows the breach extents for the 20, 0.5 and 0.1% AEP events at Elmer.



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Figure 7-6: Flood extents from a breach at Elmer