The Sizewell C Project

5.2 Main Development Site Flood Risk Assessment
Appendices 1-7 Part 1 of 14

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APPENDIX 1 - COASTAL MODELLING REPORT

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1 INTRODUCTION

1.1 Background

1.1.1. The purpose of this report is to describe the approach and present the results for the coastal modelling of wave overtopping of the sea defences at Sizewell C undertaken to inform the Flood Risk Assessment (FRA) that forms part of the Development Consent Order (DCO) application. This modelling builds on the preliminary overtopping assessments carried out in previous years and follows the same modelling approach as previously agreed with EDF Energy, presented in a Technical Note on 23rd February 2017 (**APPENDIX A**).

1.1.2. The report is intended to update EDF Energy and the Environment Agency on the latest model developments, results and assumptions adopted in the modelling study for the FRA, including updates following Environment Agency's model review.

1.2 Methodology

1.2.1. In the early stages of the coastal modelling study for Sizewell C, a review of methods for estimating wave overtopping was carried out. Key comments and recommendations from the review were:

- Physical modelling – There is no intention to use physical modelling to inform the FRA. However, if required (based on computational modelling results and sensitivity tests), physical modelling of the sea defence may be undertaken at detailed stage of design development to assist in assessing the nuclear Safety Case.
- EurOtop (empirical) – There are a number of empirical methods developed for estimating wave overtopping, derived from laboratory data (i.e. physical modelling). EurOtop is the most updated, well tested and documented empirical method, and was therefore considered for use in this study. As empirical methods are based on laboratory data, they are only valid for defence profiles and wave/water level conditions that are within the tested range;
- EurOtop (neural network method) – The EurOtop neural network model is also based on laboratory data; the same database that the EurOtop empirical methods were derived from. It was considered for use in this study, however, the EurOtop neural network model is also limited to tested defence profiles and wave/water level conditions;
- Hydrodynamic models – Hydrodynamic models offer detailed analysis of wave propagation and overtopping. Considering computational speed, only non-linear shallow water equation models were considered as

appropriate for use in the Sizewell C study. AMAZON is a world-leading well tested non-linear shallow water equation model and was therefore considered for use in this study.

a) Use of AMAZON modelling package

1.2.2. AMAZON is a high resolution, two-dimensional finite volume numerical model capable of simulating supercritical flow and capturing moving hydraulic jumps. It is based on solving the non-linear shallow water equations (Ref 1 and Ref 2). AMAZON was designed for 'violent' (referring to moving flow discontinuity) flows, such as hydraulic jumps, tsunamis, bore waves (including tidal bores) and dam breaks. Because of these strengths, AMAZON has also been used extensively for simulating wave run-up and overtopping.

1.2.3. AMAZON-Wavewatch is a one-dimensional software package with a graphic user-interface (GUI) specifically designed for simulating wave overtopping of coastal structures based on the AMAZON model. AMAZON has been tested for wave overtopping calculations on single slope walls, slope walls with berms, and vertical seawalls (Ref 2).

1.2.4. As input data, AMAZON requires bathymetric information (or cross-section profiles for a 1-D calculation) and incident waves. AMAZON is an unsteady state model so random waves can be simulated as well as monochromatic waves. Popular wave spectra including Bretshneider-Moskowitz, JONSWAP and TMA have been built into the AMAZON software. User-defined wave spectrum and measured wave trains are also accepted by AMAZON for incident wave input.

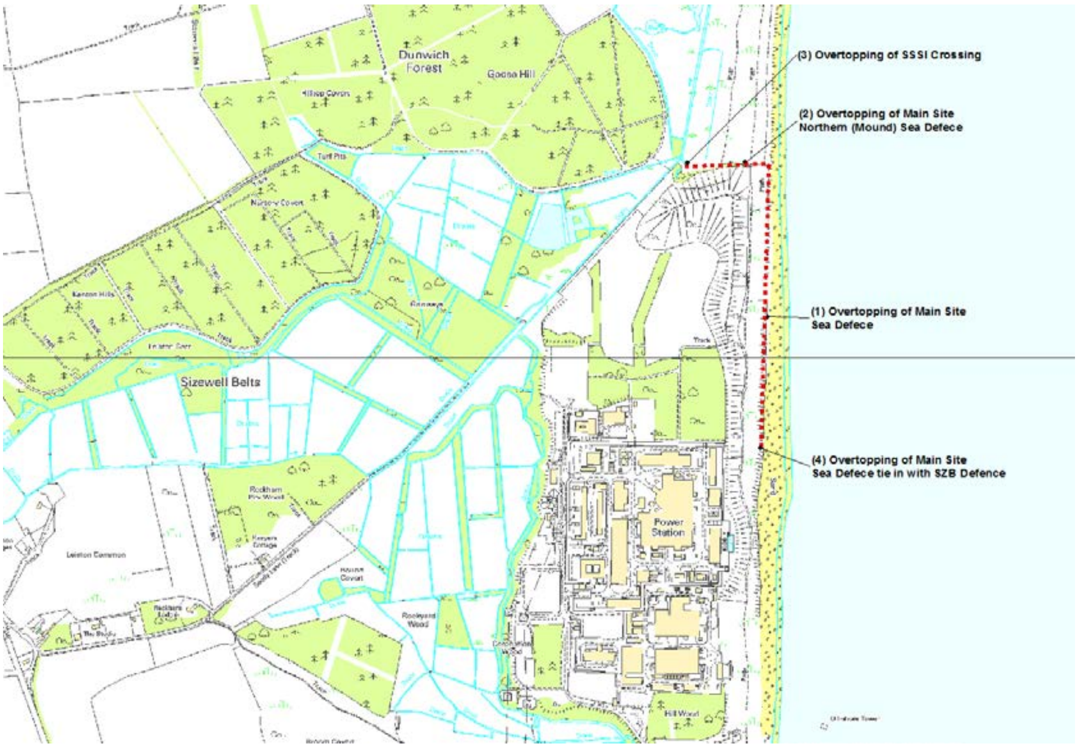
1.2.5. Further information on AMAZON and its suitability for overtopping assessment at SIZEWELL C is provided in **APPENDIX B**: 'Flood Risk Assessment Sizewell C: AMAZON for overtopping prediction', RHDHV 2014.

b) Qualitative comparison of AMAZON with EurOtop for use at Sizewell C

1.2.6. This section provides a qualitative comparison of the use of AMAZON for assessing coastal flood risk at Sizewell C. Some quantitative comparisons have also previously been undertaken on the design defence profile with eroded beach profile. Results of this assessment are described in the technical note issued to EDF in January 2015 (Ref 3).

1.2.7. EurOtop methods are regarded as the UK industry standard for predicting wave overtopping, particularly for 'standard' defence profiles, which have been well tested and are incorporated into the EurOtop database (Ref 4). The primary issue with using EurOtop at Sizewell C is that the defence profiles at SIZEWELL C are not 'standard', as they comprise of shingle beach and vegetated sand dunes. **Plate 1.1** illustrates the four locations where flooding from wave overtopping requires consideration and assessment; none of these locations have ‘standard profiles’.

Plate 1.1: Potential routes for flooding from overtopping



1.2.8. **Plate 1.2** to **Plate 1.4** illustrate the non-standard nature of this coast by showing the different types of profiles that are encountered in front of the proposed Sizewell C, where the development may be exposed to wave overtopping.

Plate 1.2: Existing frontage of the proposed Sizewell C development – shingle beach

Plate 1.3: Existing frontage of the proposed Sizewell C development – sand dunes

Plate 1.4: Existing northern frontage of the proposed Sizewell C development

1.2.9. **Plate 1.2** to **Plate 1.4** demonstrate that there is a substantial shingle beach and vegetated sand dunes at all of the potential flood routes involving wave overtopping at Sizewell C. It is important that those features are considered in the wave overtopping analysis as they may have a significant impact on wave propagation and run-up before waves reach the formal new engineered sea defences, referred to as 'Hard Coastal Defence Feature (HCDF). These defence shapes cannot be properly represented by the EurOtop methods, and therefore AMAZON software was recommended and used for predicting wave overtopping for the Sizewell C coastal flood risk assessment.

1.2.10. Updated quantitative comparison of wave overtopping rates produced by AMAZON and EurOtop were carried as a part of the 2019 works and are presented in **section 2**.

c) Use of CEFAS input data

1.2.11. The TOMOWAC wave model has been developed by Cefas for investigating wave propagation from offshore to nearshore areas. To derive inputs to the wave transformation model, a comprehensive joint probability assessment of offshore significant wave height and extreme still water levels was carried out (Ref 5).

1.2.12. The JOIN-SEA method of calculating the joint probability for combined waves and sea levels was adopted in this study as the methodology recommended by the Environment Agency at the time of the study; published in the "Best Practice Guide – FD2308/TR2" (Ref 6).

1.2.13. It is recognised that other approaches using multi-variate methods (e.g. Heffernan and Tawn) exist, however it was assumed that the Join-Sea method was most appropriate at the time of the study. This would be revised in line with the latest guidance and the joint probability assessment would be updated at the next stages of works.

1.2.14. Outputs from the TOMOWAC wave model were then provided along pre-selected transects along Sizewell frontage for points up to 300m offshore, for the 1 in 200, 1 in 1,000 and 1 in 10,000-year storm events and a range of sea-level rise scenarios. They comprise of nearshore significant wave height (Hs) and peak wave period (Tp) for the peak of the storm (Ref 7).

1.2.15. During the early stages of the assessment, it was found that the JONSWAP wave spectrum used in AMAZON for the offshore wave period has more wave energy in the lower frequency region than the TOMOWAC modelled wave spectrum for the nearshore wave period (extracted from the modelled nearshore wave climate). This means that the Amazon model results using JONSWAP with offshore wave periods and inshore significant wave height are more conservative. Therefore, that approach has been adopted for the assessments. Further details on addressing the uncertainties in TOMOWAC model are provided in **APPENDIX A**, **section 4**.

1.2.16. For the AMAZON wave overtopping model, the nearshore wave heights from the TOMOWAC outputs were applied with associated 'still water' peak tide levels for considered climate change scenarios. By using 'still water' peak tidal input data, the above approach to wave overtopping modelling is conservative compared to a time-varying tidal cycle.

1.3 Previous sensitivity tests

1.3.1. A number of checks have previously been completed using the AMAZON model to determine sensitivity of predicted overtopping rates to beach profile variations and derived nearshore wave heights. Outlined in the following subsections is a short summary of the tests undertaken and results obtained. Further details on the sensitivity test are provided in **APPENDIX A**, **section 5**.

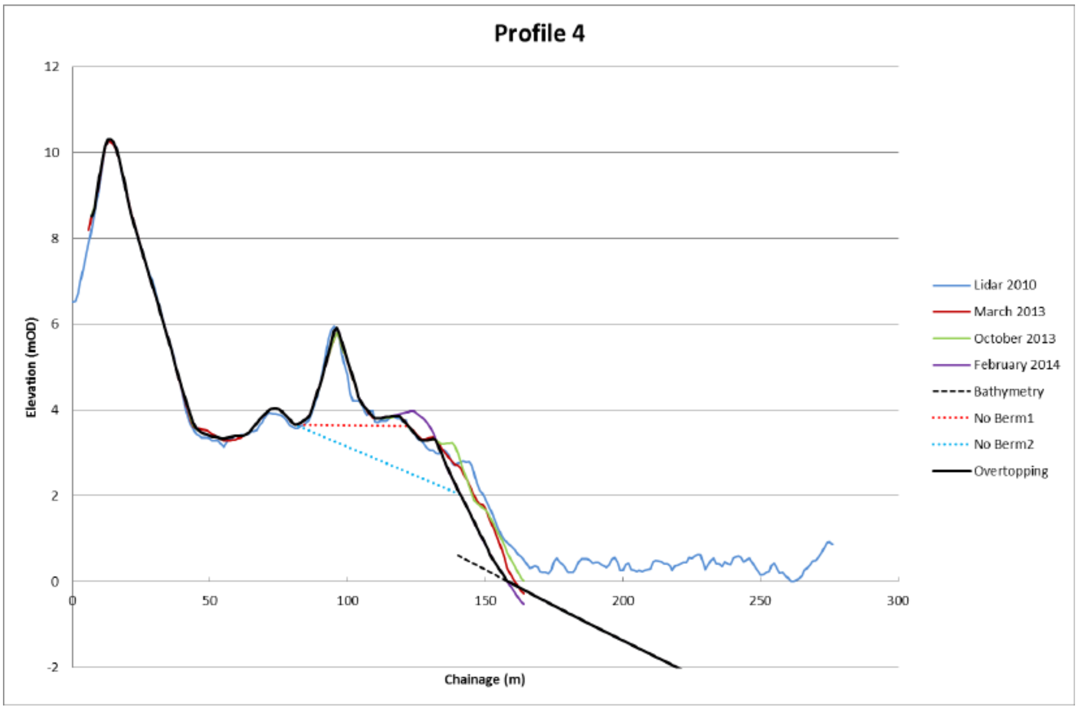
1.3.2. Further tests on the sensitivity of predicted overtopping to potential coastal erosion resulting in change of the beach profiles was assessed as a part of the Safety Case and is presented in a separate technical note.

a) Berm removal

1.3.3. At some locations along the Sizewell frontage there is a 'sacrificial berm' immediately landward of the active zone that could be washed away ('sacrificed') during a significant storm surge event, and therefore potentially result in increased overtopping risk at the main sea defence.

1.3.4. To assess this risk, AMAZON simulations were carried out for one of the profiles along the frontage (profile 4 at the southern end of proposed Sizewell C platform) for with and without berm scenarios (**Plate 1.5)**.

Plate 1.5: Representative defence/beach profile S4 (black thick line) for the sensitivity overtopping analysis



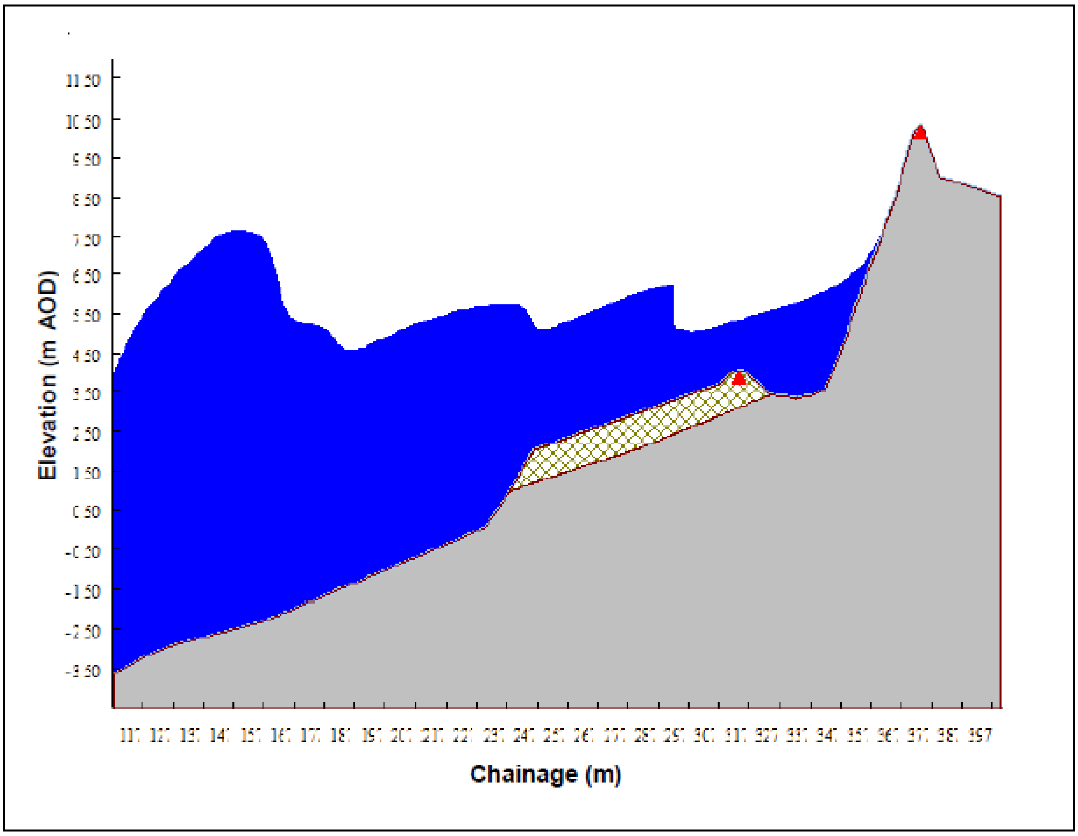
1.3.5. The results suggest that overtopping rates for a 1 in 1,000-year event with climate change allowance (1.55m relative sea level rise) for the scenario without the berm are slightly higher than those with the berm in place (**APPENDIX A**, **section 5.1**). Therefore, the more conservative approach with removed sacrificial berm was adopted for overtopping assessment at Sizewell C.

b) Beach sediment

1.3.6. As noted in **section 1.2b**, the Sizewell frontage has a substantial upper beach composed of shingle, however, the exact depth and nature of the interface between shingle and any underlying sand layers is not known and highly likely to vary along the length of the frontage.

1.3.7. To assess potential impacts of the shingle on wave energy dissipation and resulting overtopping rates, AMAZON simulation was carried out for the profile derived in the previous sensitivity test with assumed 1m deep shingle layer.

Plate 1.6: Profile used in AMAZON model, S4 with shingle layer for the sensitivity overtopping analysis



1.3.8. The results show that the difference between with and without shingle layer scenarios was marginal with a maximum of 3.6% for the 1 in 1,000-year event with climate change (1.55m relative sea level rise). Further details are provided in **APPENDIX A**, **section 5.2**.

1.3.9. Due to the relatively small effect of the shingle layer depth on overtopping rates and uncertainty regarding its depth and extent, no shingle layer was applied in the assessment for Sizewell C.

c) Offshore sand banks

1.3.10. There is the potential for the offshore bathymetry at the Sizewell frontage to change over time. In particular, sand banks offshore from Sizewell could migrate or disappear, resulting in change to the nearshore wave climate.

1.3.11. To determine whether the disappearance of the offshore sand banks could have a significant impact on wave overtopping, the TOMOWAC wave transformation model was tested for three scenarios. These are the baseline, 'Low 5' with the offshore sand banks lowered by 5m and 'ST1' with a shallow south trough.

1.3.12. The nearshore wave conditions were compared for the three scenarios for two profiles, one along proposed Sizewell C frontage (S4) and one along the Minsmere Belts (B10).

1.3.13. Overall, the 'baseline' scenario predicted slightly higher nearshore waves than the other scenarios and was therefore taken forward for assessment for the FRA overtopping model runs (further details are presented in **APPENDIX A**, **section 5.3**).

1.4 Model requirements

1.4.1. Outcomes of the wave overtopping assessment were used to provide an understanding of flood risk to the development site itself and inform safe access, egress and operation of the power station during an extreme coastal event throughout the development lifetime.

1.4.2. Current and future flood risk were assessed at different phases of the Sizewell C development's lifespan. Four key points in time identified for the overtopping flood risk are:

- 2034 – End of Construction / Start of Commissioning, 2030 used for assessment of construction phase flood risk as coastal defences would be completed by that time;
- 2090 – End of Operation;
- 2140 – Interim Spent Fuel Store Decommissioned, used for assessment of end to risks on site; and
- 2190 – end of theoretical maximum site lifetime.

1.4.3. To inform the FRA, a range of return period events were assessed with the wave overtopping model, namely 1 in 200, 1 in 1,000 and 1 in 10,000 years including allowances for climate change. It is recognized that the 1 in 10,000-year event is not required for the FRA, however it was assessed as basis of design. Further assessments for extreme events beyond basis of design, namely an estimated 1 in 100,000-year return period, were carried out as a part of the Safety Case study and are presented in a separate technical note.

1.4.4. Ideally, validation numerical modelling would be undertaken using hindcast wave data and historic information on overtopping to check the performance of the model against many events over the past 20 to 30 years. However, given the location and the height of the existing defences, no validation data could be found, as no overtopping had been reported at the proposed Sizewell C location. Therefore, it was assumed that the results would not be affected by lack of validation.

2 EUROTOP COMPARISON

2.1.1. Further to qualitative comparison of AMAZON with EurOtop discussed in **section 1.2b**, a quantitative assessment was carried out to determine whether overtopping rates predicted by AMAZON are comparable to those predicted by EurOtop.

2.1.2. Two calculation methods using the EurOtop manual were used, i.e. Neural Network tool (NN tool) developed by University of Bologna (Ref 8) and the Bayonet GPE overtopping modelling tool of coastal engineering developed by HR Wallingford (Ref 9). Both tools apply the principles of the EurOtop manual and are recognised in the industry for performing overtopping calculations.

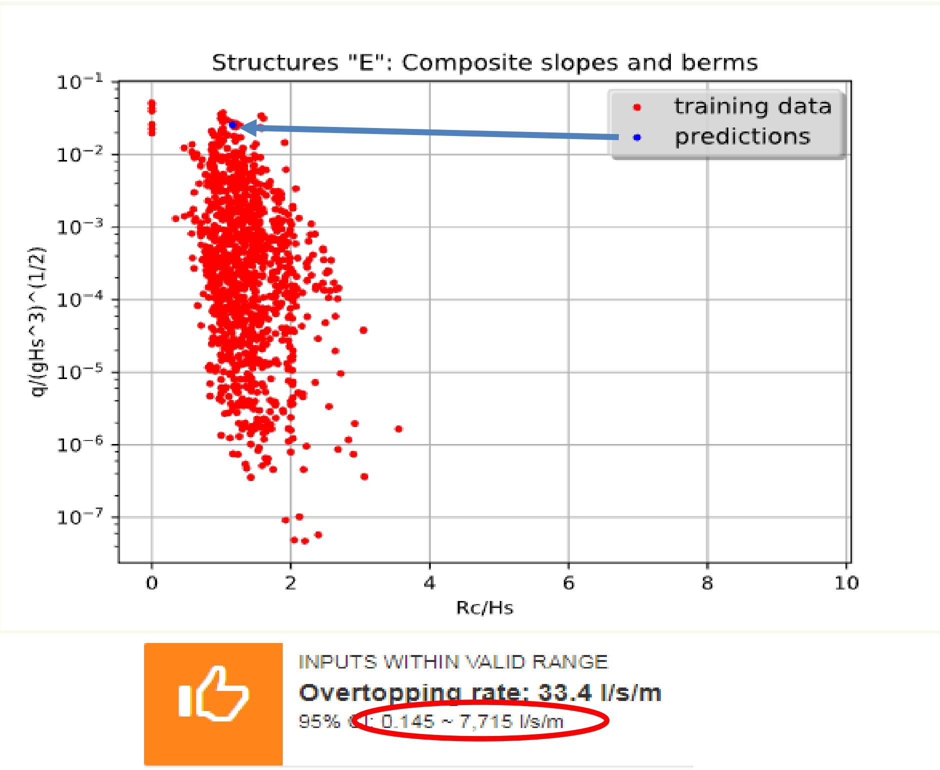
2.1.3. The NN tool has been developed for the assessment of the hydraulic performance of coastal and harbour structures to support scientists, engineers and consultants in the design of breakwaters, dikes, seawalls, sea embankments etc. The neural network was trained against an updated database of nearly 18,000 model-scale tests.

2.1.4. The Bayonet GPE tool uses statistical 'Gaussian Process Emulator' (GPE) technique, replacing the more traditional neural network model used to predict overtopping. The GPE modelling better predicts uncertainty by capturing a wide range of sources, including the fitting process and errors from laboratory data that historically haven't been taken into account. The tool allows engineers to test a wide range of sea wall types and specifications such as berms, rock-armoured toes and upper walls, against a detailed range of sea conditions.

2.1.5. The comparison was intended to be carried out on profile representative of the frontage in front of Sizewell C development with derived water level and wave conditions for a 1 in 10,000-year return period event (basis of design) with climate change allowance up to 2140.

2.1.6. A series of tests was carried out for different sets of parameters and varying wave conditions but it was found that the adopted settings did not fit within the group of the NN tool training data (**Plate 2.1**, top) and the range for rates predicted by the Bayonet GPE was too large to have confidence in the given results (**Plate 2.1**, bottom).

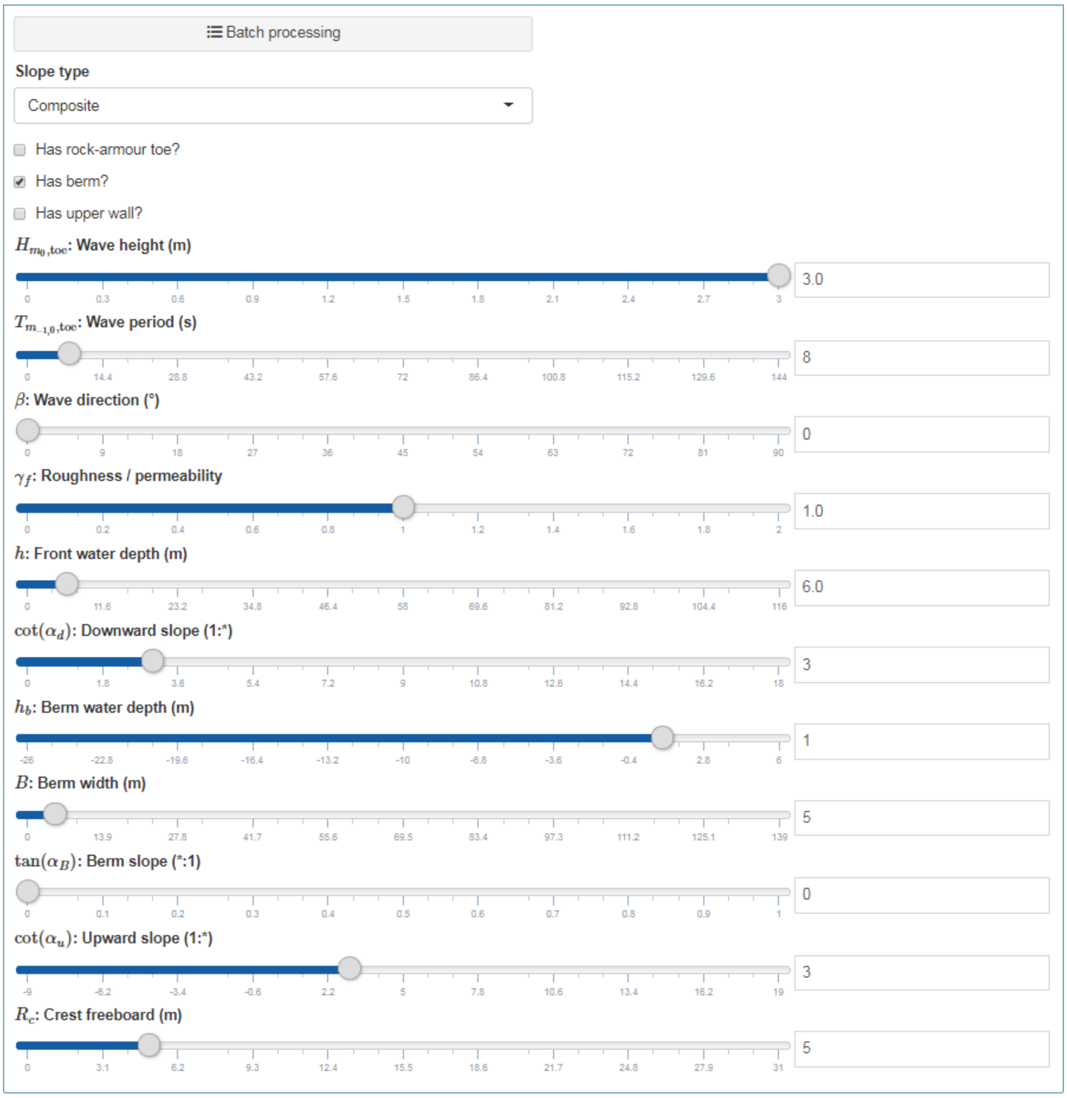
Plate 2.1: Initial NN tool (top) and Bayonet GPE (bottom) predicted overtopping rates



2.1.7. Following the initial tests, it was decided to adopt a simplified profile and less conservative water level and wave conditions that would be well represented within the training and laboratory data used in the NN tool and the Bayonet GPE. The same scenario was then run in the AMAZON software for direct comparison of predicted overtopping rates.

2.1.8. A composite profile with a 5m berm was selected. A 3m wave height with 8sec wave period was adopted with 6m water depth at the toe of the structure. Settings adopted for the overtopping runs for all three tools are presented in **Plate 2.2**.

Plate 2.2: Settings adopted for the EurOtop comparison test



2.1.9. Results from the NN tool show that the adopted scenario fits well within training data (**Plate 2.3**). The predicted overtopping rates from the three calculations tools are:

- NN tool – 5 l/s/m;
- Bayonet GPE – 20 l/s/m (**Plate 2.4**);
- AMAZON – 33 l/s/m.

2.1.10. The comparison shows that the NN tool predicts the lowest overtopping rates. Results from the Bayonet GPE and AMAZON are similar, especially considering range of overtopping rates provided by the Bayonet GPE.

Plate 2.3: Fit of predicted overtopping rates with the neural network training data

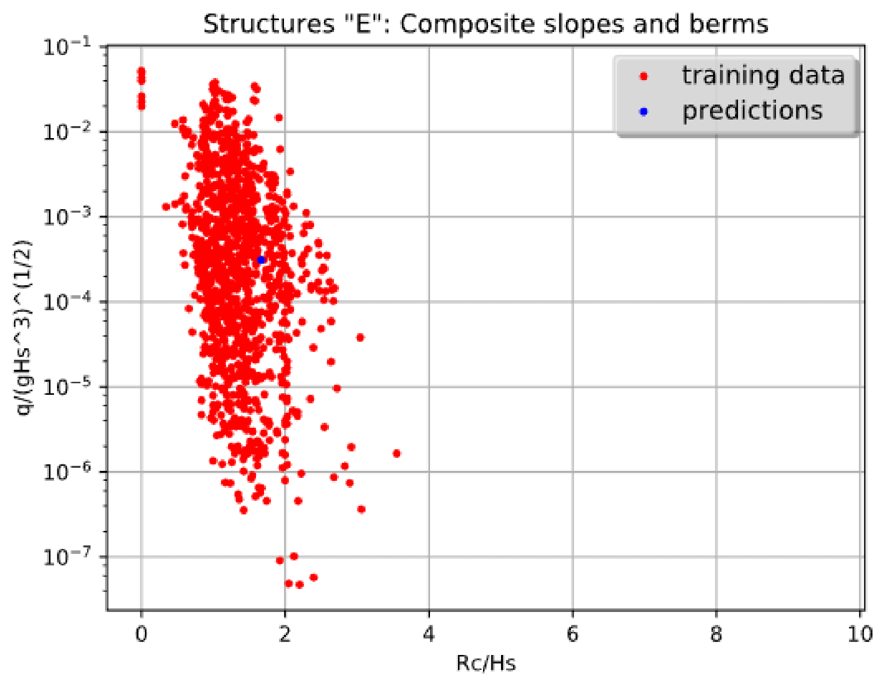


Plate 2.4: Predicted overtopping rates with the Bayonet GPE

2.1.11. From the three tested tools AMAZON predicts slightly higher overtopping rates, although they are within the order of magnitude of the rates predicted by Bayonet GPE. Bearing in mind that AMAZON allows overtopping predictions for non-standard beach profiles (found at Sizewell frontage) and gives slightly more conservative results, it was therefore considered appropriate to adopt for the Sizewell C coastal flood risk assessment.

3 SCENARIO SELECTION

3.1.1. In order to inform the Sizewell C FRA, a number of overtopping simulations at the Sizewell C sea defences were required for a range of return period events and climate change scenarios. Initially EDF identified a list of scenarios for the overtopping assessments (Ref 10) of the Sizewell C main sea defence (HCDF), the northern mound section and the SSSI crossing that were required to support the PEIR for Stage 3 Consultation. Further runs were then identified to complete the range of overtopping scenarios for the main sea defence (HCDF) and the SSSI crossing required for the FRA.

3.1.2. For the climate change allowances, two main datasets were used, UK Climate Projections (Ref 11) published in November 2018 (UKCP18) for the reasonably foreseeable scenario, and more conservative BECC Upper allowances derived for the Sizewell C project (Ref 12) applied for the credible maximum scenario.

3.1.3. The previous overtopping assessments and sensitivity tests for the reasonably foreseeable climate change scenarios were carried out including relative sea level rise based on UKCP09 dataset as the latest available at the time.

3.1.4. After release of the UKCP18 projections, the allowances for sea level rise were updated and compared with those derived based on UKCP09. Details of this assessment can be found in the UKCP18 Review report (Ref 13), first issued to EDF in March 2019 and then updated in October 2019 following Environment Agency comments and further advice.

3.1.5. As mentioned in **section 1.2c**, AMAZON requires input of wave height, wave period and water level. Following updates to the climate change projections, the wave transformation model should be also updated to account for different sea level rise allowances.

3.1.6. However, considering project programme constraints, the TOMOWAC model has not been re-run at this stage of the works to account for changes in climate change allowances. For the purpose of this overtopping study, the closest case (combination of wave height and still water level) from the currently available joint probability assessment has been identified. This closest case was then used to obtain the nearshore wave conditions for wave overtopping modelling, but the extreme still water levels were applied in line with the updated climate change allowances. It is recognised that for future stages of the project, the joint probability assessment should be revisited and updated accordingly to the latest guidance as recommended in **section 1.2c**.

3.1.7. In the previous assessment, a sensitivity test was carried out to determine the worst joint probability combination of extreme still water levels and wave heights, i.e. one that results in highest overtopping rates. It was found that the combination with higher (but not highest) water level with lower offshore wave heights resulted in worst nearshore wave conditions and therefore greatest overtopping rates (Ref 14). These combinations were therefore adopted for further analysis as a more conservative approach.

3.1.8. The extreme still water levels derived for the Sizewell C project during the BEEMS study (Ref 15) were adopted for the overtopping assessment as they were more conservative than those available from the Environment Agency 'UK Coastal Flood Boundary Conditions'. Further details on derivation of the extreme still water levels and comparison with the Environment Agency CFB dataset from 2018 are provided in UKCP18 Review report (Ref 13).

3.1.9. **Table 3.1** provides a summary of overtopping scenarios carried out to inform the FRA, outlining considered return period events, climate change scenarios and adopted joint probability combinations. These scenarios were selected as the most informative when assessing potential coastal flood risk to the Sizewell C site at key points of the development's lifespan.

3.1.10. Not all scenarios presented in the table below were simulated for all the three identified defence profiles but were rather selected based on the requirement for the individual defence. 2025 epoch was used for the initial construction phase with the most exposed profiles, whereas 2030 epoch was used for the end of construction phase.

3.1.11. List of model runs for each defence profile is provided in **section 5.2**, later in this note.

Table 3.1: Summary of the adopted climate change scenarios and closest joint probability cases for selected epochs

Epoch	Climate Change Scenario	Relative Sea Level Rise (m)	Closest joint probability case		
			1 in 200	1 in 1,000	1 in 10,000
2025 / 2030[1]	95% High Emissions (UKCP09)	0.113	Case D	Case D	×

Epoch	Climate Change Scenario	Relative Sea Level Rise (m)	Closest joint probability case		
			1 in 200	1 in 1,000	1 in 10,000
	RCP8.5 (UKCP18)	0.148	Case D	Case D	×
2090	RCP8.5 (UKCP18)	0.921	×	Case 3	Case 3
	H++ with Land Motion plus 1m Surge	1.550	×	×	Case 8
2140	RCP8.5 (UKCP18)	1.731	Case 7	Case 7	Case 7
	BECC Upper	3.920	×	Case 15	Case 15
2190	RCP8.5 (UKCP18)	2.645	Case 9	Case 9	Case 9

Note 1: Construction phase is currently estimated to be circa 2034 with estimated completion of the main sea defences (HCDF) no further than by 2030. This model run will use nearshore wave conditions from 2025 (previously provided by CEFAS), with updated RCP8.5 Sea Level Rise.

4 PROFILES DERIVATION

4.1 Overview

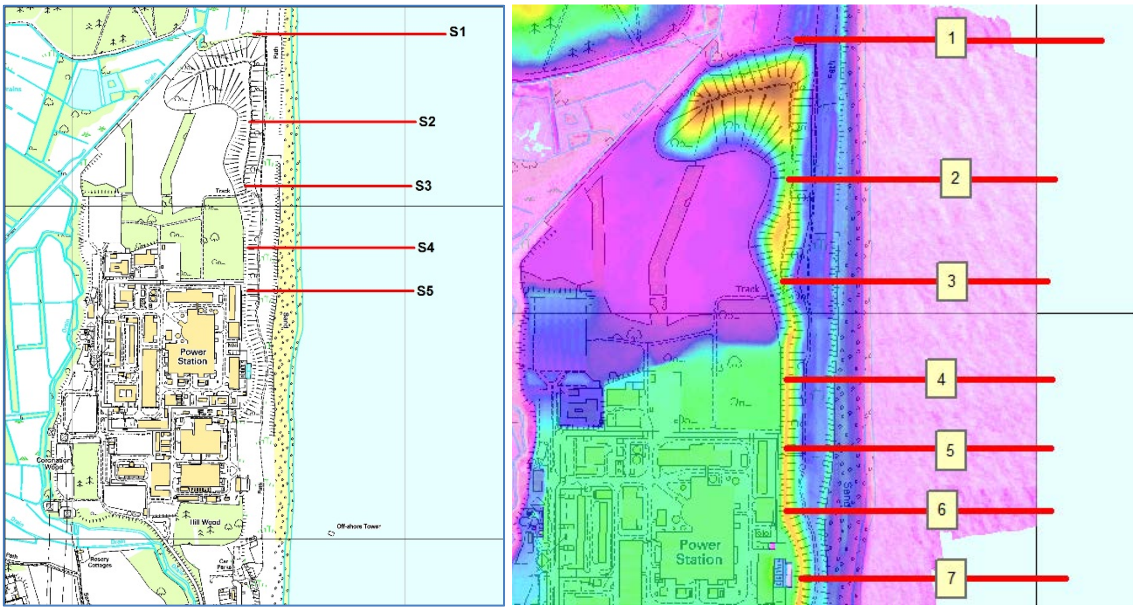
4.1.1. Profiles used in the preliminary assessments have previously been extracted at locations shown in **Plate 4.1** below, with LiDAR used to highlight changes in elevations on the right-hand image. These profiles were derived by merging various datasets of near-shore bathymetry and beach profiles (topographic surveys and Environment Agency's 1m composite LiDAR supplied in 2013 for the wave transformation modelling).

4.1.2. The topographic surveys were carried out as a part of EDF Energy Beach Monitoring Programme for Sizewell B Power Station Coast Protection. The surveys were undertaken as follows:

- October 2011;
- February 2012;
- March and October 2013; and
- March 2014.

4.1.3. Selected profiles were combined with the proposed design of sea defences to derive the modelling profiles for the main sea defence (HCDF), the northern mound and SSSI crossing, and then used for the overtopping assessment. Derivation of each profile is described in the following sub-sections.

Plate 4.1: Profile Locations (OS Map – left, Lidar – right)

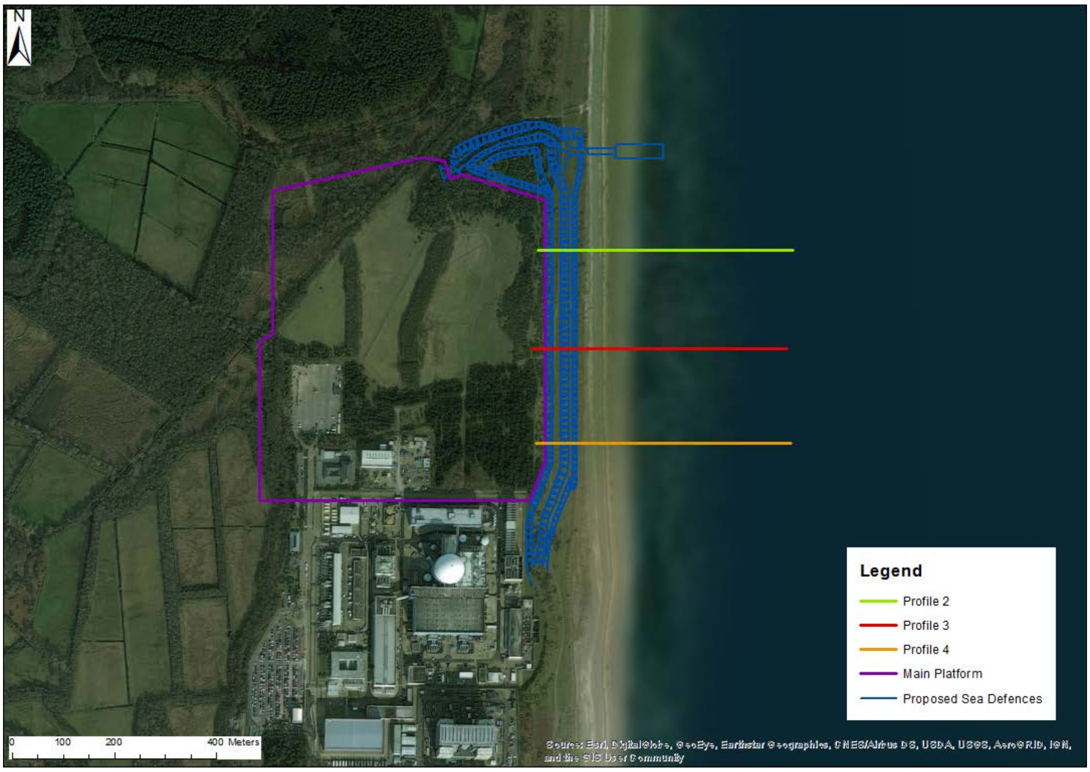


4.2 Main sea defence (HCDF)

4.2.1. For the sea defences at the main development site, previously derived Profile 3 (in front of the proposed main platform) was chosen, as it produced the highest overtopping rates in the preliminary overtopping modelling (**APPENDIX A**).

4.2.2. The overtopping profile for the construction phase was derived by combining the previous profile 3 and the proposed defence design based on drawing no. SZC-SZ0100-XX-000-DRW-100025 (dated 14/07/2018). The location of the profile relative to the proposed main platform is shown in **Plate 4.2** below.

Plate 4.2: Location of profile 3 relative to the proposed platform



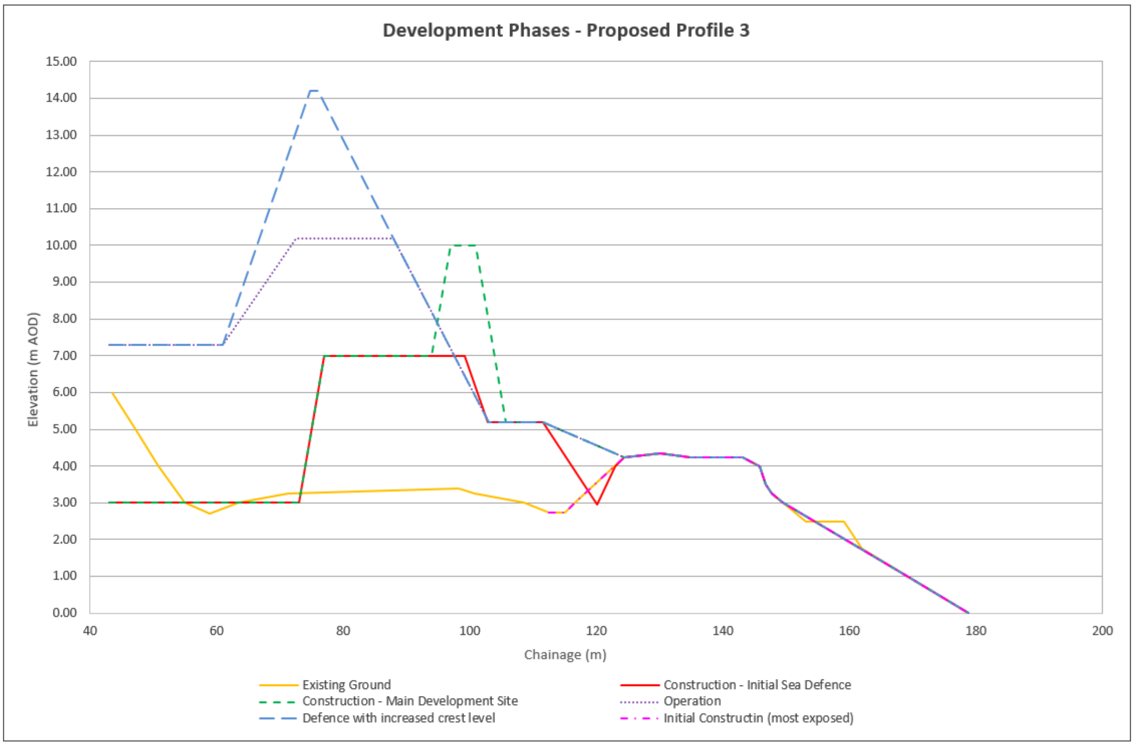
4.2.3. The derived profile is illustrated in **Plate 4.3**. The pink (dash-dot) line represents the most exposed profile during initial construction stage with crest of 4.36m AOD. The crest of the sand dunes varies along the frontage in front of the proposed Sizewell C development with levels up to 5.2m AOD. For more conservative approach, the lower crest of 4.36m AOD was modelled as the 'most exposed' profile.

4.2.4. The green line in **Plate 4.3** represents the interim 'construction of the main development site' phase, i.e. once the preliminary sea defence is completed with crest level at 10.2m AOD but construction of the main platform is ongoing. The purple line shows the defence profile at the end of construction and through the operation phases.

4.2.5. The 'existing ground' profile in **Plate 4.3** (orange line) was derived from the design drawing (drawing no. SZC-SZ0100-XX-000-DRW-100025) that included existing ground levels prior to construction. This was understood to be surveyed in 2018 with beach levels set back in comparison to the survey from February 2014.

4.2.6. Further details on phasing of the construction of the new sea defences for Sizewell C are available in the Description of Development for the Main Development Site: Chapter 3 of the Sizewell C Project Environmental Statement, Volume 2 (Ref 17).

Plate 4.3: Profile 3 merging for overtopping model (construction phase)



4.2.7. Respective profiles were selected for the overtopping modelling depending on which development phase was being assessed.

4.3 Northern mound

4.3.1. The northern mound overtopping profile was constructed using the latest provided drawing of the sea defences design at the time of the modelling (drawing no. SZC-SZ0100-XX-000-DRW-100031, dated 29 June 2018). Section 2-2, shown in **Plate 4.4** below, was combined with profile 1 that was previously derived by selecting the worst (lowest) data from the various topographic datasets, with bathymetry data used for the offshore end of the profile.

4.3.2. The overtopping profile was derived by merging profile 1 and the northern mound section to form the design drawing at the intersection point shown in **Plate 4.5**. The northern mound profile is at an angle to profile 1, so the final composite profile has a change of angle partway along it. This broadly represents the effect of the waves turning due to refraction as they would approach the northern mound defence. The AMAZON 1D wave model assumed that waves arrive normal to shore and simply follow the combined profile, which is more conservative as doesn't account for wave energy dissipation when waves change the approach angle.

Plate 4.4: Location of northern mound profile (section 2-2, extracted from drawing no. SZC-SZ0100-XX-000-DRW-100031)

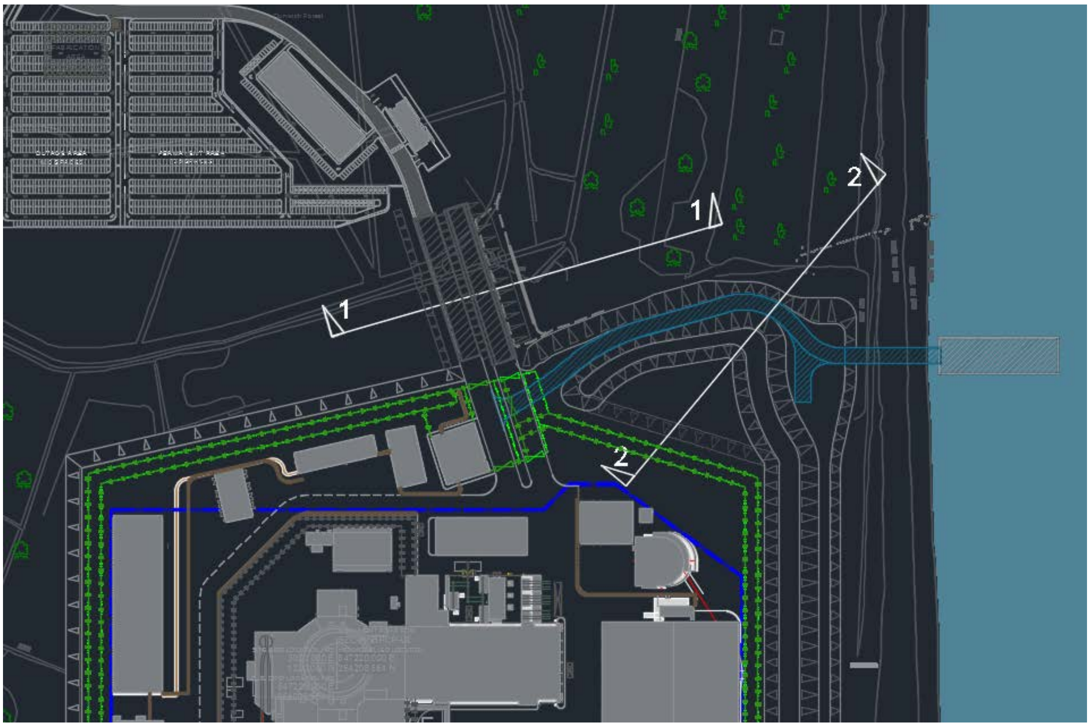
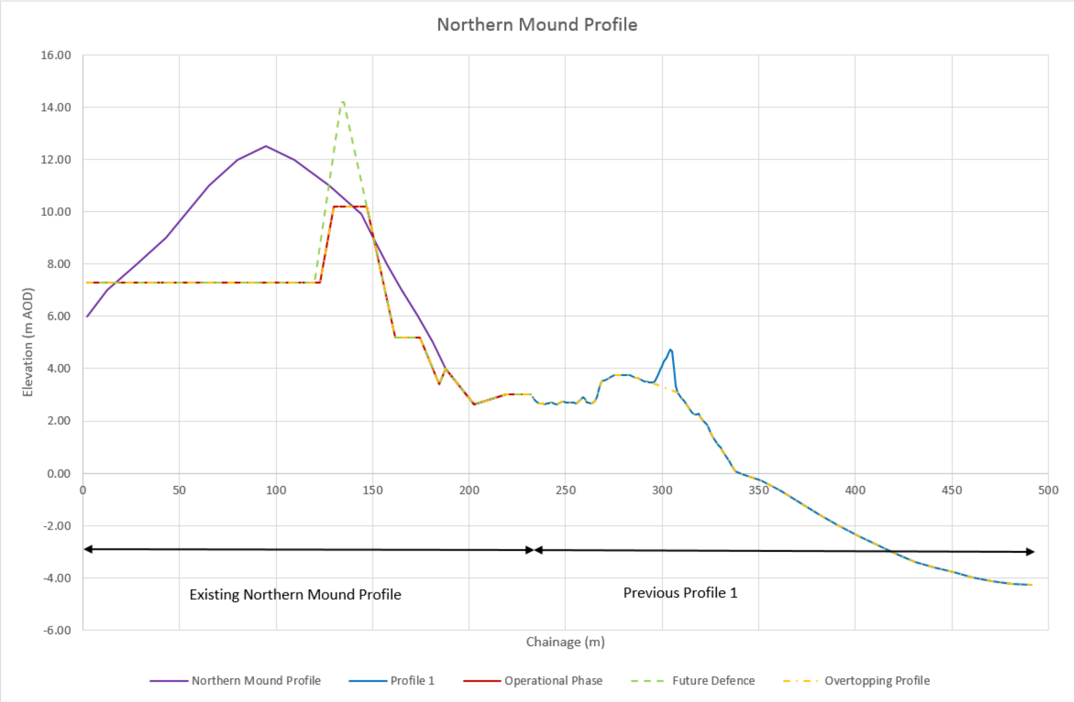


Plate 4.5: Plan view of northern mound profiles prior to merging

4.3.3. The derived overtopping profile for the northern mound is shown in **Plate 4.6.** The 'operation phase' profile was used for the 1 in 200-year and 1 in 1,000-year scenarios, and 'future defence' profile was used where applicable for 1 in 10,000-year scenarios listed in **section 5.2**.

Plate 4.6: Northern mound profile derived for overtopping



4.3.4. According to 'Figure 3A.9 - Indicative cross-section coastal defence' of the Sizewell C MDS Description of Development for construction phase (Ref 17), during the initial construction phase, the existing Northern Mound would be first demolished in order to carry out necessary ground works. It will then be reconstructed to a level of 10.2m AOD with built access road to the beach landing facility in front (sea side) of the defence.

4.3.5. The most exposed profile would therefore be at that initial construction phase when existing mound is demolished, with existing sand dunes and shingle beach temporarily acting as primary defence. Such scenario was modelled for the main sea defence (HCDF), with slightly lower crest of the existing sand dunes than it is in front of the Northern Mound, and therefore additional scenario for the Northern Mound profile was not modelled.

4.3.6. Further details on phasing of the construction of the Northern Mound defences for Sizewell C are available in the Description of Development for the Main Development Site: Chapter 3 of the Sizewell C Project Environmental Statement, Volume 2 (Ref 17).

4.4 SSSI crossing

4.4.1. The SSSI crossing beach profile was based on a modification of previously derived profile 1 (**Plate 4.5**), but with the shoreline moved landward to account for potential future 'coastal roll-back' scenario (the existing beach profile 1 is assumed to retreat landward in line with the predicted future shoreline position, only front beach profile was rolled back, subsea profile was not changed/eroded).

4.4.2. The position of the coastline relative to the existing shoreline was informed by the Sizewell Coastal Geomorphology and Hydrodynamics study results (Ref 16). The 'Future shoreline with SZC' shown in **Plate 4.7**.

4.4.3. The proposed design of the SSSI crossing profile was based on the latest drawing of sea defences supplied by EDF at the time of the modelling (SZC-SZ0100-XX-000-DRW-100031, dated 29 June 2018), section 1-1, shown in **Plate 4.4**.

4.4.4. As with the northern mound profile, the proposed SSSI crossing profile is at an angle relative to the shoreline (**Plate 4.8**). In the AMAZON 1D model it was assumed that waves arrive normal to the shoreline, simply following the composite profile, whereas in reality, the waves would not make an immediate turn due to refraction when approaching the SSSI crossing.

4.4.5. The overtopping profile was derived by merging the 'rolled-back' beach profile 1 with the proposed design of the causeway, as illustrated in **Plate 4.9**. The 'operation phase' profile was used for all three considered return period events, namely 1 in 200-year 1 in 1,000-year and 1 in 10,000-year, whereas 'future defence', i.e. adaptive profile was used for future epochs, as listed in **section 5.2**.

Plate 4.7: Extract from ‘Sizewell Coastal Geomorphology and Hydrodynamics’ Report (Ref 16), Figure 60: Projected shorelines and cliff-lines with and without Sizewell C

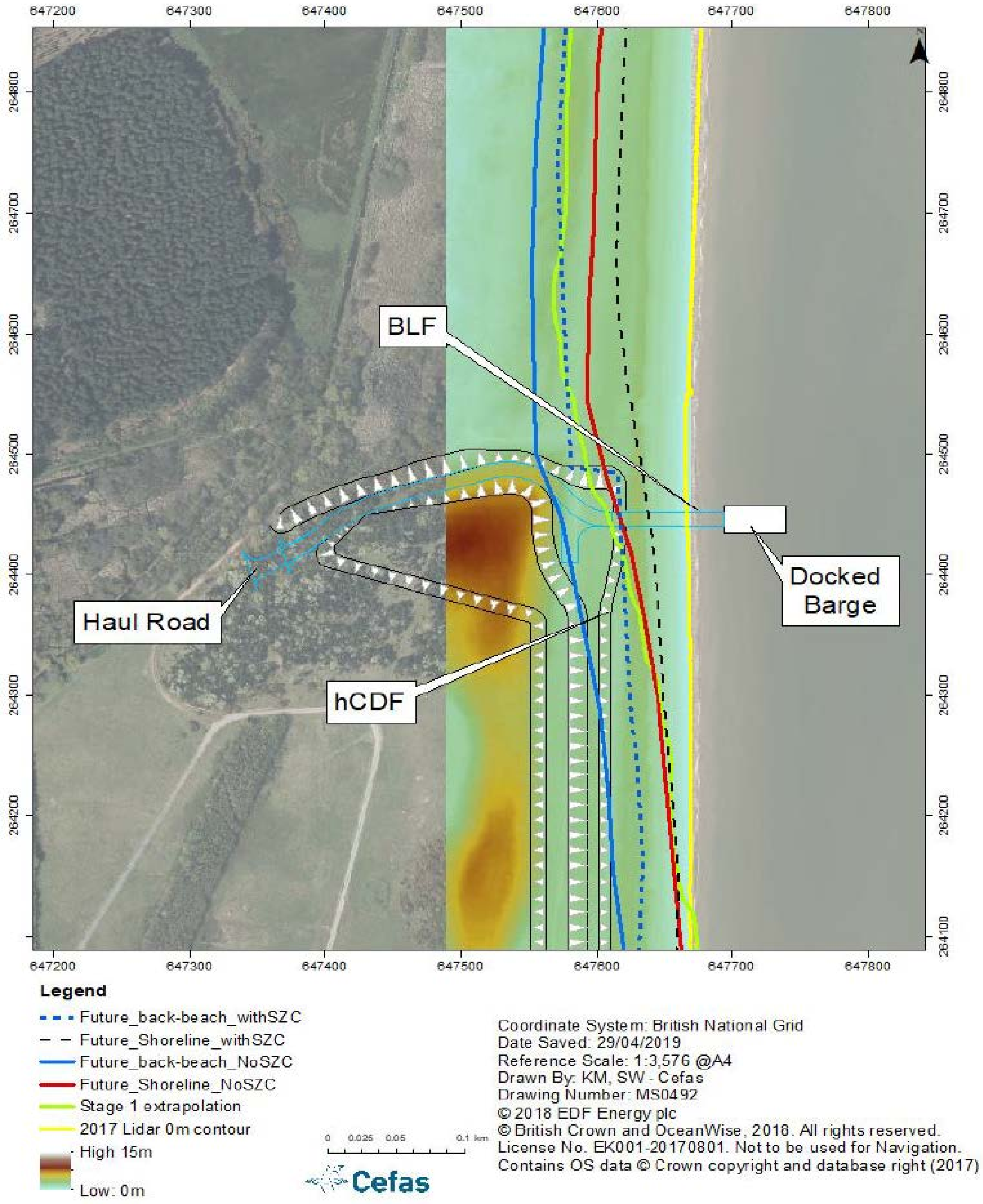


Plate 4.8: Plan view of the SSSI crossing profiles prior to merging

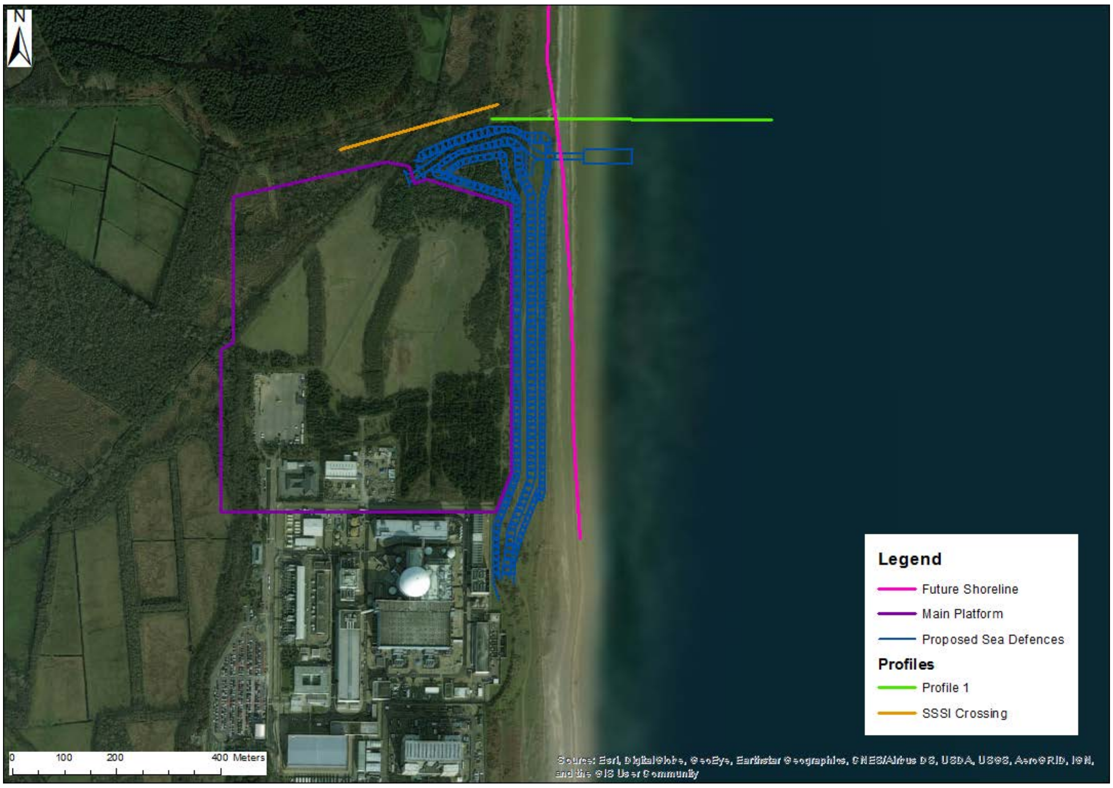
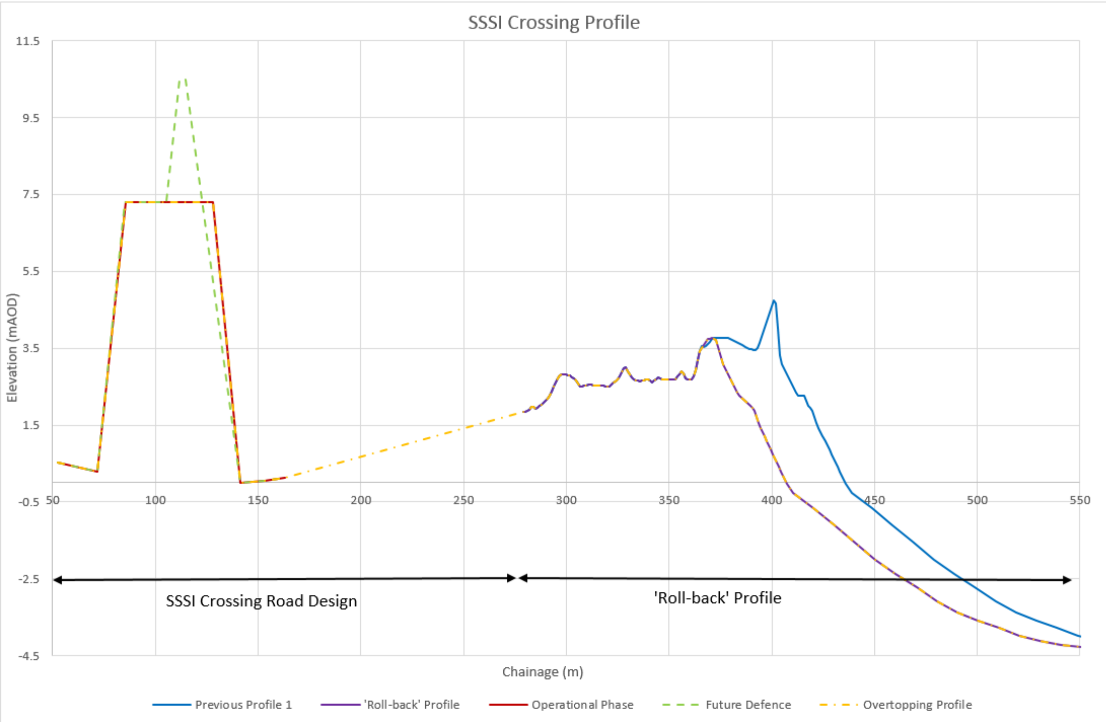


Plate 4.9: SSSI crossing Profile merging for overtopping



5 MODEL RUNS AND RESULTS

5.1 Overview

5.1.1. AMAZON model settings were adopted following the same approach as in the preliminary overtopping assessments, i.e. the JONSWAP wave spectrum with offshore wave period was used with nearshore significant wave height from the TOMOWAC wave transformation model, and ‘still water’ peak tide levels from the joint probability combinations (adjusted to account for UKCP18 sea level rise where applicable). The overtopping model was run for the three considered defence profiles, i.e. main sea defence (HCDF), northern mound and the SSSI crossing.

5.2 List of model runs

5.2.1. A summary of wave overtopping simulations and corresponding model input conditions for the selected return period events and climate change scenarios is presented in **Table 4.1**, **Table 4.2** and **Table 4.3** for the main sea defence (HCDF), the northern mound and SSSI crossing respectively. Simulations carried out in 2018 (pre UKCP18) for the northern mound and SSSI crossing used the 2110 epoch, which is between end of operation and the interim spent fuel store decommissioning phases.

5.2.2. All proposed runs were carried out for the worst joint probability combination of extreme water level and significant wave height that was determined in the preliminary overtopping assessments (**APPENDIX A**). The runs represent worst case combination of high wave conditions coinciding with the peak of the surge event at high tide.

5.2.3. The joint probability combination codes determined in the previous sensitivity tests to produce highest overtopping rates are: F1, F2 and B3 for the 1:10,000, 1:1,000 and 1:200-year return periods respectively (Ref 3 and **APPENDIX A**).

5.2.4. Some of the preliminary overtopping runs for the 1 in 200-year return period were carried out for the joint probability combination C3. It was then determined that joint probability combination B3 produces slightly higher overtopping rates and was therefore adopted in further assessments.

5.2.5. To inform the FRA, overtopping scenarios for up to 1 in 1,000-year event were required with reasonably foreseeable climate change scenario (UKCP18 RCP8.5). Further calculations were carried out for the 1 in 10,000-year event and the credible maximum climate change scenarios. Additional runs for the 1 in 10,000 and 1 in 100,000-year events were undertaken for Safety Case assessment, presented in a separate technical note.

Table 5.1: List of wave overtopping scenarios carried out for the main sea defence (HCDF)

Return Period (year)	Epoch – Climate Change Scenario	JP Code	Defence Crest (m AOD)	Relative SLR (m)	Water Level (m AOD)	Wave period (Tp, s)	Inshore Wave Height (Hs, m)
200	2025 – 95% High Emissions (UKCP09)	C3	10.0	0.113	3.77	9.11	2.62
	2030 – RCP8.5 (UKCP18)	B3	4.36 / 7.0*	0.148	3.33	11.30	2.85
	2140 – RCP8.5 (UKCP18)	B3	10.2 / 14.2	1.815	5.00	11.82	3.73
	2190 – RCP8.5 (UKCP18)	B3	10.2 / 14.2	2.645	5.83	11.82	4.03
1,000	2025 – 95% High Emissions (UKCP09)	F2	10.0	0.113	4.13	10.90	3.25
	2030 – RCP8.5 (UKCP18)	F2	10.2	0.148	4.17	10.90	3.25
	2140 – RCP8.5 (UKCP18)	F2	10.2 / 14.2	1.815	5.84	11.40	3.94
	2140 – BECC Upper	F2	14.2	3.920	7.94	11.64	4.11
	2190 – RCP8.5 (UKCP18)	F2	10.2 / 14.2	2.645	6.67	11.4	4.10
10,000	2090 – RCP8.5 (UKCP18)	F1	10.2	0.921	5.85	11.95	4.15
	2090 – H++ with Land Motion plus 1m Surge	F1	10.2	1.530	6.46	11.95	4.34

Return Period (year)	Epoch – Climate Change Scenario	JP Code	Defence Crest (m AOD)	Relative SLR (m)	Water Level (m AOD)	Wave period (Tp, s)	Inshore Wave Height (Hs, m)
	2140 – RCP8.5 (UKCP18)	F1	10.2 / 14.2	1.731	6.66	11.95	4.42
	2190 – RCP8.5 (UKCP18)	F1	10.2 / 14.2	2.645	7.58	12.20	4.77
	2140 – BECC Upper	F1	14.2	3.920	8.85	12.20	4.96

**4.36m AOD is the crest level of the existing shingle beach profile. This run is for the most exposed profile during initial construction phase when current defence will be removed to carry out ground improvement work for construction of the reinforced defence. 7.0m AOD is the level of the haul road for construction phase.*

Table 5.2: List of wave overtopping scenarios carried out for the northern mound defence

Return Period (year)	Epoch – Climate Change Scenario	JP Code	Defence Crest (m AOD)	Relative SLR (m)	Water Level (m AOD)	Wave period (Tp, s)	Inshore Wave Height (Hs, m)
200	2110 – 95% Medium Emissions (UKCP09)	C3	10.2	0.744	4.40	9.52*	2.63*
1,000	2110 – 95% High Emissions (UKCP09)	F2	10.2	1.014	5.03	11.40	3.73
10,000	2110 – H++ with Land Motion plus 1m Surge	F1	14.2	3.200	8.13	12.20	4.83

Table 5.3: List of wave overtopping scenarios carried out for the SSSI crossing

Return Period (year)	Epoch – Climate Change Scenario	JP Code	Defence Crest (m AOD)	Relative SLR (m)	Water Level (m AOD)	Wave period (Tp, s)	Inshore Wave Height (Hs, m)
200	2110 – 95% Medium Emissions (UKCP09)	B3	7.3	0.744	3.92	11.82	3.21
	2140 – RCP8.5 (UKCP18)	B3	7.3 / 10.5	1.815	5.00	11.82	3.73
	2190 – RCP8.5 (UKCP18)	B3	7.3 / 10.5	2.645	5.83	11.82	4.03
1,000	2090 – RCP8.5 (UKCP18)	F2	7.3	0.921	4.94	11.40	3.73
	2140 – RCP8.5 (UKCP18)	F2	7.3	1.731	5.75	11.40	3.94
	2140 – BECC Upper	F2	7.3 / 10.5	3.920	7.94	11.64	4.09
	2190 – RCP8.5 (UKCP18)	F2	7.3 / 10.5	2.645	6.67	11.40	4.08
10,000	2090 – RCP8.5 (UKCP18)	F1	7.3	0.921	5.85	11.95	4.14
	2090 – H++ with Land Motion plus 1m Surge	F1	7.3	1.530	6.46	11.95	4.32
	2140 – RCP8.5 (UKCP18)	F1	7.3 / 10.5	1.731	6.66	11.95	4.41
	2140 – BECC Upper	F1	10.5	3.920	8.85	12.20	4.92

5.2.6. **Table 5.2** lists scenarios considered for the northern mound defence. That assessment was carried out in 2018, prior to UKCP18 climate projections being published. Considering results obtained based on the UKCP09 projections it was assumed that relative difference in sea level rise allowances (of less than 0.2m at 2090, Ref 5.2. 24) would not change conclusions of flood risk to the northern mound and therefore the assessment was not revised.

5.3 Overtopping threshold guidance

5.3.1. Results of the wave overtopping simulations were assessed against the tolerable overtopping rates published in the EurOtop Manual on wave overtopping of sea defences and related structures (Ref 4).

5.3.2. The EurOtop manual specifies tolerable overtopping rates for property and equipment as well for people and vehicles. Corresponding overtopping limits are presented in **Table 5.4** and **Table 5.5** respectively. Visual reference may also be made to associated overtopping rate videos from http://www.overtopping-manual.com/eurotop/videos-of-wave-overtopping/.

5.3.3. For the purpose of the SSSI crossing assessment, results were compared mainly against the tolerable overtopping rates for vehicles on seawall crest with wave height of 3m (the largest available), i.e. < 5 l/m/s. Such tolerable rate was adopted as considered SSSI crossing as access road to site only, not a highway or a road with fast traffic where any overtopping is not safe.

Table 5.4: General limits for overtopping for property behind the defence

Hazard type and reason	Mean overtopping (l/s per m)	Max Volume V_{max} (l per m)
Building structure elements; Hm0 = 1-3m	=<1	<1,000
Damage to equipment set back 5-10m	=<1	<1,000

Table 5.5: Limits for overtopping for people and vehicles

Hazard type and reason		Mean overtopping (l/s per m)	Max Volume V_{max} (l per m)
People at structures with possible violent overtopping, mostly vertical structures		No access for any predicted overtopping	No access for any predicted overtopping
People at seawall /	H_{m0} = 3m	0.3	600

Hazard type and reason		Mean overtopping (l/s per m)	Max Volume V_{max} (l per m)
dike crest. Clear view of the sea	H_{m0} = 2m	1	600
	H_{m0} = 1m	10-20	600
	H_{m0} < 0.5m	No limit	No limit
Cars on seawall / dike crest, or railway close behind crest	H_{m0} = 3m	<5	2000
	H_{m0} = 2m	10-20	2000
	H_{m0} = 1m	<75	2000
Highways and roads, fast traffic		Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous

5.3.4. The EurOtop Manual does not provide clear guidance on tolerable overtopping thresholds for people behind coastal defences. Since the main Sizewell C platform would be set back from the coastal defence, the overtopping threshold for people at seawall crest of 0.3 l/s/m would be considered too conservative. Therefore, the same tolerable overtopping threshold as for the SSSI crossing of 5 l/s/m was adopted in analysis of the results for the main platform, although still considered very conservative as it doesn't account for energy dissipation between the overtopped defence and the main platform area where potential people and building at risk would be located.

5.3.5. It is understood that access to the top of the main sea defence (HCDF) would be fenced off and not available to public and therefore there would be no people present at the crest of the defence.

5.4 Overtopping Results

5.4.1. The AMAZON model provides mean overtopping rates at specified output points. For this study, points on the landward side of the defence crest for each of the at the assessed profiles were adopted. Where overtopping was predicted, an assessment was made to determine whether the overtopping rates are tolerable based on the thresholds outlined in **section 5.3**, taking into account development phase and safety of access and egress routes.

5.4.2. Further assessment of the results and discussion on the risk to the development site and off-site receptors are provided in the Sizewell C FRA.

a) Main Sea Defence

5.4.3. **Table 5.6** presents the predicted mean overtopping rates over the main sea defence (HCDF) for each of the modelled scenarios.

Table 5.6: Predicted mean overtopping rates (l/s/m) for the main sea defence (HCDF)

Return Period (year)	Epoch – Climate Change Scenario	JP Code	Defence Crest (m AOD)	Mean Overtopping Rates (l/s/m)
200	2025 – 95% High Emissions (UKCP09)	C3	10.0	0.00
	2030 – RCP8.5 (UKCP18)	B3	4.36 / 7.0*	140.36 / 0.03
	2140 – RCP8.5 (UKCP18)	B3	10.2 / 14.2	0.30 / 0.00
	2190 – RCP8.5 (UKCP18)	B3	10.2 / 14.2	4.50 / 0.00
1,000	2025 – 95% High Emissions (UKCP09)	F2	10.0	0.00
	2030 – RCP8.5 (UKCP18)	F2	10.2	0.00
	2140 – RCP8.5 (UKCP18)	F2	10.2 / 14.2	3.79 / 0.00
	2140 – BECC Upper	F2	14.2	2.29
	2190 – RCP8.5 (UKCP18)	F2	10.2 / 14.2	23.17 / 0.02
10,000	2090 – RCP8.5 (UKCP18)	F1	10.2	5.80
	2090 – H++ with Land Motion plus 1m Surge	F1	10.2	21.05
	2140 – RCP8.5 (UKCP18)	F1	10.2 / 14.2	36.42 / 0.29
	2190 – RCP8.5 (UKCP18)	F1	10.2 / 14.2	153.62 / 4.41
	2140 – BECC Upper	F1	14.2	41.83

5.4.4. Results in **Table 5.6** show that during the early construction phase when the existing defences are removed for ground improvement works, the site would be most exposed (i.e. only form of defence would be existing shingle beach with crest of 4.36m AOD). The predicted overtopping rate for a 1 in 200-year return period event is relatively high and therefore a robust risk mitigation plan for the construction site should be put in place. This would be discussed further in the Sizewell C FRA.

5.4.5. Once the proposed interim sea defence is constructed, there is no predicted overtopping for the 1 in 1,000-year return period event and therefore the construction site would not be at risk. During operation phase and up to end of interim spent fuel store decommissioning, the overtopping rates are below the 5l/s/m threshold for up to 1 in 1,000-year event.

5.4.6. For the 1 in 10,000-year event (basis of design) the overtopping rates are beyond the tolerable limit with the design defence height, however with the adaptive defence with crest at 14.2m AOD, the overtopping is reduced below the threshold for the reasonably foreseeable scenario. The adaptive defence would also be sufficient to limit overtopping below threshold rates for the 1 in 1,000-year event with credible maximum climate change allowance.

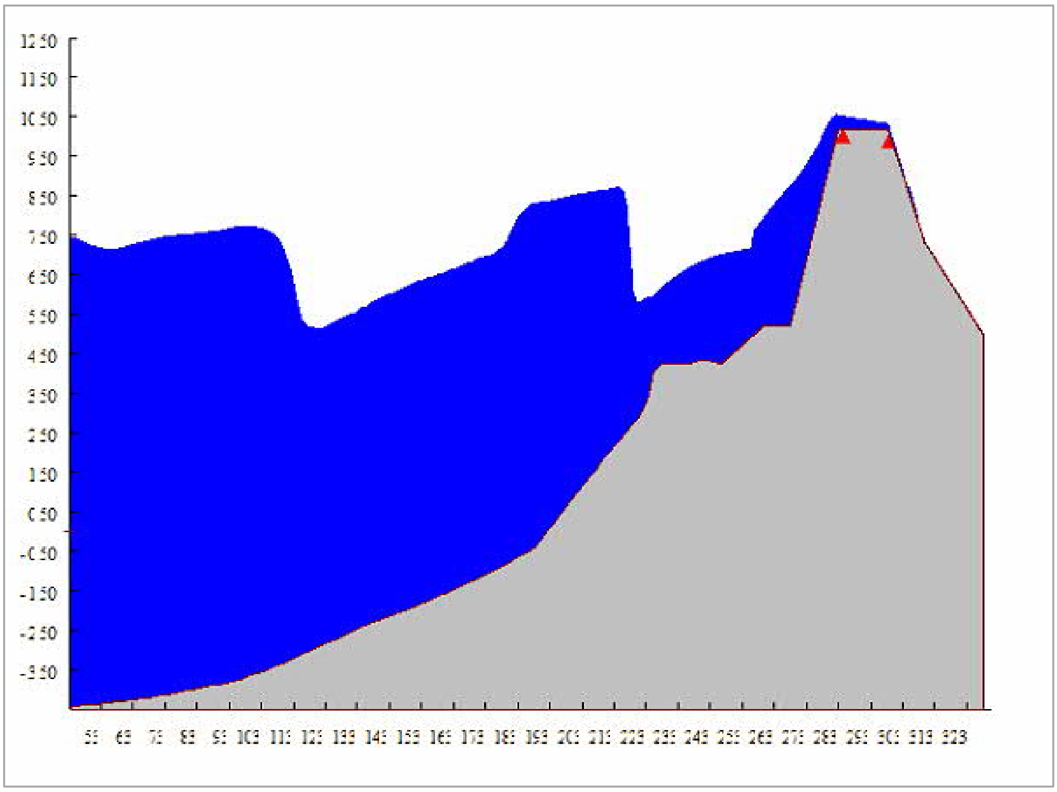
5.4.7. For the end of theoretical maximum site lifetime at 2190, the results suggest that with the design defence crest at 10.2m AOD the overtopping rates might be dangerous to people and vehicles for the in 1in 1,000 and 1 in 10,000-year events, however with adaptive defence the risk would be mitigated to a tolerable overtopping rate of 5 l/s/m.

5.4.8. Overtopping for the credible maximum scenario for the 1 in 200-year and 1 in 1,000-year events at 2190 have not been specifically modelled. Sea level rise allowance for the credible maximum scenario at 2190 (based on the BECC Upper climate estimates) would be 4.82m giving an extreme water level for the overtopping assessment of 8.0m AOD and 8.84m AOD for the 1 in 200-year and 1 in 1,000-year respectively. Scenario with an extreme water level of 8.85m AOD was assessed for the 1 in 10,000-year return period scenario (with more conservative nearshore wave height).

5.4.9. The overtopping rates presented relate to the peak water level at high tide and therefore the duration of such overtopping will typically be limited to approximately 2 hours.

5.4.10. **Plate 5.1** illustrates the overtopping of the main sea defence (HCDF) profile with 10.2m AOD crest adopted in the AMAZON model for the 1 in 10,000-year return period event at 2140 epoch (with RCP8.5 climate change scenario). The very steep faces indicate incipient or broken waves. The start of the calculation is selected at least one or more wavelengths from the toe of the defence in order to capture potential wave reflection effects which are not captured in the TOMOWAC model.

Plate 5.1: Overtopping of the main sea defence (HCDF) in AMAZON model for 1 in 10,000-year at 2140 epoch (UKCP18 RCP8.5)



b) Northern Mound

5.4.11. **Table 5.7** presents the predicted mean overtopping rates at the defence crest for each of the modelled scenarios for the northern mound defence.

Table 5.7: Predicted mean overtopping rates (l/s/m) for the northern mound defence

Return Period (year)	Epoch – Climate Change Scenario	JP Code	Defence Crest (m AOD)	Mean Overtopping Rates (l/s/m)
200	2110 – 95% Medium Emissions (UKCP09)	C3	10.2	0.00
1,000	2110 – 95% High Emissions (UKCP09)	F2	10.2	0.00
10,000	2110 – H++ with Land Motion plus 1m Surge	F1	14.2	0.64

5.4.12. The results in **Table 5.7** show that the proposed defence is not at risk of overtopping for the medium and high emissions climate change scenarios at 2110. The defence is overtopped by the credible maximum H++ climate change scenario at 2110 with mean overtopping rate of 0.64 l/s/m, which is substantially lower than the adopted tolerable threshold. This suggests that the proposed defence at the northern mound is sufficient to manage coastal flood risk during operation of the development up to 1 in 10,000-year return period event.

c) SSSI crossing

5.4.13. **Table 5.8** presents the predicted mean overtopping rates over the top of the SSSI crossing defence for each of the modelled scenarios. Based on the preliminary results for the 1 in 200 and 1 in 1,000-year events at 2140 it was decided not to run the overtopping model for the SSSI crossing for lower climate change scenario or return periods as there would be no or very limited overtopping.

Table 5.8: Predicted mean overtopping rates (l/s/m) for the SSSI crossing defence

Return Period (year)	Epoch – Climate Change Scenario – Cefas JP Case	JP Code	Defence Crest (m AOD)	Mean Overtopping Rates (l/s/m)
200	2110 – 95% Medium Emissions (UKCP09) – Case 2	C3	7.3	0.01
	2140 – RCP8.5 (UKCP18) – Case 7	B3	7.3 / 10.5	3.72 / 0.00
	2190 – RCP8.5 (UKCP18)	B3	7.3 / 10.5	37.01 / 0.00
1,000	2090 – RCP8.5 (UKCP18) – Case 3	F2	7.3	2.95
	2140 – RCP8.5 (UKCP18) – Case 7	F2	7.3 / 10.5	36.04 / 0.00
	2140 – BECC Upper – Case 15	F2	10.5	28.34
	2190 – RCP8.5 (UKCP18)	F2	7.3 / 10.5	216.54 / 0.47
10,000	2090 – RCP8.5 (UKCP18) – Case 3	F1	7.3	45.64
	2090 – H++ with Land Motion plus 1m Surge – Case 8	F1	7.3	170.71

Return Period (year)	Epoch – Climate Change Scenario – Cefas JP Case	JP Code	Defence Crest (m AOD)	Mean Overtopping Rates (l/s/m)
	2140 – RCP8.5 (UKCP18) – Case 7	F1	7.3 / 10.5	289.23 / 1.40
	2140 – BECC Upper – Case 15	F1	10.5	192.55

5.4.14. Results in **Table 5.8** show that the predicted overtopping rates for the 7.3m AOD defence crest (road level) are within the tolerable limits up to 1 in 1,000-year return period events with climate change up to 2090 (end of operation). For scenarios beyond 2090, the predicted mean overtopping rates exceed the guideline tolerable thresholds for all considered return period events.

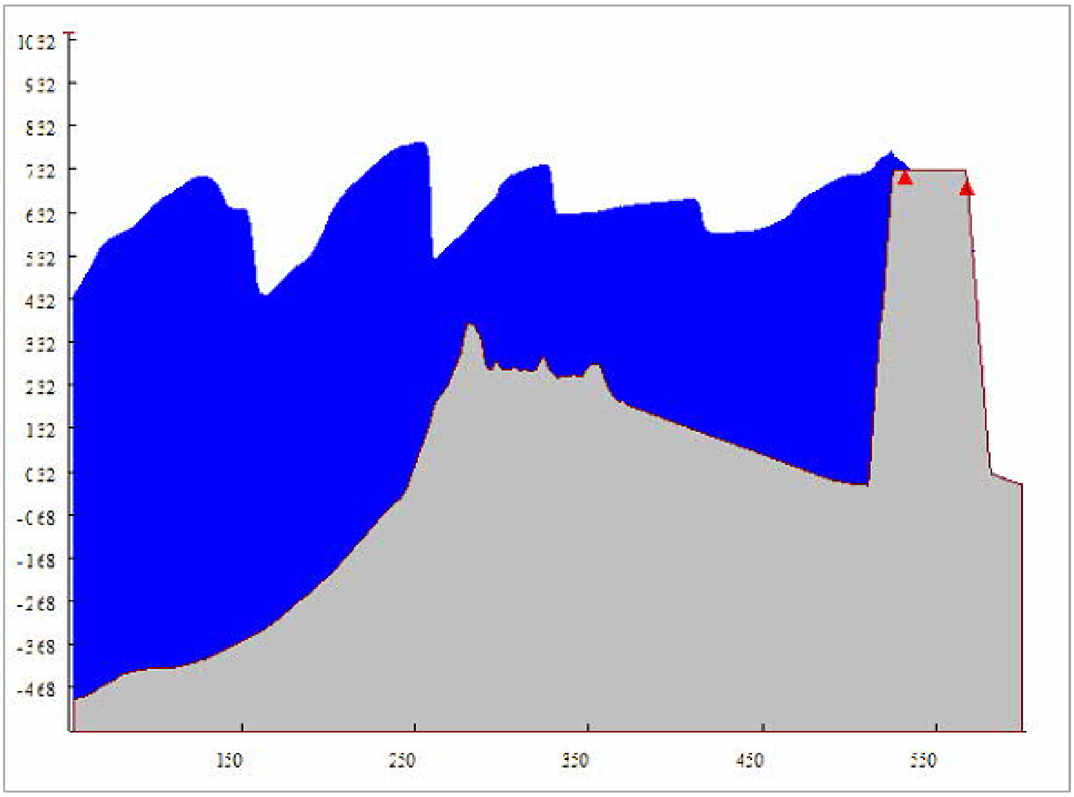
5.4.15. With the adaptive defence at 10.5m AOD the overtopping would be below the threshold for safe vehicle and people access and egress for up to 1 in 10,000-year event (basis of design) up to end of interim spent fuel store decommissioning at 2140 considering reasonably foreseeable climate change allowances.

5.4.16. Results for the credible maximum climate change suggest that, even with adaptive defence, the overtopping would be significant posing risk to people and vehicles on the causeway. In such a case the crossing would have to be closed during the extreme events and further mitigation measures, i.e. warning and forecasting system, emergency response plan should be in place.

5.4.17. **Plate 5.2** illustrates overtopping of the SSSI crossing profile with the crest of 7.3m AOD adopted in the AMAZON model for the 1 in 10,000-year return period event at 2090 (with RCP8.5 climate change scenario). The very steep faces indicate incipient or broken waves.

5.4.18. The start of the calculation is selected at least one or more wavelengths from the toe of the defence in order to capture potential wave reflection effects in AMAZON (which are not captured in the TOMOWAC model).

Plate 5.2: Overtopping of the SSSI crossing in AMAZON model for 1 in 10,000-year at 2090 (RCP8.5)



5.4.19. Further interpretation of the results and possible mitigation measures are discussed in the FRA and the nuclear Safety Case report.

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

6.1.1. Coastal wave overtopping modelling was carried out to inform Sizewell C FRA in support of the DCO application. Modelling considered three representative profiles to test effectiveness of the proposed defences throughout the development lifetime.

6.1.2. Three key return period events were assessed, namely, 1 in 200-year and 1 in 1,000-year to inform the FRA, and 1 in 10,000-year as the basis of design. Appropriate climate change allowances were included for key development phases, adopting reasonably foreseeable scenario based on UKCP18 and credible maximum scenario based on the more conservative BECC Upper projection derived for the Sizewell C project.

6.1.3. The results of the overtopping modelling for the main sea defence (HCDF) indicate following key conclusions:

- During the initial construction phase, when the site would be most exposed with line of defence provided only by the existing shingle beach and sand dunes, the overtopping would be significant, posing risk to the site. Further mitigation measures would be required, e.g. temporary works, or forecasting and warning systems, to enable temporary suspension of construction and evacuation of workers if such an event occurs;
- Once the temporary and design defence has been constructed, there would be no overtopping for the rest of construction phase and well into the operation phase. The defence could still get overtopped from a 1 in 200 and a 1 in 1,000-year event beyond the 2140 epoch, however predicted overtopping rates are below threshold for safe people and vehicle operation and therefore would be managed by staff on site;
- With the adaptive defence the overtopping risk would be limited up to the end of theoretical maximum site lifetime at 2190 considering reasonably foreseeable scenario and up to 2140 considering credible maximum scenario;
- For the 1 in 10,000-year event, predicted overtopping rates are above the set threshold of 5l/s/m at the end of operation phase and beyond. However, the adaptive defence would significantly reduce overtopping up to end of end of theoretical maximum site lifetime at 2190.

6.1.4. The result of overtopping modelling for the SSSI crossing conclude:

- The site is set back from the shoreline and is protected by natural shingle defences and therefore there would be no risk of overtopping during the construction phase;
- Once the causeway is constructed with the defence at 7.3m AOD the crossing would provide safe access and egress for up to end of operation phase for 1 in 1,000-year event. Beyond the operation phase overtopping rates would be significant, however adaptive defence at 10.5m AOD would limit overtopping to safe rates for up to 2140 epoch and 1 in 10,000-year event considering reasonably foreseeable scenario;
- Considering the credible maximum climate change scenarios, the predicted overtopping rates are significant and further mitigation measures would be in place, including closure of the crossing during an event and having emergency response plan in place with clean-up and inspection team to ensure safe crossing following an event.

6.2 Recommendations

6.2.1. Following an earlier review of the different modelling options, AMAZON was selected as best numerical tool to predict overtopping at this stage of the Sizewell C project. Given the uncertainties associated with wave overtopping calculations it is recommended that physical model is used at the design stage.

6.2.2. As stated in **section 1.4** no calibration or validation of the model was undertaken due to lack of sufficient data. It is therefore recommended that, once construction is completed, a monitoring system is put in place to enable performance testing during the 10-yearly reviews. If the physical modelling is undertaken, results could be used to review the overtopping rates predicted with the numerical model in the assessment.

6.2.3. It is recognised that the current best practice methodology for joint probability assessment is the multi-variate method (e.g. Heffernan and Tawn). Although the Join-Sea method was the most appropriate at the time of the study it is recommended that a sensitivity check is carried out in line with the latest guidance and, if required, the joint probability assessment updated at the next stages of works.

6.2.4. If the joint probability assessment is revised, it is also recommended that the wave transformation modelling is revised to account for any potential changes in sea levels or wave heights.

6.2.5. Overall it is recommended that the trajectory of climate change projections is re-assessed at regular (e.g. 10-yearly) intervals and the overtopping rates re-assessed during the reviews to take account of advances in numerical modelling and climate change science.

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APPENDIX A:

‘Sizewell C Flood Risk Assessment Modelling overtopping of sea defences’ Technical Note

RHDHV, February 2017

RIVER, DELTAS & COASTS

HASKONINGDHV UK LIMITED

Note

To	:	[redacted]
From	:	[redacted]
Date	:	23rd February 2017
Copy	:	[redacted]
Our reference	:	PB1452/OD/080/Coastal Overtopping
Subject	**:**	**Sizewell C Flood Risk Assessment** **Modelling overtopping of sea defences**

Purpose

The purpose of this technical note is to describe the approach that will be taken to modelling overtopping of the sea defences at Sizewell C (SZC) in order to inform the Flood Risk Assessment (FRA) that will form a part of the Development Consent Order (DCO) application.

This note covers a number of different aspects of the overtopping modelling, specifically:

- Overall approach to overtopping modelling.
- Overtopping model set up and principles, and how inputs from Cefas will be included within the modelling process, including how uncertainty will be approached.
- The profiles that will be used to represent the main site sea defence, including construction sequencing for assessing risks during the construction phase.
- How the impact of a breach of the main site sea defence will be assessed.
- Model sensitivity checks.

The timings of each phase of Sizewell C are:

- 2017: start of construction (baseline for assessment of flood risk);
- 2025: commissioning (baseline for assessment of construction phase flood risk);
- 2085: end of operation (60 years predicted operational lifetime);
- 2110: end of decommissioning (20-25 years); and
- 2140: interim spent fuel store decommissioned.

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1 Background

1.1 Reason for Technical Note

The Flood Risk Assessment (FRA) for Sizewell C (SZC) must consider the risk of flooding from all sources. One of the potential sources of flooding is wave overtopping of the existing or proposed sea defences during extreme weather conditions including wave and/or surge impacts, both at the present time and into the future with climate change. In addition, the FRA will include an assessment of the risk from a breach of the main site sea defences, which could potentially include wave overtopping and/or full tidal overflowing at the breach location.

For consistency with other sources of flood risk, the overtopping and breach risks must be considered for the following return period events:

- 1 in 200 year event in order to ensure that safe access to, and egress from, the site is maintained (construction only); and
- 1 in 1,000 year event in order to ensure safety of personnel (staff and visitors during construction and operation).

In order to assess the wave overtopping risk, it is proposed to undertake the required assessments using the AMAZON modelling package. The purpose of this Technical Note is to set out the overall approach to wave overtopping. The note includes a number of sections, as listed below:

Section 2: Describes the reasons for selecting the AMAZON modelling package and the way in which the package will be used.

Section 3: Provides an overview of the AMAZON modelling process including input data from Cefas that will be used in the AMAZON model.

Section 4: Defines how uncertainties within the Cefas input data, such as errors in nearshore wave prediction, will be handled within the AMAZON modelling process.

Section 5: Describes sensitivity tests that have already been undertaken on the use of AMAZON as a modelling tool, and additional sensitivity tests that may be completed at a later stage. Some of these sensitivity tests may be specifically aimed at addressing uncertainties within the Cefas input data.

Section 6: Describes the various profiles that will be used within the AMAZON model for each phase of development, including the existing situation, during the construction phase, and on completion of construction (operational phase).

Section 7: Describes how the impacts of a breach of the main site sea defence will be assessed. This will be based on an assumption that such a breach has occurred and the FRA therefore needs to demonstrate what the potential impact of a breach could be. As the main site sea defence is claimed under the nuclear Safety Case, consideration of *how* such a breach could occur is excluded from the FRA.

Section 8 Builds on previous work by Royal HaskoningDHV, notably in respect of climate change and sea-level rise, to describe the approach to undertaking the overtopping assessments efficiently by minimising the number of model runs that need to be carried out whilst ensuring an appropriate level of assessment is completed.

2 Use of AMAZON modelling package

2.1 Introduction

The FRA Scoping Report: Sizewell C Nuclear New Build (Royal HaskoningDHV, 2014) recommended a review of methods for estimating wave overtopping at SZC. The methods that have been considered include:

- **Physical.** Physical modelling of the sea defence may be undertaken at a future stage of design development to assist in assessing the nuclear Safety Case. However, there is no intention to use physical modelling to inform the FRA.
- **EurOtop (empirical).** There are a number of empirical methods developed for estimating wave overtopping, derived from laboratory data (i.e. physical modelling). EurOtop is the most updated, well tested and documented empirical method, and was therefore considered for use in this study. As empirical methods are based on laboratory data, they are only valid for defence profiles and wave/water level conditions that are within the tested range.
- **EurOtop (neural network method).** The EurOtop neural network model is well tested and available for this study. It is also based on laboratory data; the same database that the EurOtop empirical methods were derived from. For this reason, the EurOtop neural network model is also limited to tested defence profiles and wave/water level conditions.
- **Hydrodynamic models.** Considering computational speed, only non-linear shallow water equation models were considered as appropriate for use in this study. AMAZON is the only well tested and available non-linear shallow water equation model.

The following sections provide more information about the AMAZON model, discuss its strengths and weaknesses, and compare it with EurOtop methods mentioned above, in terms of suitability for use in the assessment of coastal flood risk at Sizewell.

2.2 About AMAZON

AMAZON is a high resolution, two dimensional finite volume numerical model capable of simulating supercritical flow and capturing moving hydraulic jumps. It is based on solving the non-linear shallow water equations (Hu et al., 1998; Hu, 2000). AMAZON was designed for 'violent' (referring to moving flow discontinuity) flows, such as hydraulic jumps, tsunamis, bore waves (including tidal bores) and dam breaks. Because of these strengths, AMAZON has also been used for simulating wave run-up and overtopping.

AMAZON-Wavewatch is a one-dimensional software package with a graphic user-interface (GUI) specifically designed for simulating wave overtopping of coastal structures based on the AMAZON model.

AMAZON has been tested for wave overtopping calculations on single slope walls, slope walls with berms, and vertical seawalls (Hu et al., 1998). The computed results compared well with those derived from laboratory tests. In the tested 'standard' profiles, AMAZON performed as well as some empirical methods. A further series of validity tests on AMAZON were undertaken by Hu and Meyer (2005) and Reis et al. (2005). The tests compared AMAZON results with experimental data and other overtopping models, which provided confidence in AMAZON as well as guidance for practical engineering applications.

In 2009, AMAZON was further developed to include a porous layer for rock armour or shingle (Reis et al., 2009). A series of validity tests on the porous layer version were subsequently undertaken by Reis et al. (2011). The paper by Reis et al. (2011) received an Institution of Civil Engineers Award in 2012. AMAZON's capability in considering the effect of a porous layer makes it unique when compared to other overtopping tools.

As input data, AMAZON requires bathymetric information (or cross-section profiles for a 1-D calculation) and incident waves. AMAZON is an unsteady model so random waves can be simulated as well as monochromatic waves. Popular wave spectra including Bretshneider-Moskowitz, JONSWAP and TAM have been built into the AMAZON software. User-defined wave spectrum and measured wave trains are also accepted by AMAZON for incident wave input.

For SZC, wave input data will be based on information provided by Cefas. The way in which Cefas input data will be used is described in Sections 3 (use of Cefas input data) and 4 (addressing uncertainties in Cefas data).

The rest of this section focusses on a more detailed justification for selecting AMAZON as an appropriate modelling tool for SZC.

2.3 Strengths and weaknesses of AMAZON

Shallow water assumption

For wave overtopping, AMAZON has two limitations inherited from the non-linear shallow water equations. These are the shallow water assumption and the wave breaking approximation. The shallow water assumption limits its use to relatively long waves. It requires water depths to be less than one tenth of the wave length, which is not a problem at SZC because wave lengths for extreme waves are greater than 100m long and the water depth at the toe of the defence is less than 10m. Wave breaking in the non-linear shallow water equations is approximated by steep fronts represented by bores. This approach, which ignores the detailed structure of the breaking wave, is often a reasonable approximation, especially in the swash zone.

Ability to model irregular profiles

One of the main weaknesses of empirical overtopping methods (any method that is derived from the results of laboratory tests, including the neural network model) is that the validation of the method is limited by the tested conditions. This introduces additional uncertainty when attempting to use the empirical method to model a defence where the modelled profile is different to the profiles against which the method has been validated (e.g. irregular profiles).

This weakness of empirical methods is a strength of AMAZON. AMAZON considers the effect of every bed-level change and calculates water movement governed by the basic hydraulic principles of mass, momentum and energy conservation laws.

Ability to provide peak overtopping rates

From a time series of simulated overtopping volumes, AMAZON can provide not only mean overtopping rates but also peak overtopping rates. Peak overtopping rate is particularly relevant when assessing the potential risk to people from wave overtopping of sea defences.

2.4 Track record of AMAZON

The technology of using non-linear shallow water equations (NLSWE) for simulating wave overtopping was initially introduced by van Gent (1994). Strictly speaking, AMAZON is not the only NLSWE model for overtopping, but AMAZON is probably the only NLSWE overtopping modelling software designed for commercial use, and is the most used NLSWE model in Europe. The development of AMAZON was initiated by the lack of reliable modelling tools for 'violent' flows. The fact that AMAZON was developed and driven by the industry has made it unique, and the development of AMAZON has been substantially aided by inputs from coastal and maritime engineers, particularly in respect of model validation and safety margins. AMAZON is now benefitting from collaboration with the Laboratório Nacional de Engenharia Civil (Portuguese National Laboratory for Civil Engineering) where a more comprehensive guide on validation conditions is being developed by comparing AMAZON results with those from physical models.

AMAZON has been used extensively by Royal HaskoningDHV for numerous clients, including:

- EDF Energy (Japanese Earthquake Response Flood Modelling);
- UK Environment Agency (North East Coastal Tidal Flood Forecasting and Warning System, Isle of Wight Coastal and Harbour Modelling, Paignton Coastal Flood Risk Assessment);
- Maritime Councils including Havant, Portsmouth and Scarborough; and
- Port Authorities including Port of Dover, Dublin Port and Peel Ports.

2.5 Qualitative comparison of AMAZON with EurOtop for use at SZC

This section provides a qualitative comparison of the use of AMAZON for assessing coastal flood risk at SZC. Some quantitative comparisons have also already been undertaken; these are described in Section 5 of this report.

EurOtop methods are regarded as the UK industry standard for predicting wave overtopping, particularly for 'standard' defence profiles, which have been well tested and are incorporated into the EurOtop database. The primary issue with using EurOtop at SZC is that the defence profiles at SZC are not 'standard'. Figure 1 illustrates the two locations where flooding from wave overtopping could potentially create a problem; neither of these locations have 'standard profiles'.

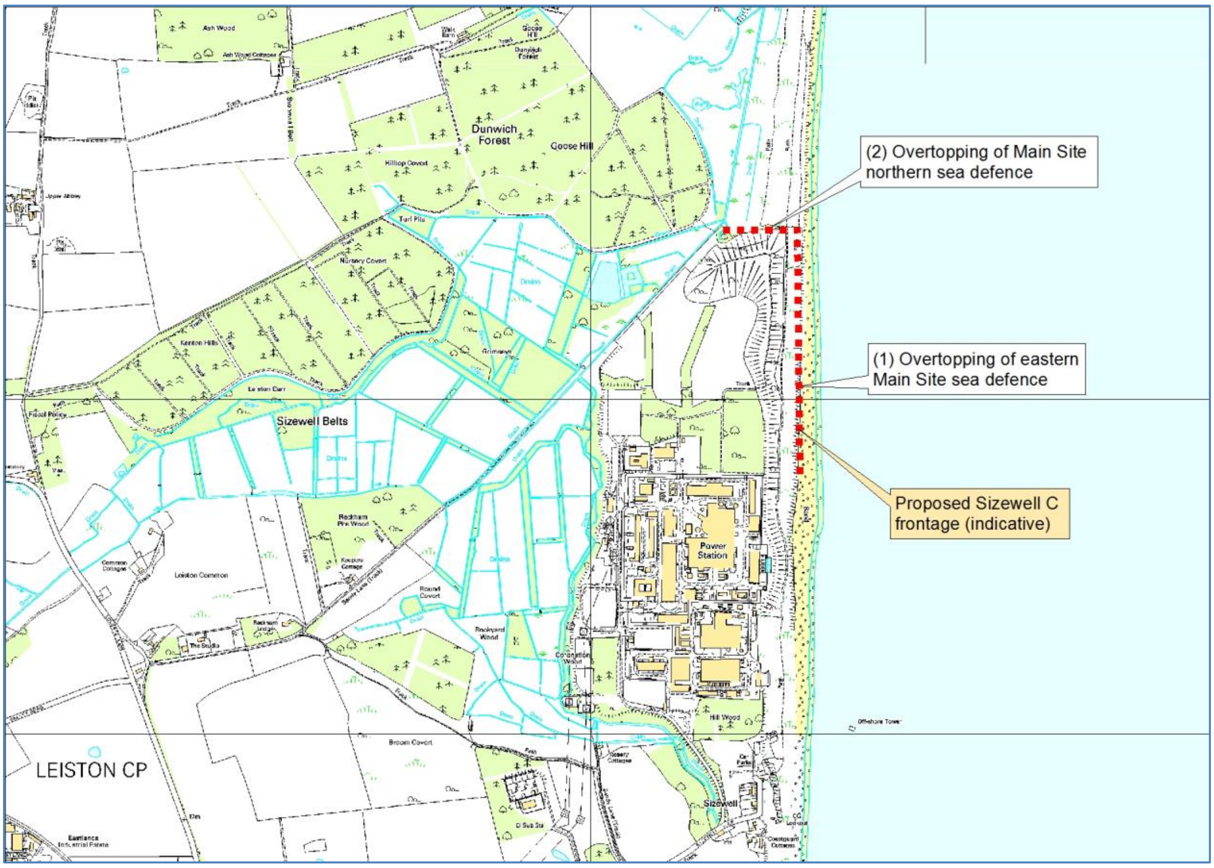

Figure 1: Potential routes for flooding from overtopping

Figures 2 and 3 illustrate the non-standard nature of this coast by showing the different types of profile that are encountered in front of the proposed SZC, from where the development may be exposed to wave overtopping.

Figure 2: Existing frontage of the proposed SZC development

Figure 3: Existing northern frontage of the proposed SZC development

The above figures demonstrate that there is a substantial shingle beach and vegetated sand dunes at all of the potential flood routes involving wave overtopping at SZC. It is important that those features are considered in the wave overtopping analysis as they may have a significant impact on wave propagation and run-up before waves reach the hard sea defences. These defence shapes cannot be properly represented by the EurOtop methods.

For the above reasons, AMAZON software will be used for predicting wave overtopping for the SZC coastal flood risk study.

3 AMAZON modelling process

3.1 Use of CEFAS input data

The TOMOWAC wave model has been developed for investigating wave propagation from offshore to nearshore areas. Cefas has provided Royal HaskoningDHV with outputs from its TOMOWAC wave model at points 300m offshore from SZC. Outputs have been provided for both the 1 in 200 and 1 in 1,000 storms across a range of sea-level rise scenarios, and comprise:

- water levels as a time series of the entire storm duration; and
- wave height (H_s) and wave period (T_p) for the peak of the storm at 5m intervals along each of the selected profiles.

The TOMOWAC data will be used as input data for the AMAZON model. Preliminary runs will be undertaken using the 'still water' peak tide levels as initially provided by Cefas. In addition, the first model runs to be carried out will use high return periods (i.e. 1 in 1,000 storm) and high climate change scenarios (sea-level rise). If these initial model runs show that wave overtopping is significant, then additional model runs will be carried out using progressively less extreme input data (i.e. 1 in 200 storm and smaller allowances for sea-level rise).

The reason for taking this approach to modelling wave overtopping is to keep the number of model runs required to a reasonable level. If at any point in undertaking the model runs a point is reached where wave overtopping risk is considered to be insignificant (see Section 3.2), or even reduces to zero, then there is no value in continuing to model progressively less extreme scenarios as these will, by definition, also show insignificant (or zero) overtopping risk.

It should also be noted that by using 'still water' peak tidal input data, the above approach to wave overtopping modelling is a conservative one. Should the level of overtopping calculated merit a more detailed assessment, time series tidal data will be obtained from Cefas and used in additional AMAZON model runs.

The proposed modelling sequence is described in more detail in Section 8 of this note.

3.2 AMAZON outputs

Outputs from the AMAZON model will comprise overtopping rates and volumes at the assessed profile locations. Where overtopping is predicted to occur, an assessment will be made as to whether the overtopping rates are tolerable. This assessment of 'tolerability' will include:

- Using standard guidance, such as EurOtop, to define whether overtopping rates could put people at risk. This assessment of risk will take into account the proposed use and access arrangements for the area where overtopping is occurring (such as whether access to the area affected is controlled, falls within the SZC security zone, etc).

- Consulting with the main site design team to liaise over drainage arrangements for the area behind the main sea defence to relate the predicted overtopping rates to the capacity of the drainage system and to confirm it has adequate capacity.

4 Addressing uncertainties in TOMOWAC model outputs

The primary potential uncertainty in the TOMOWAC model outputs that may affect the AMAZON overtopping modelling outputs relate to predicted nearshore wave heights. This uncertainty will be addressed by adding the estimated mean error on top of the modelled wave height (by percentage) and adopting offshore wave periods for the overtopping calculation (see further discussion on wave periods below).

It has previously been reported, in TR319 (BEEMS, 2015) that 10% is a conservative estimate for model accuracy with respect to predicted wave heights for all waves.

TOMOWAC is a spectral wave model which transforms directional wave spectra but not directly wave heights or periods. The JONSWAP wave spectrum is assumed offshore as input to the TOMOWAC model. To define a JONSWAP spectrum requires significant wave height (H_s), peak wave period (T_p) and peak enhancement factor (γ which is assumed to be a fixed value of 3.3). At an output location nearshore, H_s and T_p may be 'extracted' from the modelled spectrum. Figure 4 presents a modelled wave spectrum nearshore for Amazon for a 1 in 1,000 year wave condition under sea-level rise scenario of '95% medium emissions at 2110'. Figure 4 shows that the wave spectrum has been transformed from a single peak JONSWAP spectrum offshore to a two-peaked spectrum nearshore.

Amazon also requires wave spectrum as its input, so in theory there is no need to use wave height and period. However, in practice, it is time-consuming to use modelled wave spectra as Amazon input since the modelled spectrum varies from scenario to scenario. Therefore, it is proposed to adopt the JONSWAP spectrum with offshore wave period for Amazon input. Figure 4 shows that the JONSWAP spectrum with the offshore wave period has more wave energy in the lower frequency region than the TOMOWAC modelled spectrum with the nearshore wave period ('extracted' from the modelled nearshore wave spectrum). This means that the Amazon model results using JONSWAP with offshore wave periods are conservative. It is also proposed that the key Amazon models (the key models are likely those that produce the highest overtopping rates) will be verified with modelled nearshore wave spectra to ensure that the Amazon results are conservative.

As explained in TR319 (BEEMS, 2015), a number of conservative assumptions have been made in the boundary conditions. 95% extreme values for H_s and water level have been used, in order to derive the joint probabilities, e.g. 1:10,000 year event for H_s 7.64m ± 0.50 = 8.14m and 5.06m ± 0.14 = 5.20m for water levels. In addition to this inherently conservative approach, for many scenarios, wave heights are increased by either 10 or 15% to allow for possible future changes in storminess.

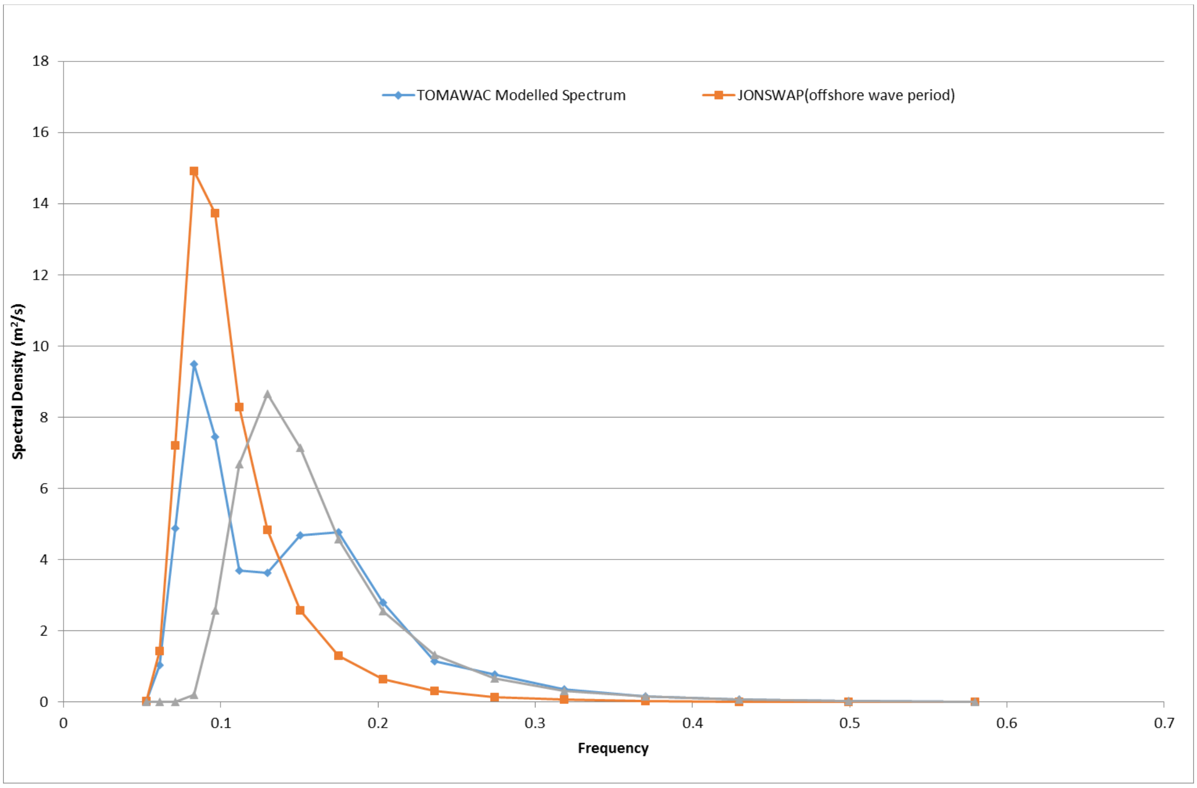

Figure 4: Comparison of nearshore wave spectra

The above approach for dealing with uncertainty in the Cefas model data is considered to be a reasonably conservative approach.

5 Sensitivity checks

A number of sensitivity checks have already been completed using the AMAZON model. These checks, their results and the conclusions that have been drawn are described below. Potential additional sensitivity checks are also discussed.

5.1 Berm removal

Reason for undertaking sensitivity check

The defence/beach profile varies along the Sizewell frontage and in some locations there is a 'sacrificial berm' immediately landward of the active zone. There is the possibility that this berm could be washed away ('sacrificed') during a storm surge and its removal could potentially result in increased overtopping risk at the main sea defence. Checks have therefore been carried out in order to understand the sensitivity of the modelled overtopping rates to the existence or loss of the berm.

Modelling approach

A single profile, S4 was selected for this sensitivity check (Figure 5). Figure 6 shows the defence and beach profile at location S4, extracted from LiDAR (2010) and ground level surveys (2013-2014).

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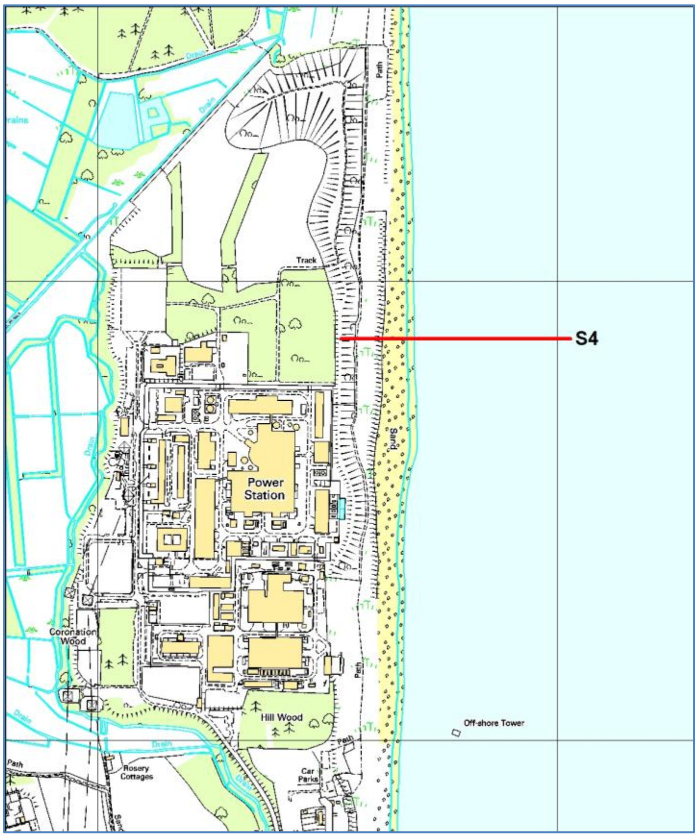

Figure 5: Location of profile S4 for the sensitivity runs

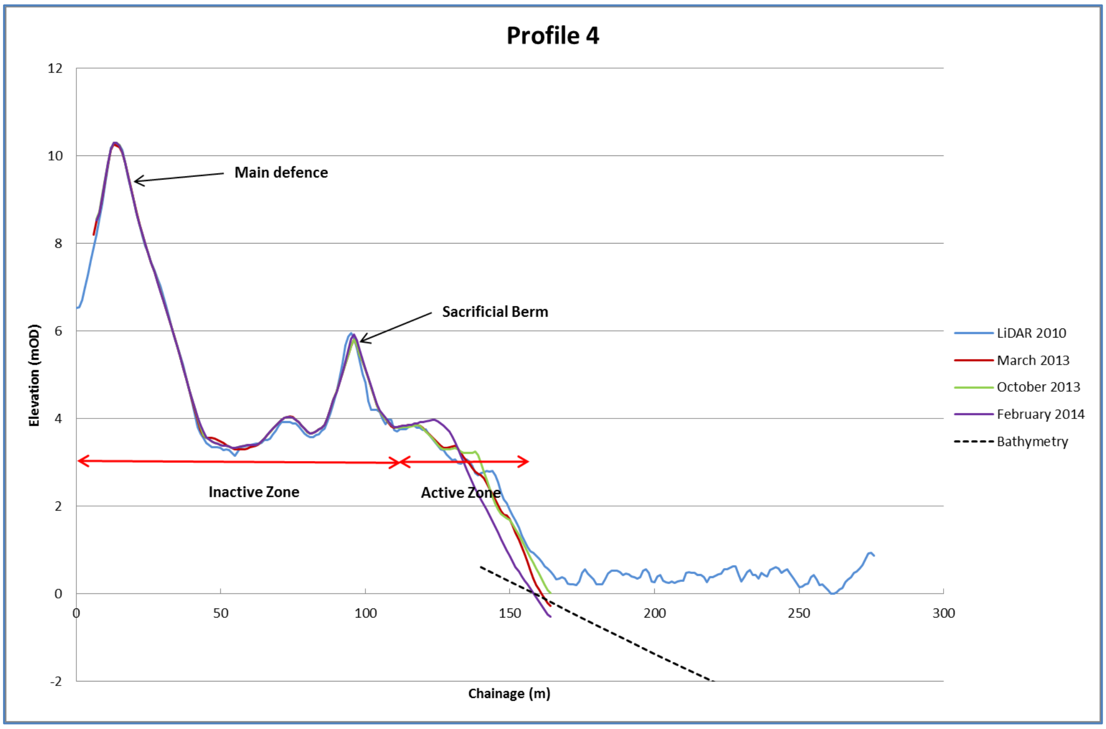

Figure 6: Defence/beach profiles extracted from LiDAR and ground level survey data at S4

Profile S4 comprises an 'active zone', where beach morphology is subject to seasonal changes driven by wave climate and tidal variations, and an 'inactive zone', where the ground is rarely flooded by overtopped sea water.

The three ground level surveys closely match each other within the 'inactive zone', and therefore in this zone the modelled profile is based on the ground survey data rather than LiDAR. For the 'active zone', the representative profile has been compiled from the lowest points along each of the ground level surveys and the LiDAR data, providing a conservative beach profile for the purposes of the wave overtopping assessment. The solid black line in Figure 7 describes the compiled representative defence and beach profile used in this set of model sensitivity runs for the 'with berm' scenarios.

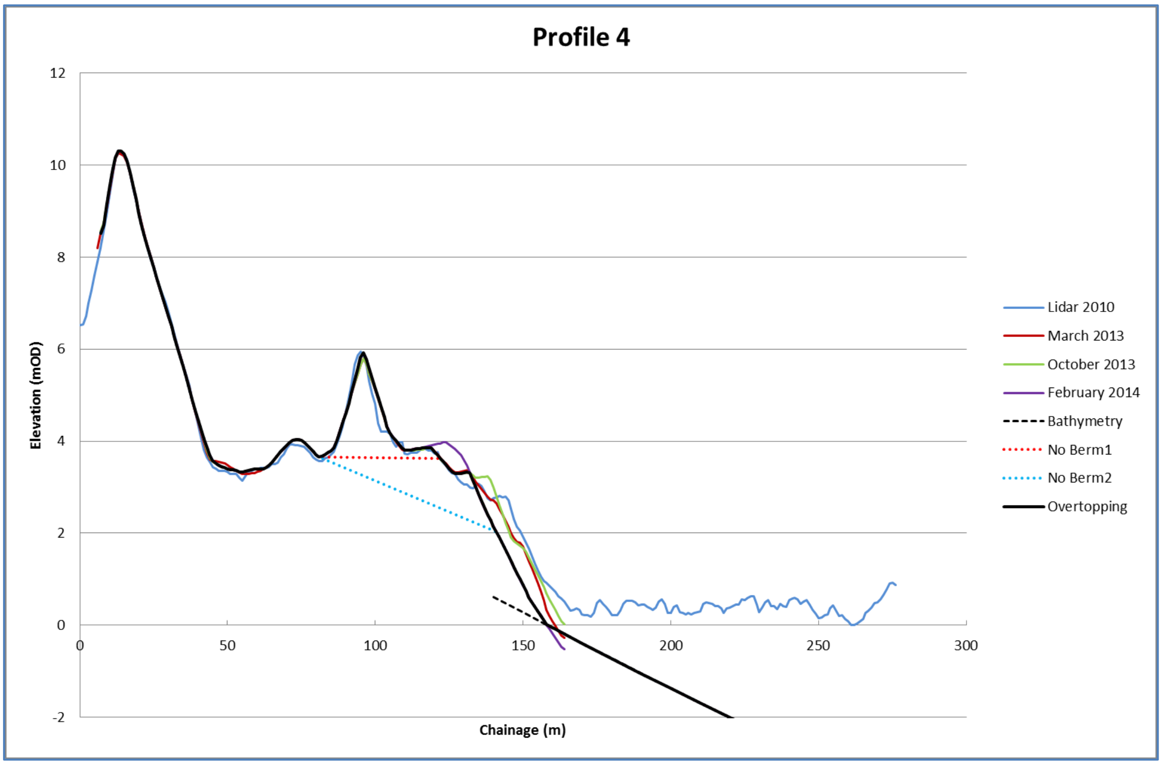

Figure 7: Representative defence/beach profile (black thick line) at S4 for overtopping analysis

The 'sacrificial berm' is a ridge that reaches approximately 6mAOD at chainage 95m (Figure 7). There are two options for removal of this berm to represent the 'without berm' scenario in the model. Given that it is difficult to determine how the berm would fail, and therefore which of the options is most appropriate, both options have been modelled to establish the worst case scenario for overtopping rates.

'No berm 1' assumes that the sacrificial berm is removed with an approximately horizontal base at the same elevation as the land directly behind the berm (Figure 7). 'No berm 2' assumes that a slope develops between the base at the back of the berm and a point in the active zone.

Input data was obtained from the Cefas TOMOWAC model for their model scenario F2[1].

[1] A joint combination of 1 in 1,000 year return period and sea-level rise of 1.55m (BECC Lower at 2110); combination code refers to the combination of wave direction, wave height, tidal peak and return period used in the Cefas TOMOWAC modelling, and referenced in the spreadsheet '2110_HS_WL_boundary conditions.xlsx' provided by Cefas.

Results and conclusions

Table 1 presents the predicted mean overtopping rates at the location of the main defence for both removal scenarios. The results show that the predicted volumes of water that spill over this point are similar for the two options.

Table 1: Overtopping rates for two berm removal scenarios (for 1 in 1,000 year return period with a climate change scenario of 'BECC Lower 2110')

Berm scenario	Predicted mean overtopping rate (l/s/m)
No berm 1	0.482
No berm 2	0.550

Based on the results presented in Table 1, the conclusion from this sensitivity test is that 'No berm 2' (sloping profile) produced the worst overtopping, although the difference in overtopping rates between the two berm removal options is small. Given that the sloping profile is the most conservative for overtopping, it is therefore recommended to be carried forward for all profiles modelled for overtopping in the FRA.

5.2 Beach sediment

Reason for undertaking sensitivity check

As noted in Section 2.2, one of the developments of the AMAZON modelling software has been to allow it to model porous layers, such as shingle. This is relevant to the Sizewell frontage because the frontage has a substantial upper beach composed of shingle (Figures 2 and 3). However, the exact depth and nature of the interface between shingle and any underlying sand layers is not known and, indeed, highly likely to vary along the length of the frontage. Therefore, in order to understand the potential impact of shingle on the AMAZON wave overtopping model, sensitivity tests have been carried out under extreme water level and wave conditions at profile S4.

Modelling approach

In order to assess the effect of a shingle layer, initial runs were carried out to identify the water level and wave conditions that would produce overtopping. It was found that the 1 in 1,000 year return period event with the 'BECC Lower 2110' climate change scenario produced overtopping of the existing defence. Therefore, this condition was used to investigate the effect of a shingle layer at profile S4.

The input data used in AMAZON, comprising water level and nearshore wave conditions, are presented in Table 2. This input wave data was provided by Cefas from the TOMOWAC model.

Table 2: Input data for beach sediment sensitivity checks (for 1 in 1,000 year return period and 'BECC Lower 2110' climate change scenario)

Cefas combination code	Wave height H_s (m)	Wave period T_p (sec)	Water level (m AOD)	Water depth (m)
B2	3.77	12.25	5.17	9.67
F2	3.78	11.40	5.57	9.24
C2	2.87	9.78	5.77	8.08

Note: C2 is the combination of highest water level and lowest wave height; B2 is the combination of mid-range water level and wave height; and F2 is in between C2 and B2

The beach profile for S4 without the 'sacrificial' berm ('No berm 2', Section 5.1), was adopted in this assessment. An approximately 1m thick shingle layer was introduced in the AMAZON model (Figure 8). This representation assumes that the shingle layer is unlikely to extend up to the slope of the existing flood defence.

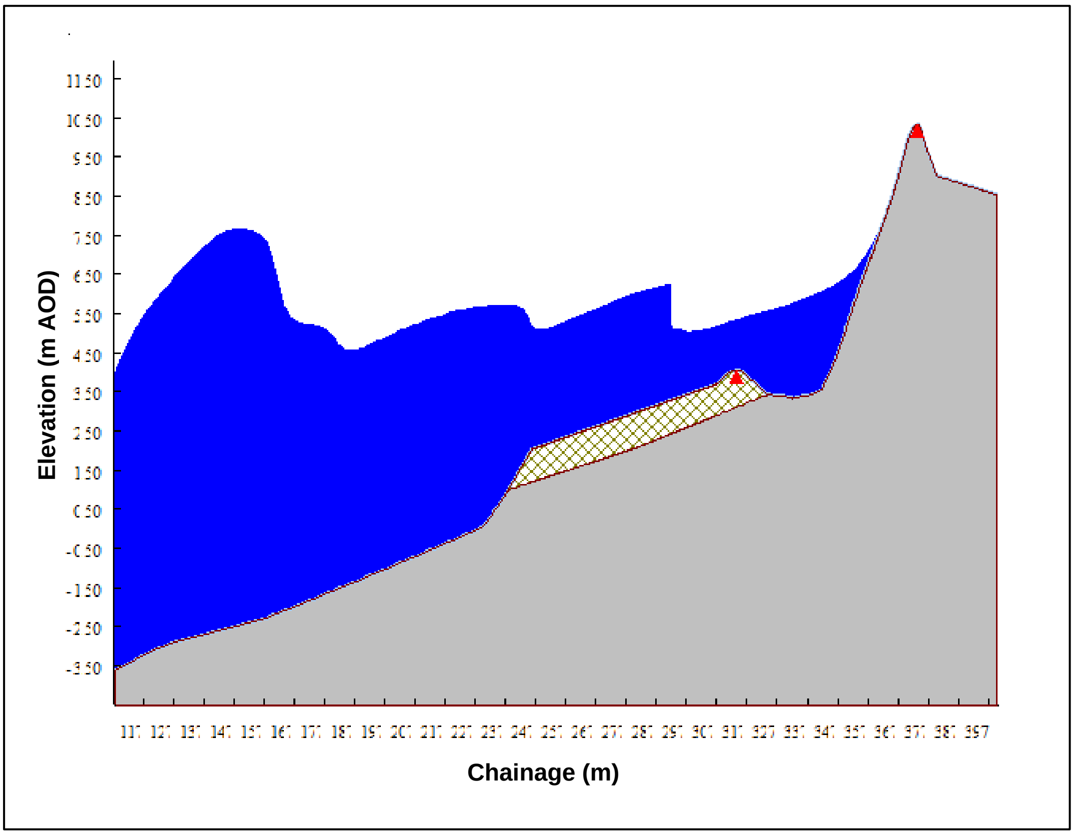

Figure 8: Profile used in AMAZON model for a representative profile at S4 with shingle layer

Results and conclusions

Table 3 presents the predicted mean overtopping rates for the modelled beach profile with and without a shingle layer. The results show that the difference between with and without scenarios was marginal; a maximum of 3.6% for combination F2.

Table 3: Predicted mean overtopping rates for a 1 in 1,000 year return period event with a climate change scenario of 'BECC Lower 2100'

Combination code[2]	Mean overtopping rates (l/s/m)	
	Without shingle	With shingle
B2	0.17	0.17
F2	0.55	0.53
C2	0.01	0.01

Based on these results, it is concluded that a shingle layer has only a negligible effect on the overtopping rates under the conditions of 1 in 1,000 year return and a climate change allowance to year 2110. This may be explained by the fact that the shingle layer was entirely submerged at most times under the tested extreme water level, and thus could not offer any storage to absorb run-up waves. As a result, we recommend that a shingle layer is not included in Amazon models and this approach would give slightly conservative results.

5.3 Offshore sand banks

Reason for undertaking sensitivity check

There is the potential for the offshore bathymetry at the Sizewell frontage to change over time. In particular, sand banks offshore from Sizewell could migrate or disappear. Should this occur, the nearshore wave climate might be affected. The purpose of this sensitivity check was to determine whether the disappearance of the offshore sand banks could have a significant impact on wave overtopping.

Modelling approach

Cefas provided TOMOWAC outputs at 30 cross sections as shown in Figure 9. The wave output was provided at 5m intervals along each of these cross sections. The wave height at a distance of one wave length from the waters' edge is used as the input condition for the AMAZON modelling, and this is therefore the location where wave height sensitivity to offshore sand bank depletion was measured. TOMOWAC data was provided for two scenarios: 'baseline' with the sand bank in place and 'Low 5' where the offshore sand banks are lowered by 5m and the sediment is assumed to be lost from the system entirely (as opposed to being redistributed across the sea bed).

[2] Combination code refers to the combination of wave direction, wave height, tidal peak and return period as used in the Cefas TOMOWAC modelling, and referenced in the spreadsheet '2110_HS_WL_boundary conditions.xlsx' provided by Cefas.

Comparison of 'baseline' with 'Low 5' and 'Shallow South Trough' (ST1) scenarios

Tables 4 to 7 compare the nearshore wave heights at cross sections S4 (adjacent to Sizewell C) and B10 (just north of Minsmere Sluice). B10 was chosen as an additional point as it is close to Minsmere sluice, a common reference point for the Minsmere frontage. The results show that the TOMOWAC model predicted lower nearshore waves at S4 for the 'Low 5' and 'Shallow South Trough' (ST1) scenarios compared to the 'baseline' scenario. This is due to the change in the shoaling effect caused by the bank which coupled to wave propagation means that waves were slightly reduced in the location near the station frontage and increased slightly elsewhere. A more complete analysis is available in TR319 Ed2.

It was noticed that the 'Shallow South Trough' scenario produced higher nearshore waves at B10 than the 'baseline' and 'Low 5' scenarios.

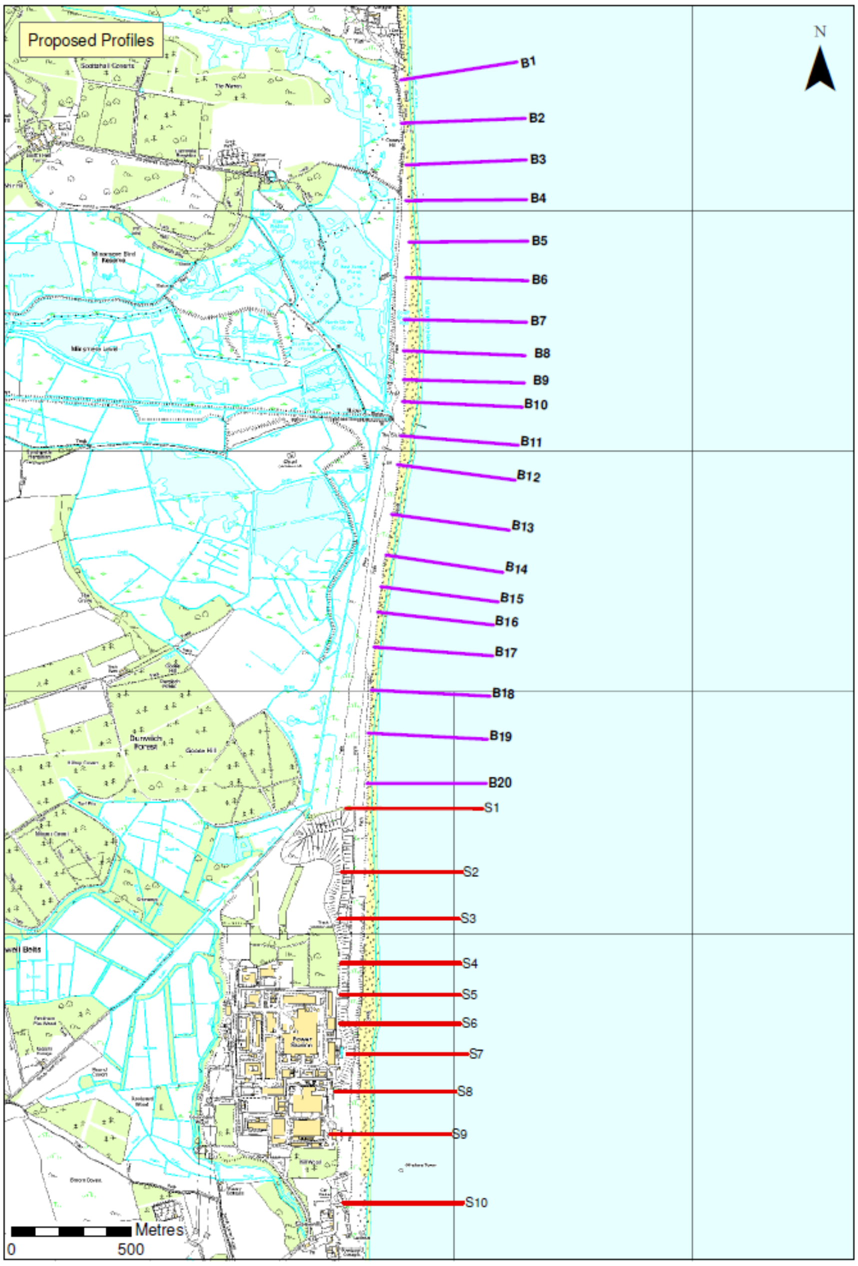

Figure 9: Locations of sections for wave model outputs (red lines cover the Sizewell frontage and the purple lines cover the Minsmere coast)

Table 4: Modelled wave heights at S4 (climate: 2008) (one wave length from shore)

Cefas Joint Combination codes (for 1 in 1,000 years)	Water Level (m AOD)	Baseline	Low5	ST1	Difference			
					BL – Low5	BL / Low5	BL – ST1	BL / ST1
A2	1.89	2.16	2.27	1.84	-0.11	0.95	0.32	1.17
E2	3.03	2.78	2.63	2.02	0.14	1.05	0.76	1.38
B2	3.62	3.00	2.81	1.85	0.18	1.07	1.14	1.62
F2	4.02	3.15	2.98	1.98	0.17	1.06	1.17	1.59
C2	4.22	2.61	2.53	1.93	0.08	1.03	0.69	1.36

Table 5: Modelled wave heights at B10 (climate: 2008) (one wave length from shore)

Cefas Joint Combination codes (for 1 in 1,000 years)	Water Level (m AOD)	Baseline	Low5	ST1	Difference			
					BL – Low5	BL / Low5	BL – ST1	BL / ST1
A2	1.89	2.36	2.10	2.64	0.26	1.12	-0.27	0.90
E2	3.03	2.93	2.63	3.13	0.30	1.11	-0.21	0.93
B2	3.62	3.18	2.83	3.39	0.35	1.12	-0.21	0.94
F2	4.02	3.29	3.03	3.42	0.26	1.09	-0.13	0.96
C2	4.22	2.57	2.51	2.46	0.06	1.02	0.11	1.04

Table 6: Modelled wave heights at S4 (climate: BECC Lower 2110) (one wave length from shore)

Cefas Joint Combination codes (for 1 in 1,000 years)	Water Level (m AOD)	Baseline	Low5	ST1	Difference			
					BL – Low5	BL / Low5	BL – ST1	BL / ST1
A2	3.44	2.97	3.02	2.62	-0.05	0.98	0.35	1.13
E2	4.58	3.51	3.56	3.14	-0.05	0.99	0.37	1.12
B2	5.17	3.77	3.62	3.00	0.15	1.04	0.76	1.25
F2	5.57	3.78	3.64	2.76	0.15	1.04	1.03	1.37
C2	5.77	2.87	2.87	2.58	0.00	1.00	0.29	1.11

Table 7: Modelled wave heights at B10 (climate: BECC Lower 2110) (one wave length from shore)

Cefas Joint Combination codes (for 1 in 1,000 years)	Water Level (m AOD)	Baseline	Low5	ST1	Difference			
					BL – Low5	BL / Low5	BL – ST1	BL / ST1

A2	3.44	3.13	2.78	3.37	0.35	1.13	-0.24	0.93
E2	4.58	3.64	3.32	3.89	0.31	1.09	-0.25	0.93
B2	5.17	3.85	3.59	4.08	0.25	1.07	-0.23	0.94
F2	5.57	3.87	3.67	3.90	0.21	1.06	-0.03	0.99
C2	5.77	2.83	2.78	2.69	0.05	1.02	0.14	1.05

Offshore to nearshore wave transformation

Tables 8 to 11 present the offshore and nearshore wave heights at S4 and B10. The results show a consistent pattern of:

- The C2 combination (highest water level with lowest waves) gives higher nearshore/offshore transformation ratios than the A2 combination (lowest water level with highest waves).
- The climate change scenario 'BECC Lower 2110' gives higher nearshore/offshore transformation ratios than the '2008' climate scenario.

The nearshore/offshore wave transformation ratio varies between 0.3 and 0.8 in the examined cases. This indicates that the wave transformation ratio largely depends on the water level at the time of wave propagation.

Table 8: Comparison of offshore and inshore wave heights at S4 (climate: 2008)

Combination Codes (for 1 in 1,000 years)	Water Level (m AOD)	Wave Height (m)		Nearshore / Offshore
		Offshore	Nearshore (Baseline)	
A2	1.89	7.1	2.16	0.30
E2	3.03	6.28	2.78	0.44
B2	3.62	5.21	3.00	0.57
F2	4.02	4.47	3.15	0.70
C2	4.22	3.23	2.61	0.81

Table 9: Comparison of offshore and inshore wave heights at B10 (climate: 2008)

Combination Codes (for 1 in 1,000 years)	Water Level (m AOD)	Wave Height (m)		Nearshore / Offshore
		Offshore	Nearshore (Baseline)	
A2	1.89	7.10	2.36	0.33
E2	3.03	6.28	2.93	0.47

B2	3.62	5.21	3.18	0.61
F2	4.02	4.47	3.29	0.74
C2	4.22	3.23	2.57	0.80

Table 10: Comparison of offshore and inshore wave heights at S4 (climate: BECC Lower 2110)

Combination Codes (for 1 in 1,000 years)	Water Level (m AOD)	Wave Height (m)		Nearshore / Offshore
		Offshore	Nearshore (Baseline)	
A2	3.44	7.81	2.97	0.38
E2	4.58	6.91	3.51	0.51
B2	5.17	5.73	3.77	0.66
F2	5.57	4.92	3.78	0.77
C2	5.77	3.55	2.87	0.81

Table 11: Comparison of offshore and inshore wave heights at B10 (climate: BECC Lower 2110)

Combination Codes (for 1 in 1,000 years)	Water Level (m AOD)	Wave Height (m)		Nearshore / Offshore
		Offshore	Nearshore (Baseline)	
A2	3.44	7.81	3.13	0.40
E2	4.58	6.91	3.64	0.53
B2	5.17	5.73	3.85	0.67
F2	5.57	4.92	3.87	0.79
C2	5.77	3.55	2.83	0.80

Conclusion and recommendation

The ‘baseline’ scenario predicted higher nearshore waves than the ‘Low 5’ scenario. It is therefore recommended that the ‘baseline’ scenario is taken forward for assessment in the FRA model runs.

6 Profiles to be used in overtopping modelling through development lifespan

6.1 Development timeline

The following timeline and key dates for construction, operation and decommissioning of SZC will be adopted in the overtopping modelling.

- 2017: start of construction (baseline for assessment of flood risk);
- 2025: commissioning (baseline for assessment of construction phase flood risk);
- 2085: end of operation (60 years predicted operational lifetime);
- 2110: end of decommissioning (20-25 years); and
- 2140: interim spent fuel store decommissioned (consistent with HPC FRA).

For the construction phase, all relevant existing profiles and all construction sequence profiles will be modelled at the year 2025, which is the target year for completion of construction. This has been chosen because the input data to overtopping will include the whole of the (relatively small) increase in sea level expected during the construction phase and so will provide a conservative baseline assessment of overtopping risk during construction.

For the operational phase, it is necessary to assess the overtopping for a range of dates in order to understand the potential impacts that climate change could have on risk. These potential impacts, which include relative sea-level rise, storm surges, and wind and waves, are fully described in Royal HaskoningDHV's Climate Change Note (Royal HaskoningDHV, 2015). For the operational phase, the range of dates extends from commissioning (at the completion of construction in 2025), through the end of decommissioning (2110), to complete removal of the spent fuel store (2140).

6.2 Locations and forms of profiles

The outputs of Amazon will be provided for up to five profiles along the Sizewell frontage. These are profile S1 in the north to profile S5 at the point where SZC meets Sizewell B, as shown in Figure 10 below. Profiles S6 to S9, located in front of Sizewell A and Sizewell B, will not be considered. This is because the platform height of SZC will be at an elevation of 7.3m AOD, which is about 1m higher than the Sizewell B platform. Hence, there will be no feasible flood route from either Sizewell A or Sizewell B to SZC.

Wave overtopping of the coastline further to the north (north of Goose Hill) will also not be assessed as a part of the FRA, as the volume of overtopping from waves alone will not be significant enough to present a risk to SZC. However, 'overflowing' or 'green-water overtopping' of natural defences to the north and south will be considered as a part of the breach assessments (as a separate workstream to the wave overtopping assessments)[3].

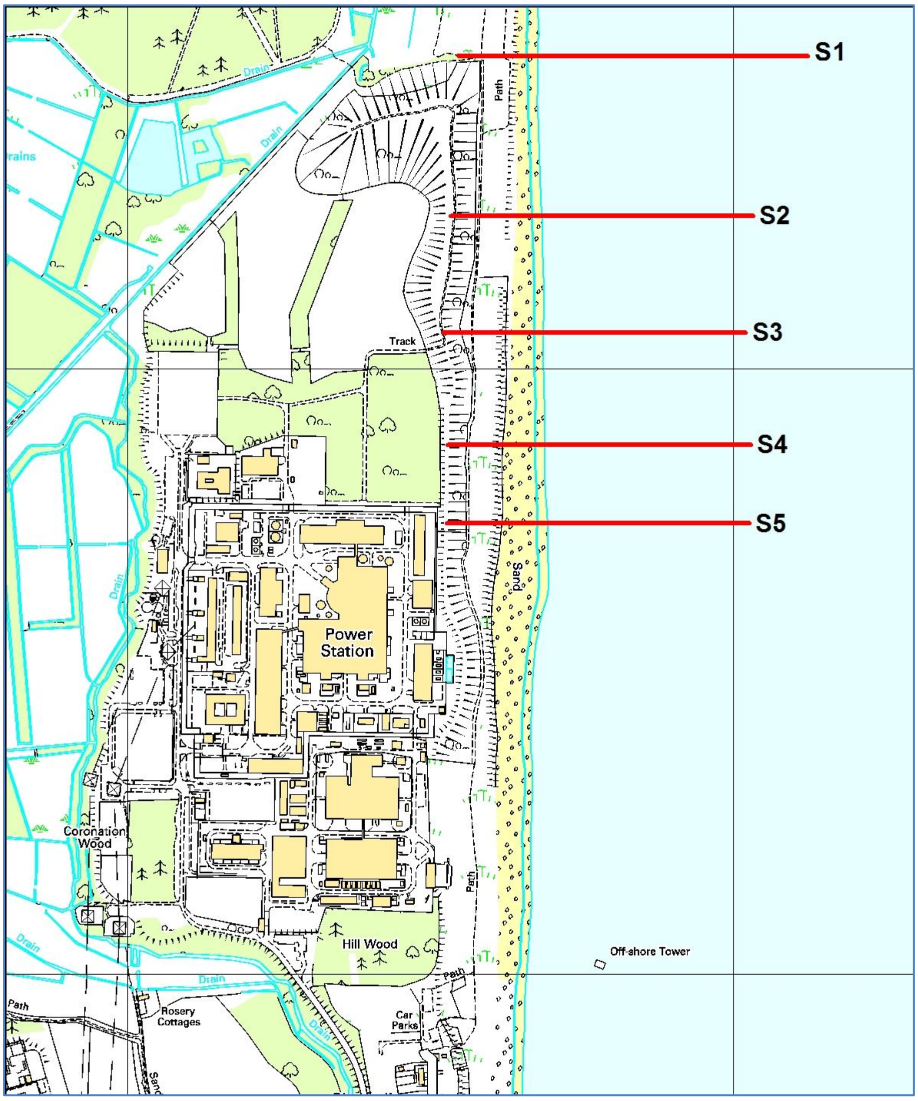

Figure 10: Profiles selected for wave overtopping

[3] 'Overflowing' can be considered as the overtopping case where still water tidal levels exceed the defence crest height, and thus potentially significant volumes of water can overtop a defence, compared to the relatively low volumes that are experienced with wave overtopping.

.For each of locations S1 to S5 it will be necessary to define the form of the profile, so that the volume of wave overtopping can be modelled using AMAZON, and the risks from wave overtopping understood and mitigated. The remaining sections of this Technical Note set out how the various profile forms will be defined, and the information that will be used to inform digitising the profiles into the AMAZON wave overtopping model.

6.2.1 Existing situation

For the existing situation, profile forms have been derived using LiDAR data, as previously discussed with EDF and presented to stakeholders[4]. Along the section of the Sizewell frontage incorporating profiles S4 and S5, there is an artificial 'sacrificial berm' feature, seaward of the existing main sea defences. Profiles S1 to S3 do not contain this artificial berm, and the natural profile rises to between 4.3m and 4.7m AOD. In between the sacrificial berm and the main sea defence there is a plateau at approximately 3m AOD.

During an extreme storm this berm could be destroyed ('sacrificed') and, should the storm be prolonged, there is a risk that the existing main sea defence could be exposed to wave action. As described in Section 5 of this report, preliminary modelling has considered potential wave overtopping for the two different 'berm removed' scenarios shown in Figure 7. This preliminary modelling showed marginally increased overtopping for 'Berm removal 2' compared to 'Berm removal 1'.

Therefore, for the existing situation it is proposed to model the berm removed to a sloping form ('Berm removal 2' scenario in Figure 7). This is the conservative profile form and also consistent with the approach taken by Jacobs in their preliminary design of the sea defences. For profiles S1 to S5, the actual levels of the sloping form may vary as for each profile the landward end of the slope will be taken to coincide with the hinterland level immediately behind the berm with an appropriate point to seaward. The point of connection of the seaward end of the slope will be based on expert judgement of the elevation that 'best-fits' the natural profile seaward of the berm.

It should also be noted that this approach assumes that the material that currently forms the berm is lost entirely to the system. In reality, the material would be deposited elsewhere, and this deposition could have a dissipative effect on waves. A conservative approach will therefore be taken by assuming that the material is lost to the system.

Beach erosion in front of the 5m berm will not form a part of the FRA run-up modelling. This approach has been catered for by the selective use of beach survey data whereby the lowest recorded levels from each beach survey have been collated into a single 'worst case' dataset.

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[4] SZC FRA Coastal Workshop, 13 November 2014

At all profiles, it is proposed to run the 1 in 1,000 event initially. The need to run the 1 in 200 event will then be reviewed depending on the results from the 1 in 1,000 event. Should there be no overtopping during the 1 in 1,000 event then clearly there is no point in assessing the 1 in 200 event.

The five profile shapes to be used to represent the existing situation at Profiles S1 to S5 are shown in Appendix A. Profile S5 will be confirmed once EDF has provided additional information on the tie in between the Sizewell B and SZC defences.

6.2.2 Construction phase

There are a number of reasons for assessing the risk from wave overtopping during the construction phase. These include understanding:

- the risk to construction workers and plant during construction;
- the risk to construction materials storage areas; and
- whether there is increased risk to areas around SZC during construction.

In order to assess overtopping risk during construction, it is therefore necessary to consider the potential profile form at each point during the construction sequence. Information on the proposed construction sequencing has been provided by EDF (Jacobs, 2014a) and the sections below use that information to describe the various profiles to be modelled at each stage of construction.

It should be noted that, at present, there are two separate construction sequence options under consideration; Option 1A and Option 1B. Both of these options may require assessment of overtopping risk, and are described in Appendix B. When considering the below descriptions, reference should be made to the figures included in Appendix B. These figures have been taken directly from the Jacobs report to EDF, where they are included as their Figures A.1, A.2 and A.3. Within each of Jacobs' main figures there are four separate sketches, showing sequential stages within the overall construction phase.

As noted above, each of the profiles will initially be modelled for the 1 in 1,000 event in 2025, and then for the 1 in 200 event at 2025, if necessary.

6.2.3 Operational phase

For the operational phase, the profiles will be defined by the sea defence design proposed by EDF as documented in Jacobs' sea protection report (Jacobs, 2014b). AMAZON modelling will be carried out to provide additional information on the overtopping risk, for comparison with Jacobs' EurOtop assessments, as well as providing a further independent check of overtopping risk across a range of return period events.

The Jacobs sea protection design report proposes a number of options for the sea defence. Subsequent to that report, the preferred option has now been defined as Option 2.1. For this defence form, there are two proposed defence levels. Initially, the defence will be constructed so that the hard defence crest is at 10.0m AOD. There will then be an option for increasing this height to 14.0m AOD in the future, should rates of sea-level rise dictate.

In addition to the two crest levels, there are also options for the 'finish' on the defence. Initially, the defence will be landscaped to reduce visual impact and improve accessibility. However, there will be the option for the landscaping to be removed in the future, leaving the defence exposed as a rock revetment, to increase roughness and improve hydraulic performance. The landscaping adds 0.2m to the defence crest level for both the initial and future hard defence levels, increasing the crest levels to 10.2m AOD and 14.2m AOD, respectively (Figure 11). For the purposes of the AMAZON modelling, it will be assumed that this landscaping layer is impermeable. In terms of the topography between the main sea defence and the 5m berm, for FRA purposes, it will be assumed that this will be maintained, via a formalised management plan, as a plateau at 4m AOD.

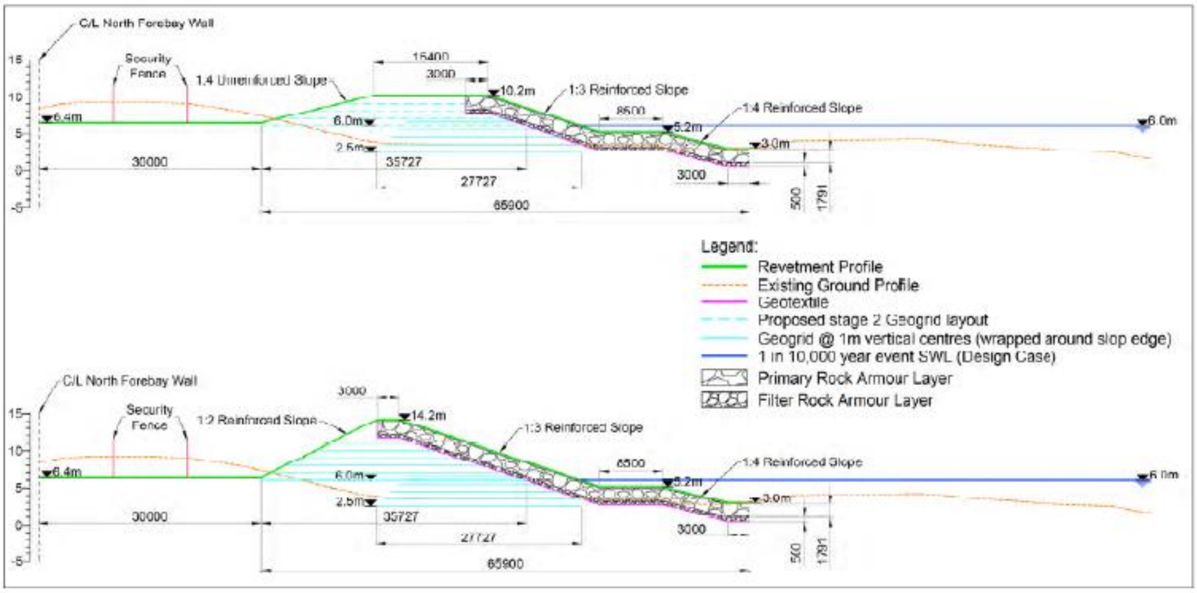

Figure 11: Concept Design Option 2.1 – Landscaped

At the northern end of SZC, it will be necessary for the new defence to tie into the existing defence. Hence, there may need to be an additional cross section added to the suite ('S1a') with a slightly different form to the other locations. If necessary, this additional cross section will be added after initial modelling has commenced. The operational phase modelling runs can therefore be summarised as follows:

- The main site sea defence option to be assessed is Option 2.1 as defined in the Jacobs report (Jacobs, 2014b).
- A number of variants to the profile shape need to be considered:
  - 10.0m AOD crest level, rock revetment;
  - 10.2m AOD crest level, landscaped (impermeable);
  - 14.0m AOD crest level, rock revetment; and
  - 14.2m AOD crest level, landscaped (impermeable).
- The modelling will assume that the topography between the main defence and the 5m berm will be infilled to 4m AOD (it is possible that further modelling using a 3m AOD topography behind the berm will be undertaken as a sensitivity test).
- An additional profile may be added if necessary at the northern end of SZC to show the tie into the existing defence.

It should be noted that the above combinations of options of crest level and finish could potentially result in a significant number of model runs, particularly when combined with two return periods (1 in 1,000 and 1 in 200) and different climate change scenarios.

A complete set of combinations will not be run if this is not necessary; for example it may be possible to confirm that there is no need to model the 14.0m AOD crest level if the volume of overtopping at 10.0m AOD is limited on a given event. Likewise, if overtopping does not occur on the 1 in 1,000 event for any particular scenario then the 1 in 200 event will not be run.

7 Assessing impact of breach of main site sea defence

The FRA will consider the potential impacts of a breach of the main site sea defence. As this defence will be designed to satisfy the requirements of the nuclear Safety Case, the mechanism by which it could be breached will not be considered by the FRA. Instead, the FRA will only examine the potential water levels on-site.

In order to model a breach in the main site sea defence, it is necessary to consider what profile the breached defence would take. In practice, the defence material that is dislodged when the breach is created will be redistributed to other locations rather than lost from the system entirely. However, there is no reasonable method for establishing how such a redistribution of defence material may occur, such as whether material may remain close to the breach, move inland or on to the beach, or whether it may propagate north or south along the coastline.

It is therefore proposed that, for FRA purposes, the profile modelled in AMAZON will assume that the breached material is lost from the system entirely. The breach level will be assumed to be the same level as the hinterland behind the main defence, which will be the same as the SZC platform level set at 7.3m AOD. This is considered to be a conservative approach, as in practice any material that is redistributed in the event of a breach is likely to continue to provide some flood defence benefit. For example, material redistributed in front of the defence would raise the beach level and thus reduce the height of any waves that reach the breach location.

AMAZON will be used to model the main site sea defence breach scenario. It is anticipated that a number of model runs may be required; modelling will start with the most extreme event (i.e. the highest sea-level rise, etc.) and the input data will be gradually reduced in severity until a point is reached whereby no wave overtopping occurs, or the magnitude of wave overtopping is reduced to a level that is considered insignificant. AMAZON will provide average and peak overtopping rates as litres per second per linear metre of defence. These rates can therefore be multiplied up by an assumed breach width (e.g. 100m) to give total predicted overtopping rates. The total overtopping rates will then be studied to determine

8 Overtopping Model Runs

Considering the six phases of SZC and the set of climate change scenarios that are being adopted (as described in Royal HaskoningDHV's Climate Change Note, Royal HaskoningDHV, 2015) and that each climate change scenario will also have to be combined with a 1 in 1,000 event and possibly a 1 in 200 event for each development phase, there is clearly the potential need to undertake a large number of overtopping model runs. However, the number of runs could be reduced if it is found that overtopping does not occur during the 1 in 1,000 event; in that instance the associated 1 in 200 event will not need to be run. In the following schedule of runs the 1 in 200 event runs are therefore optional based on the outcome of the 1 in 1,000 event runs.

Cefas have been running coastal models supported by the climate change data developed by Royal HaskoningDHV, for input into the overtopping models. A summary of the climate change scenarios (at 2110 and 2140) is shown in Table 12.

Table 12: Climate change scenarios defined in the Climate Change Note (Royal HaskoningDHV, 2015)

Year	Development Phase	Flood Risk Assessment					
		Medium Emissions 95%ile (m)	High Emissions 95%ile (m)	Upper End Estimate with Land Motion + surge (m)	BECC Lower (m)	H^{++} with Land Motion + surge (m)	BECC Upper (m)
2110	End of decommissioning	0.74	0.91	1.105 + 0.7 = 1.81	1.55	2.206 + 1.0 = 3.1	3.20
2140	Interim spent fuel store	1.01	1.24		1.95		3.92

8.1 2017: Start of Construction (Existing Situation)

The existing situation will be modelled twice using a profile at each of S1 to S5, for a 1 in 1,000 return period event with the two 2017 climate change parameters applied. Two further runs might be needed using a 1 in 200 event if the 1 in 1,000 runs show overtopping. In summary, for the existing situation, two runs will definitely be completed. Up to two optional runs might be needed based on the outcome of the first two runs. Table 13 summarises the runs for the existing situation.

Table 13: Overtopping model runs for the existing situation at 2017

Profile	Event	Climate Change Scenario	Status
S1 to S5	**1 in 1,000**	**2017 Medium emissions 95%**	**Definite**
		2017 High emissions 95%	**Definite**

	1 in 200	2017 Medium emissions 95%	Optional
		2017 High emissions 95%	Optional

8.2 2025: End of Construction (Commissioning)

A set of model runs are required that cover the two separate construction sequence options under consideration; Options 1A and 1B as described by Jacobs (Appendix B). As noted previously, all construction sequence runs will be carried out for the year 2025.

For Option 1A, two profiles associated with Activity (i)a and two profiles associated with Activity (iii)a will be modelled (Table 14). Each of these four profiles will be modelled using a 1 in 1,000 return period event with the two 2025 climate change parameters applied. For each of the Activities the same profile will be used at each of locations S1 to S5. Further (up to eight) optional runs might be needed using a 1 in 200 event if any of the first set of runs shows overtopping. Also, more optional runs (up to eight again) might be needed using the profiles of Activity (ii)a, if the modelling of Activity (i)a shows overtopping.

For Option 1B, two profiles associated with Activity (i)b and two profiles associated with Activity (ii)b will be modelled. Each of these four profiles will be modelled using a 1 in 1,000 return period event with the two 2025 climate change parameters applied, and again for each of the Activities the same profile will be used at each of locations S1 to S5. Further (up to eight) optional runs might be needed using a 1 in 200 event if any of the first set of runs shows overtopping.

In summary, for the construction phase, eight runs will be completed for each of Options 1A and 1B. A set of up to 24 optional runs might be needed based on the outcome of the first set of 16 runs. Table 14 summarises the runs for the construction phase.

Table 14: Overtopping model runs for the end of construction phase at 2025

Option	Activity	Profile	Event	Climate Change Scenario	Status
1A	(i)a	With 5m berm	**1 in 1,000**	**2025 Medium emissions 95%**	**Definite**
				2025 High emissions 95%	**Definite**
			1 in 200	2025 Medium emissions 95%	Optional
				2025 High emissions 95%	Optional
		Without 5m berm	**1 in 1,000**	**2025 Medium emissions 95%**	**Definite**
				2025 High emissions 95%	**Definite**
			1 in 200	2025 Medium emissions 95%	Optional
				2025 High emissions 95%	Optional

Option	Activity	Profile	Event	Climate Change Scenario	Status
	(ii)a	With 5m berm	1 in 1,000	2025 Medium emissions 95%	Optional
				2025 High emissions 95%	Optional
			1 in 200	2025 Medium emissions 95%	Optional
				2025 High emissions 95%	Optional
		Without 5m berm	1 in 1,000	2025 Medium emissions 95%	Optional
				2025 High emissions 95%	Optional
			1 in 200	2025 Medium emissions 95%	Optional
				2025 High emissions 95%	Optional
	(iii)a	With 5m berm	**1 in 1,000**	**2025 Medium emissions 95%**	**Definite**
				2025 High emissions 95%	**Definite**
			1 in 200	2025 Medium emissions 95%	Optional
				2025 High emissions 95%	Optional
		Without 5m berm	**1 in 1,000**	**2025 Medium emissions 95%**	**Definite**
				2025 High emissions 95%	**Definite**
			1 in 200	2025 Medium emissions 95%	Optional
				2025 High emissions 95%	Optional
1B	(i)b	With 5m berm	**1 in 1,000**	**2025 Medium emissions 95%**	**Definite**
				2025 High emissions 95%	**Definite**
			1 in 200	2025 Medium emissions 95%	Optional
				2025 High emissions 95%	Optional
		Without 5m berm	**1 in 1,000**	**2025 Medium emissions 95%**	**Definite**
				2025 High emissions 95%	**Definite**
			1 in 200	2025 Medium emissions 95%	Optional
				2025 High emissions 95%	Optional
	(ii)b	With 5m berm	**1 in 1,000**	**2025 Medium emissions 95%**	**Definite**
				2025 High emissions 95%	**Definite**
			1 in 200	2025 Medium emissions 95%	Optional
				2025 High emissions 95%	Optional
		Without 5m berm	**1 in 1,000**	**2025 Medium emissions 95%**	**Definite**
				2025 High emissions 95%	**Definite**

Option	Activity	Profile	Event	Climate Change Scenario	Status
			1 in 200	2025 Medium emissions 95%	Optional
				2025 High emissions 95%	Optional

8.3 2110: End of Decommissioning

A set of model runs will be completed to analyse overtopping risk at 2110, the end of decommissioning. **Model runs for the end of operation in 2085 are deemed optional at this stage and will only be carried out if the results of the 2110 runs show overtopping**. The end of decommissioning will be modelled using a set of profiles at S1 to S5 for each run (based on crest level and existing profile geometry between the main defence and the 5m sacrificial berm. Six runs are recommended at this stage (Table 15), with a large number of potential optional runs possible, depending on the outcomes of the initial six runs. Two of the six runs will be for Option 2.1 using a 10.0m AOD crest level rock revetment with the existing profile between the main defence and the 5m sacrificial berm (although the berm will be lost), the 1 in 1,000 return period event with two 2110 climate change parameters applied (Table 15). The other four runs will use the H^{++} scenarios applied to a sea defence with a 14.0m AOD crest level. H^{++} is used in these cases on the basis that if, during the lifetime of the development, it becomes apparent that sea-level rise is moving along that more extreme line (the sea-defences will be raised).

Table 15: Initial overtopping model runs for 2110 (end of decommissioning). Note that the worst case climate change sea-level rise scenario will be run first followed by subsequently less severe rises in sea level

Option	Profile	Event	2110 Climate Change Scenario
2.1	S1 – S5 10.0m AOD crest-level rock revetment Existing fronting profile	1 in 1,000	Medium emissions 95% (0.74m relative to 2008 baseline)
			High emissions 95% (0.91m relative to 2008 baseline)
	S1 – S5 14.0m AOD crest-level rock revetment Existing fronting profile	1 in 1,000	Upper End Estimate with Land Motion + surge (1.81m relative to 2008 baseline)
			BECC Lower (1.55m relative to 2008 baseline)
			H^{++} with Land Motion + surge (3.21m relative to 2008 baseline)
			BECC Upper (3.2m relative to 2008 baseline)

A large number of optional runs could be required if any of the initial six runs show overtopping including a variation on return period (using 1 in 200). These potential runs are too numerous to tabulate here and should be formulated as and when they are necessary, throughout the modelling process. In addition, optional runs might be needed incorporating long-term beach erosion.

8.4 2140: Interim Spent Fuel Store Decommissioned

Four initial runs will be completed for overtopping risk at 2140 when the interim spent fuel store is decommissioned (Table 16). Note that a large number of optional runs are also possible at 2140, in a similar way to 2110.

Table 16: Initial overtopping model runs for 2140 (interim spent fuel store decommissioned). Note that the worst case climate change sea-level rise scenario will be run first followed by subsequently less severe rises in sea level

Option	Profile	Event	2140 Climate Change Scenario
2.1	S1 – S5 10.0m AOD crest-level rock revetment Existing fronting profile	1 in 1,000	Medium emissions 95% (1.01m relative to 2008 baseline)
			High emissions 95% (1.24m relative to 2008 baseline)
	S1 – S5 14.0m AOD crest-level rock revetment Existing fronting profile	1 in 1,000	BECC Lower (1.95m relative to 2008 baseline)
			BECC Upper (3.92m relative to 2008 baseline)

9 Design Profile - Preliminary Overtopping Tests

Preliminary tests have been carried out for the proposed design profile at SZC Sea Defence. Overtopping analysis was performed for 1 in 10,000 year return period with three different 2110 Climate Change Scenarios, which are:

- Medium emissions 95% (0.74m relative to 2008 baseline);
- Medium emissions 95% (1.01m relative to 2008 baseline); and
- H^{++} with Land Motion + surge (3.21m relative to 2008 baseline).

Three profiles were selected for the preliminary tests, S2, S3 and S4 extracted from LiDAR (2010) and ground level surveys (2013-2014). Figure 12 shows the defence outline and beach profiles at all three locations.

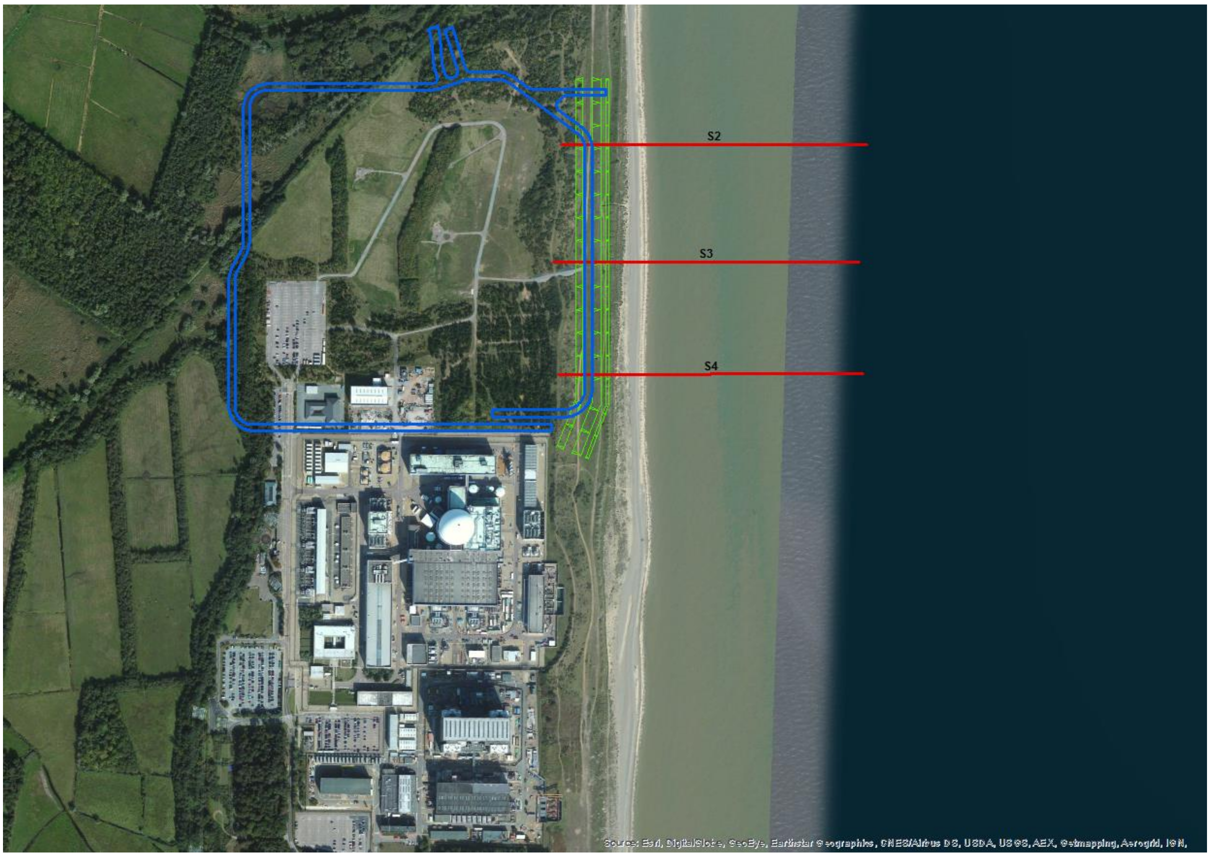

Figure 12 Locations of profiles S2, S3 and S4 for the preliminary overtopping tests

The profile of the proposed hard defence with 10.0m AOD crest level was extracted from the design drawing provided by the Client (drawing no. SZC-NNBPCP-XX-000-DRW-100090). This profile was modified to increase the crest to 14.0m AOD utilizing the width of the plateau and slopes as instructed by the Client in the email from 24th January 2017. Figure shows the design profiles.

Figure 13 to Figure 15 show the proposed defence design combined with the existing profiles S2, S3 and S4, respectively. It should be noted that the crest levels of the defences used for overtopping modelling include 0.2 m allowance for landscaping.

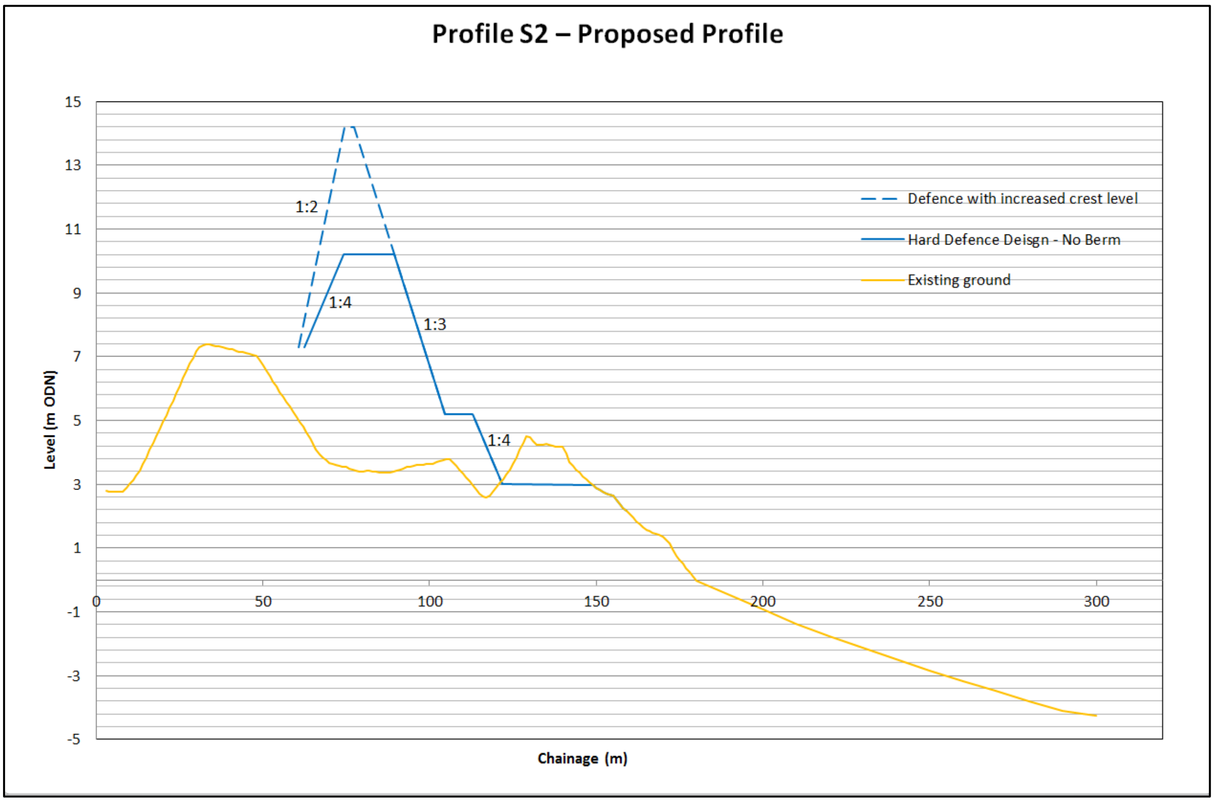

Figure 13 Combined defence/beach profiles extracted from LiDAR and ground level survey data at S2

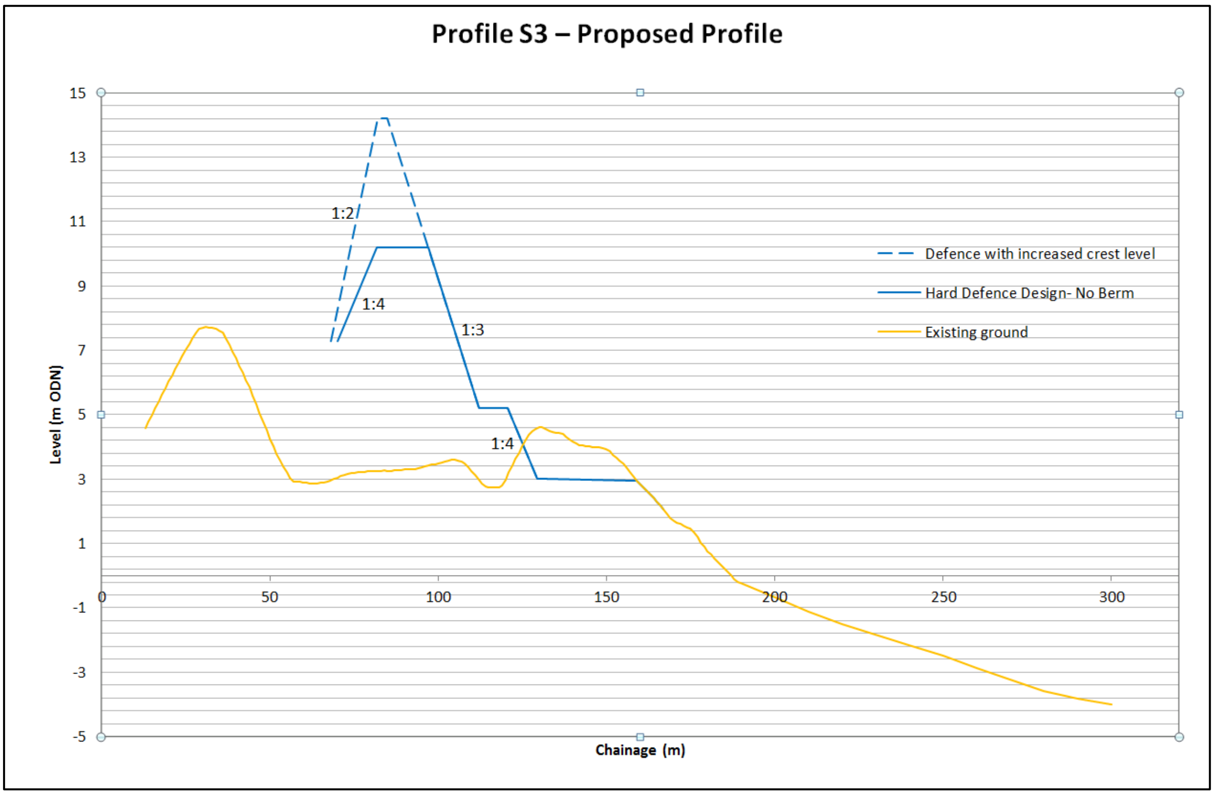

Figure 14 Combined defence/beach profiles extracted from LiDAR and ground level survey data at S3

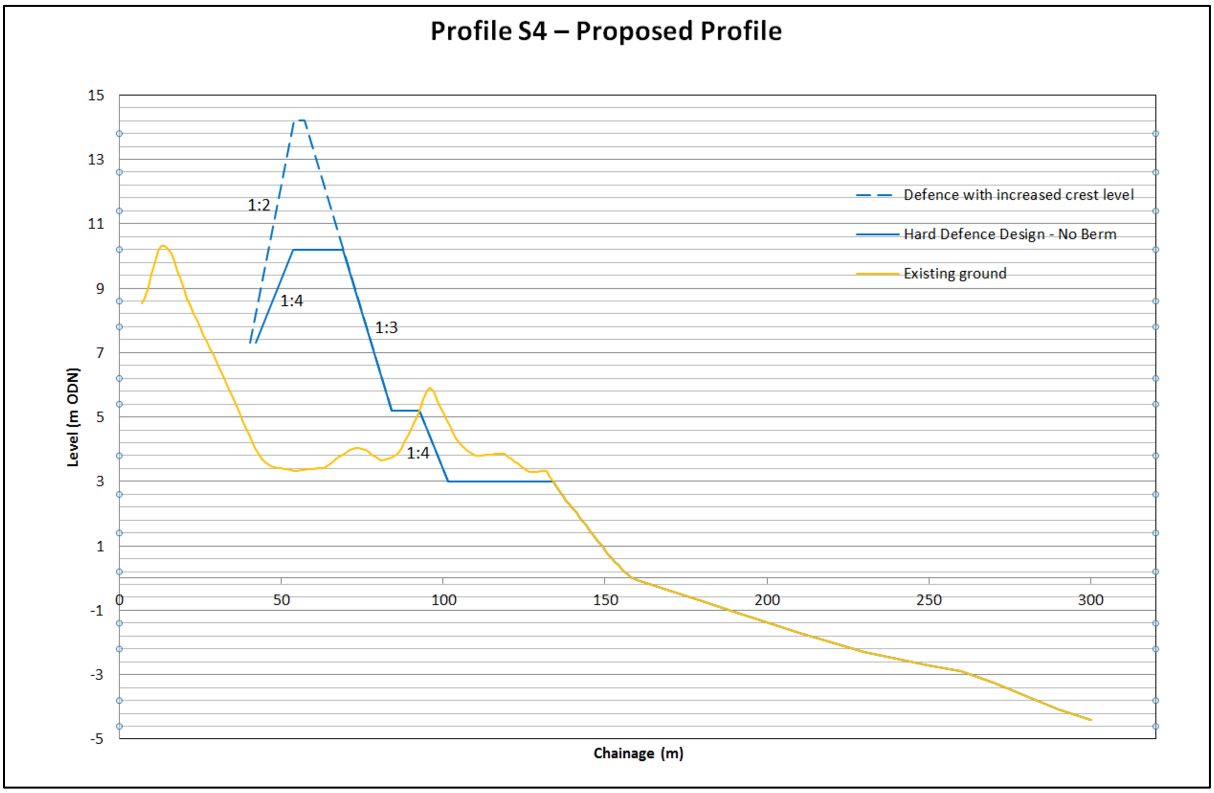

Figure 15 Combined defence/beach profiles extracted from LiDAR and ground level survey data at S4

A summary of Input conditions is presented in Table 17 to Table 19 for the three selected climate change scenarios respectively.

Table 17 Input data for 10.2m AOD crest defence sensitivity (for 1 in 10,000 year return period and the Medium emissions 95% (0.74m relative to 2008 baseline 2110) climate change scenario

Cefas JP combination code	Profile	Wave height H_s (m)	Wave period, T_p (sec)	Water level (m AOD)
F1	S2	4.02	11.95	5.67
	S3	3.98		
	S4	3.93		

C1	S2	3.12	10.11	5.94
	S3	3.11		
	S4	3.17		

Table 18 Input data for 10.2m AOD crest defence sensitivity (for 1 in 10,000 year return period and Medium emissions 95% (1.01m relative to 2008 baseline) climate change scenario

Cefas JP combination code	Profile	Wave height H_s (m)	Wave period, T_p (sec)	Water level (m AOD)
F1	S2	4.14	11.95	5.94
	S3	4.10		
	S4	4.06		
C1	S2	3.01	10.11	6.21
	S3	2.99		
	S4	3.06		

Table 19 Input data for 14.2m AOD crest defence sensitivity (for 1 in 10,000 year return period and H++ with Land Motion + surge (3.21m relative to 2008 baseline) climate change scenario

Cefas JP combination code	Profile	Wave height H_s (m)	Wave period, T_p (sec)	Water level (m AOD)
F1	S2	4.85	12.20	8.13
	S3	4.83		
	S4	4.81		
C1	S2	3.23	10.32	8.40
	S3	3.23		
	S4	3.28		

It should be noted that the preliminary tests were carried out for the Medium Emissions 95% Climate change scenarios with a defence crest level of 10.2m AOD and for H++ with Land Motion + surge Climate change scenario with a defence crest level of 14.2m AOD.

Table 20 presents the predicted mean overtopping rates at the top defence crest for each of the modelled scenario. The results show that the proposed defence crest is overtopped at all modelled climate change scenarios at varying mean overtopping rates. The highest overtopping rates are produced by Cefas joint probability combination F1 which is consistently for all profiles and climate change scenarios.

Table 20 Predicted mean overtopping rates for a 1 in 1,000 year return period event with 3 selected climate change scenarios

Cefas Climate Change Scenario / Defene Crest Level	Cefas JP combination code	Mean Overtopping Rates (l/s/m)		
		Profile S2	Profile S3	Profile S4
Medium emissions 95% (0.74m relative to 2008 baseline 2110) / Defence crest level at 10.2m AOD	F 1	1.90	2.29	1.40
	C 1	0.40	0.58	0.36
Medium emissions 95% (1.01m relative to 2008 baseline) / Defence crest level at 10.2m AOD	F 1	4.10	4.94	3.29
	C 1	0.76	1.07	0.70
H++ with Land Motion + surge (3.21m relative to 2008 baseline) / Defence crest level at 14.2m AOD	F 1	7.02	9.95	6.24
	C 1	0.93	1.56	1.01

10 Uncertainties in the wave overtopping modelling processes

Table 19 lists the key uncertainties discussed in the above sections.

Table 19: Key uncertainties in the proposed wave overtopping modelling processes

Key Uncertainties	Mitigation	Conservativeness
Inshore wave height	The uncertainty in modelled inshore wave heights is to be addressed by adding 10% wave height. The comparison between modelled and measured wave heights at a near-shore wave buoy by Cefas showed model errors below 10%	Reasonably conservative
Inshore wave period	Use offshore wave period and JONSWAP spectrum. This approach is to address two-peaked wave spectrum inshore	Reasonably conservative
Inshore wave angle	Assume shore-normal wave approach angle	The modelled inshore wave data show that most extreme waves are almost in shore-normal angles. Slightly conservative
Beach profile	Beach profiles were taken from the lowest levels from the last 5 years' survey data	Slightly conservative to neutral
Sacrificial berm	Sacrificial berm is to be removed in overtopping models	Sensitivity tests described in Section 5 show that "no berm" gave the worst overtopping. Reasonably conservative
Shingle layer	Shingle layer is not to be considered	Sensitivity tests described in Section 5 show that a shingle layer has very limited effect in overtopping for extreme events. Slightly conservative
Offshore sandbanks	Use Baseline geo-scenario	Cefas' study shows that the Baseline geo-scenario produced slightly higher inshore wave. Slightly conservative.

11 References

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Hu, K. (2000), High-resolution Finite Volume Methods for Hydraulic Flow Modelling, PhD thesis, Centre for Mathematical Modelling and Flow Analysis, the Manchester Metropolitan University, June, England.

Hu, K and Meyer, D. (2005), The Validity of the Non-linear Shallow Water Equations for Modelling Wave Runup and Reflection, International Conference on Coastlines, Structures and Breakwaters 2005, London.

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Jacobs (2014b), Sizewell C Sea Protection Phase 2 TN – Overtopping Rev A, Jacobs UK Ltd dated 9 October 2014

Reis, MT, Hu, K, Hedges, T S and Mase, H. (2005), A Comparison Of Empirical, Semi-Empirical And Numerical Wave Overtopping Models, Proceeding of the 5th International Symposium on Ocean Wave Measurement and Analysis, Madrid, Spain 3rd-7th July.

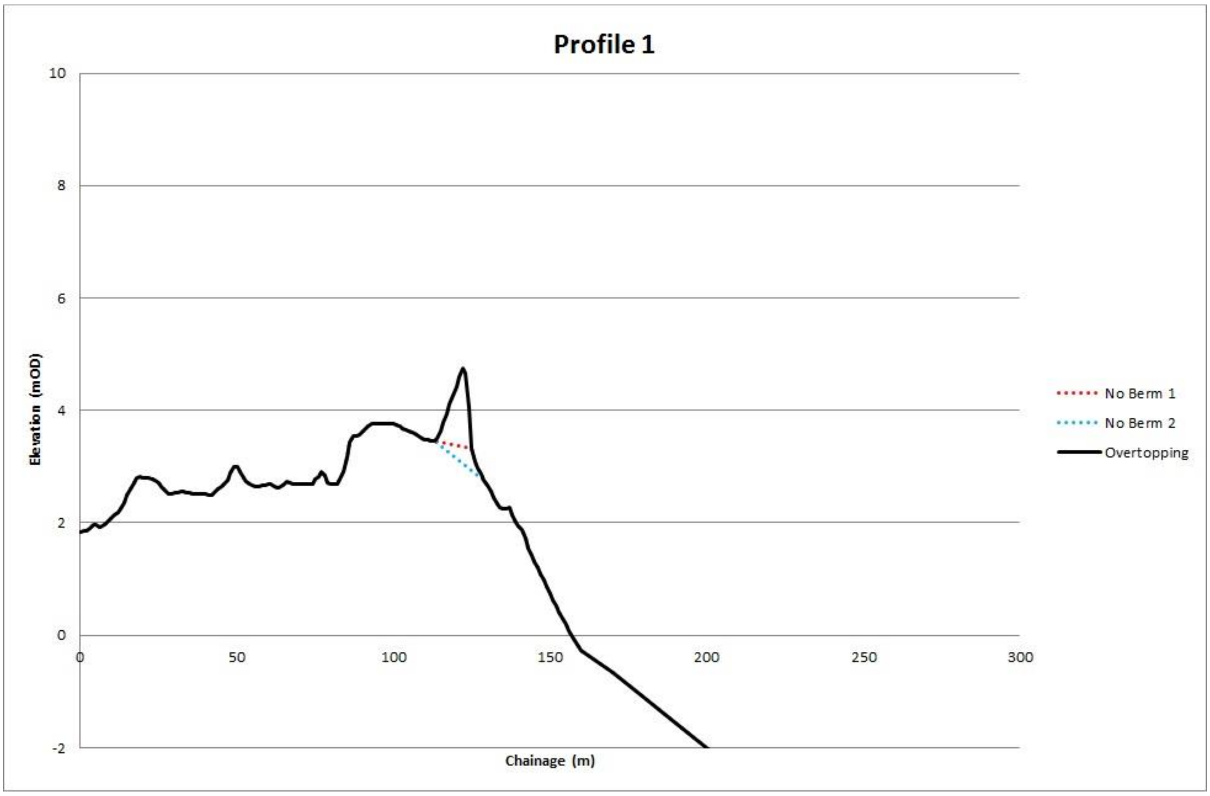
Reis, MT, Hu K, Neves MG and Hedges TS (2009), Numerical modelling of breakwater overtopping using a NLSW equation model with a porous layer. The 31st International Conference on Coastal Engineering (ICCE), Hamburg, Germany.

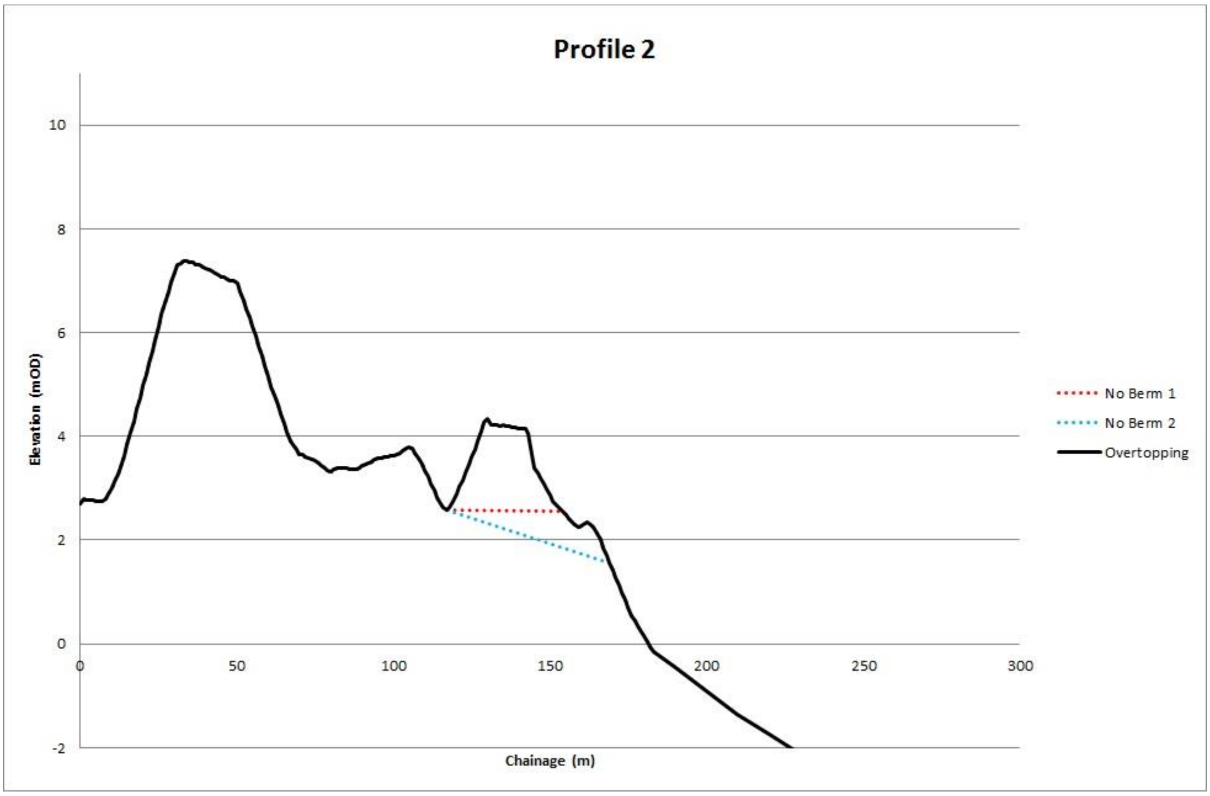
Reis MT, Neves MG, Lopes MR, Hu K and Silva LG (2011), Rehabilitation of Sines West Breakwater: Wave Overtopping Study, Proceedings of the Institute of Civil Engineers: Maritime Engineering, Volume 164.

Royal HaskoningDHV (2014), FRA Scoping Report: Sizewell C Nuclear New Build.

Royal HaskoningDHV (2015), Sizewell C Flood Risk Assessment: Recommended Climate Change Allowances (coastal) V5.1, September 2015.

Appendix A: The five profile shapes to be used to represent the existing situation at Profiles S1 to S5.




Profile 2
10
8
6
4
2
0
-2
Elevation (mOD)
0
50
100
150
200
250
300
Chainage (m)
No Berm 1
No Berm 2
Overtopping

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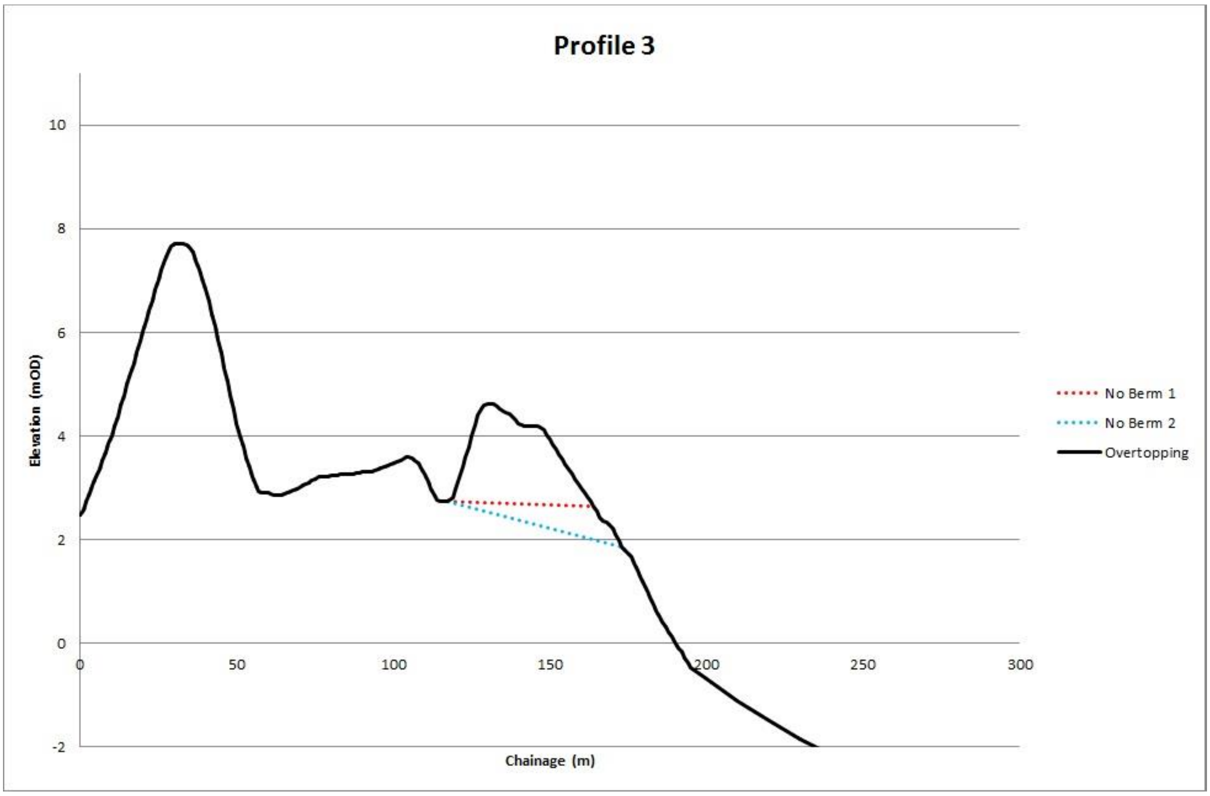
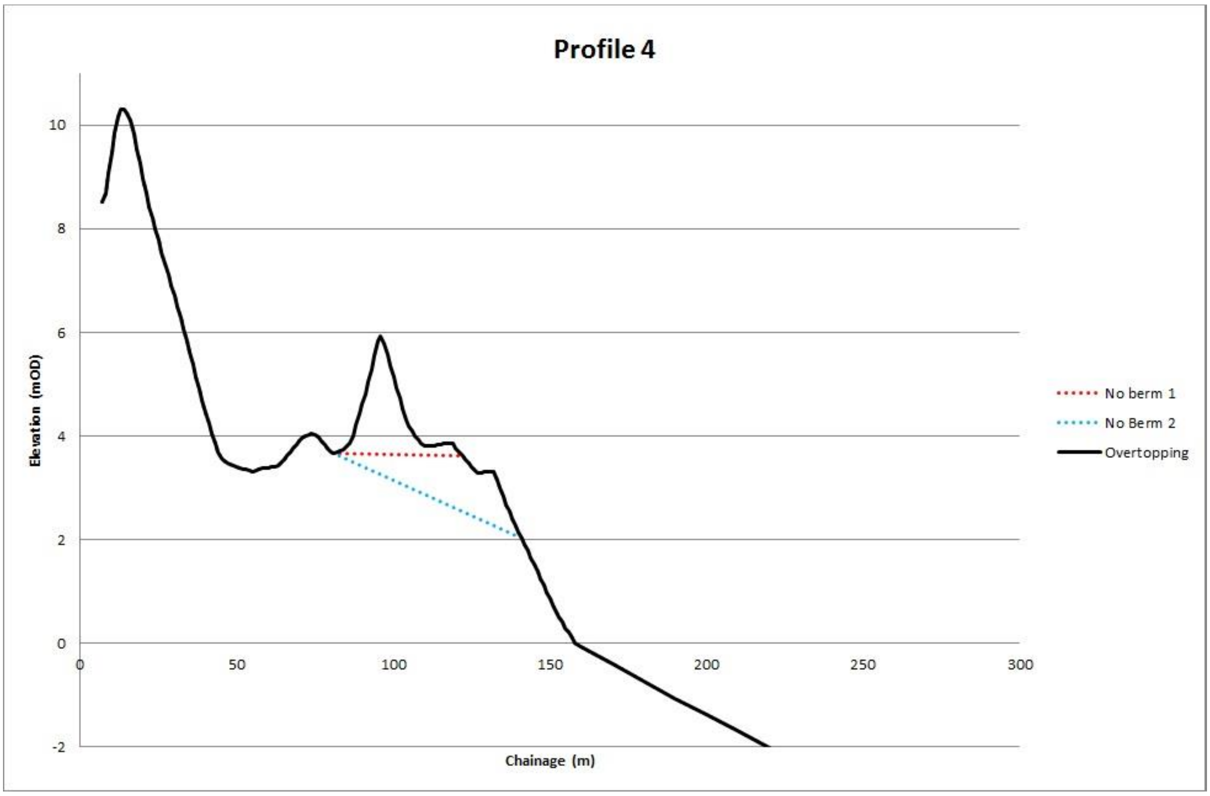
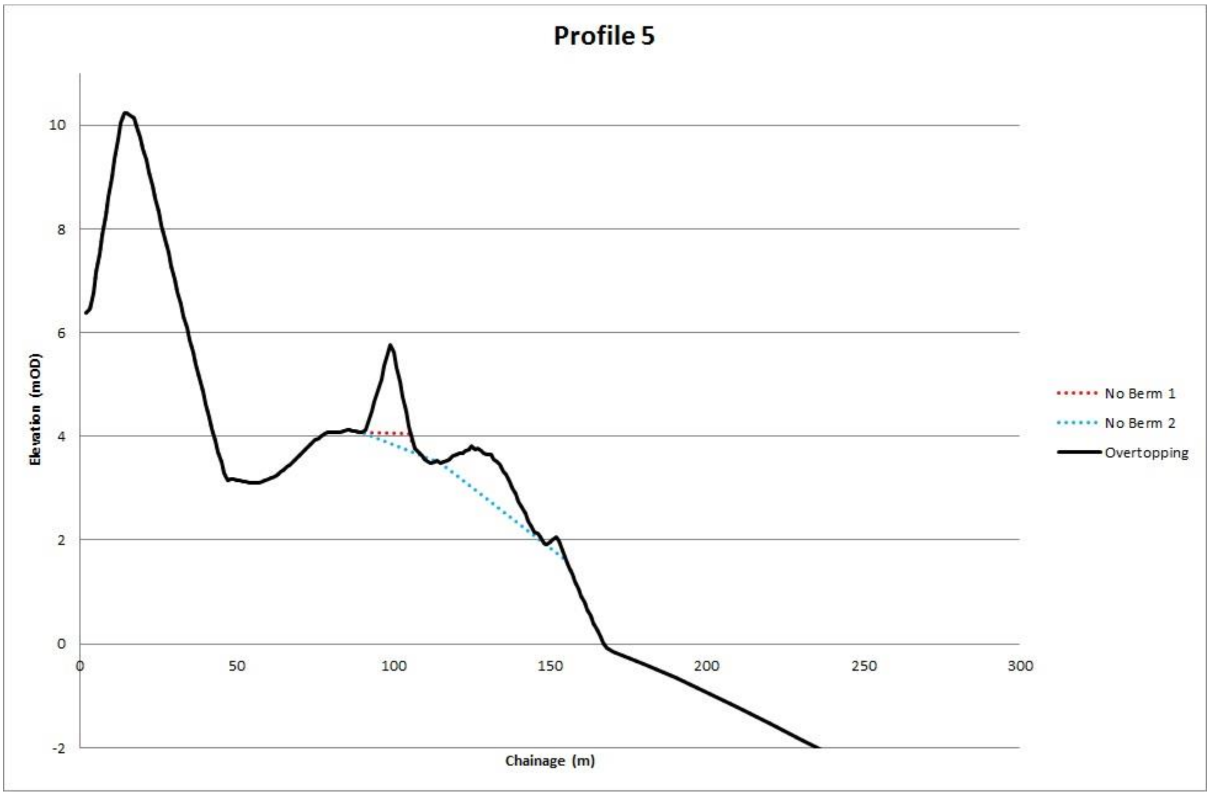



Profile 4
Elevation (mOD)
10
8
6
4
2
0
-2
0
50
100
150
200
250
300
Chainage (m)
No berm 1
No Berm 2
Overtopping


Profile 5
Elevation (mOD)
Chainage (m)
10
8
6
4
2
0
-2
0
50
100
150
200
250
300
No Berm 1
No Berm 2
Overtopping

Appendix B: Description of construction phase Options 1A and 1B.

Construction Sequence Option 1A

For Construction Sequence Option 1A, the following profiles will be modelled:

- Activity (i)a
  - Activity (i)a will be modelled. The purpose of this initial run will be to understand risk to construction workers and plant in the area between the 5m sacrificial berm (noted as 'existing sand bank' on the figure) and the existing bund that is being removed to stockpile.
  - A separate run will be carried out with the 5m sacrificial berm removed, for consistency with the 'existing situation' modelling.
- Activity (ii)a
  - Activity (ii)a will only be modelled if the results from modelling Activity (i)a show significant overtopping and risk. If the risk is not considered significant, Activity (ii)a will not be modelled.
  - As with Activity (ii)a, a separate run with the 5m sacrificial berm removed may be required.
- Activity (iii)a
  - Activity (iii)a will be modelled to determine whether there is any overtopping of the 7m bund, and also whether there is any risk actually on top of the 7m bund, as this area may be used for laying down and storing construction materials.
  - Again, this assessment will be carried out for both the profile as shown on the figure, and with the 5m sacrificial berm removed.
- Activity (iv)a – this step in the construction sequence will not be modelled as the addition of a temporary frontal extension will not alter the rate of overtopping compared to Activity (iii)a.
- Activity (v) – this step will not be modelled as the profile shape is the same as that for Activity (iii)a.
- Activity (vi) – this step will not be modelled as the addition of the rock armour stockpile will not increase overtopping compared to the previous steps; if anything it may reduce overtopping through dissipation of some of the wave energy.
- Activity (vii) – this step will not be modelled as the increase in the level of the main defence from 7m AOD to 10m AOD will only serve to reduce overtopping risk to the main site.

- Activity (viii) – this step will not be modelled as the addition of the rock armour will further reduce the risk to the main site. Further, at this stage, the profile effectively becomes the final sea defence profile, and therefore the actual sea defence profile will be modelled (as below), rather than the simplified profile that is shown in the construction sequencing report.

Construction Sequence Option 1B

For Construction Sequence Option 1B, the following profiles will be modelled:

- Activity (i)b
    - Activity (i)b will be modelled, as depicted in the figure. The purpose of this initial run will be to understand risk to construction workers and plant in the area between the 5m sacrificial berm (noted as existing sand bank on the figure) and the existing bund that is being removed to stockpile.
    - A separate run will be carried out with the 5m sacrificial berm removed, for consistency with the 'existing situation' modelling.
- Activity (ii)b
    - Activity (ii)b will be modelled, as depicted in the figure, to check for risk behind the 7m bund.
    - A separate run will be carried out with the 5m sacrificial berm removed, for consistency with the 'existing situation' modelling.
- Activity (iii)b
    - Activity (iii)b will not be modelled as the risk to plant at this stage will be negligible due to the location of the working area behind both 5m and 7m defences.
- Activity (iv)b
    - Activity (iv)b will not be modelled as the profile is the same as that at Activity (iii)b and similarly the risk to plant at this stage will be negligible due to the location of the working area behind both 5m and 7m defences.
- Activities (v) to (viii) are the same for Option 1B as they are for Option 1A; therefore no additional modelling will be undertaken for the same reasons as outlined above.

APPENDIX B:

‘Flood Risk Assessment Sizewell C: AMAZON for overtopping prediction’ Technical Note

RHDHV, March 2014

Technical Note

HASKONINGDHV UK LIMITED
RIVERS, DELTAS & COASTS

To	:	[redacted] (EDF)
From	:	Royal HaskoningDHV
Date	:	17 March 2014
Copy	:	
Our reference	:	PB1452

Subject : **Flood Risk Assessment Sizewell C: AMAZON for overtopping prediction**

---

1. Introduction

The "FRA Scoping Report: Sizewell C Nuclear New Build" report recommended a review of alternative methods for estimating wave overtopping, including EurOtop methods (including empirical and neural network methods) and AMAZON modelling. This Technical Note provides more information about the AMAZON model, discussing its strengths and weaknesses, and compares it with EurOtop methods on its suitability for coastal flood risk assessment at Sizewell.

2. About AMAZON

AMAZON is a high-resolution two-dimensional finite volume numerical model capable of simulating supercritical flow and capturing moving hydraulic jumps. It is based on solving the nonlinear shallow water equations (Hu et al., 1998; Hu, 2000). AMAZON was designed for "violent" (referring to moving flow discontinuity) flow such as hydraulic jumps, tsunamis, bore waves (including tidal bores) and dam breaks. Because of these strengths, AMAZON has been used for simulating wave runup and overtopping.

AMAZON-Wavewatch is one-dimensional software with a graphic user-interface (GUI) specifically designed for simulating wave overtopping of coastal structures based on the AMAZON model. AMAZON can also be used for two-dimensional wave overtopping simulations but generating a curvilinear mesh is a manual process with its own graphic user-interface.

AMAZON has been tested for wave overtopping calculations on single slope walls, slope walls with berms, and vertical seawalls (Hu et al., 1998). The computed results compared reasonably well with those derived from laboratory tests. In the tested 'standard' profiles, AMAZON performed as well as some empirical methods. A series of validity tests on AMAZON were undertaken by Hu and Meyer (2005) and Reis et al. (2005) compared with both experimental data and other overtopping models, which provided confidence in AMAZON as well as guidance for practical engineering applications.

In 2009, AMAZON was further developed to include a porous layer for rock armour or shingle (Reis et al., 2009) and a series of validity tests on the porous-layer version were subsequently undertaken by Reis et al. (2011). The paper by Reis et al. (2011) received a prestigious Institute of Civil Engineering Award in 2012. AMAZON's capability in considering the effect of a porous layer makes it unique comparing to other overtopping tools.

AMAZON requires input of bathymetry (or cross-sectional profile for a 1-D calculation) and incident waves. AMAZON is an unsteady model so that random waves can be simulated as well as monochromatic waves. Popular wave spectra including Bretshneider-Moskowitz, JONSWAP and TMA have been built into the AMAZON software. User-defined wave spectrum and measured wave trains are also accepted by AMAZON for incident wave input.

3. Weaknesses and Strengths of AMAZON

For wave overtopping, AMAZON has two limitations inherited from the non-linear shallow water equations, namely the shallow water assumption and wave breaking approximation. The shallow water assumption limits its use to relatively long waves. It requires water depths to be less than one tenth of wavelength (for example, for 10m deep water, it requires wave periods approximately 10 seconds or longer). Wave breaking in the nonlinear shallow water equations is approximated by steep fronts represented by bores. This approach, which ignores the detailed structure of the breaking wave, is often a reasonable approximation, especially in the swash zone.

One of the main weaknesses of the empirical overtopping method (any method that is derived from the results of laboratory tests, including neural network model) is that those formulations were derived from laboratory tests. Therefore, validation of those empirical formulae is limited by those tested conditions. For example, for a defence with an irregular profile (different to those tested), there is uncertainty in the empirical formulae. This weakness of the empirical method is in fact the strength of AMAZON. AMAZON considers the effect of every bed level change and calculates water movement governed by the very basic hydraulic principles of mass, momentum and energy conservation laws.

From a time series of simulated overtopping volumes, AMAZON can provide not only mean overtopping rates but also peak overtopping rates. Peak overtopping rate is particularly relevant to assessing potential risk of wave overtopping on people near sea defences. Because of its ability to capture peak overtopping, AMAZON can be used to estimate wave forces on structures. For example, it may be used to calculate wave force on a secondary wall. We are not aware of any empirical methods having such a capability.

The computing demand by AMAZON is tolerable. For a slope wall, it takes approximately 1-3 hours to simulate 1,000 waves on a reasonably powerful laptop, and 5-10 hours for a vertical wall for the same number of waves. It is not as computationally efficient as empirical formulae but it is much faster than the CFD model which could take days (or weeks) to simulate 1,000 waves.

4. Track Record of AMAZON

The technology of using non-linear shallow water equations (NLSWE) for simulating wave overtopping was initially introduced by van Gent (1994). The technology has been used for more than a decade. Strictly speaking, AMAZON is not the only NLSWE model for overtopping but AMAZON-Wavewatch is probably the only NLSWE overtopping modelling software designed for commercial use, and the most used NLSWE model in Europe. The development of AMAZON (including Keming Hu's PhD) was initiated by the lack of reliable modelling tools for "violent" flows. The fact that AMAZON was developed and driven by the industry made it quite unique. The development of AMAZON was helped substantially by inputs from coastal and maritime engineers, particularly on its validation and safety margin. AMAZON is now benefiting from collaboration with the Laboratório Nacional de Engenharia Civil (Portuguese national laboratory for civil engineering) where a more comprehensive guide on the validation conditions is being developed by comparing AMAZON results with those from physical models.

AMAZON has been used extensively by Royal HaskoningDHV for numerous clients worldwide, including the UK Environment Agency, (North East Coastal Tidal Flood Forecasting and Warning System (Northeast Region); Isle of Wight Coastal and Harbour Modelling (Southern Region); Paignton Coastal Flood Risk Assessment (Southwest Region), maritime councils (e.g. Havant, Portsmouth, and Scarborough) and port authorities (e.g. Port of Dover, Dublin Port and Peel Ports).

5. Comparing AMAZON with EurOtop for the Sizewell C Coastal Flood Risk Study

EurOtop methods are regarded as the industrial standard in the UK for predicting wave overtopping, particularly for "standard" defence profiles that have been well tested and incorporated in the EurOtop database. The main issue with EurOtop is that the defence profiles at Sizewell are not so "standard". Figure 1 illustrates five identified potential flood routes by overtopping and none of them have any standard profiles.

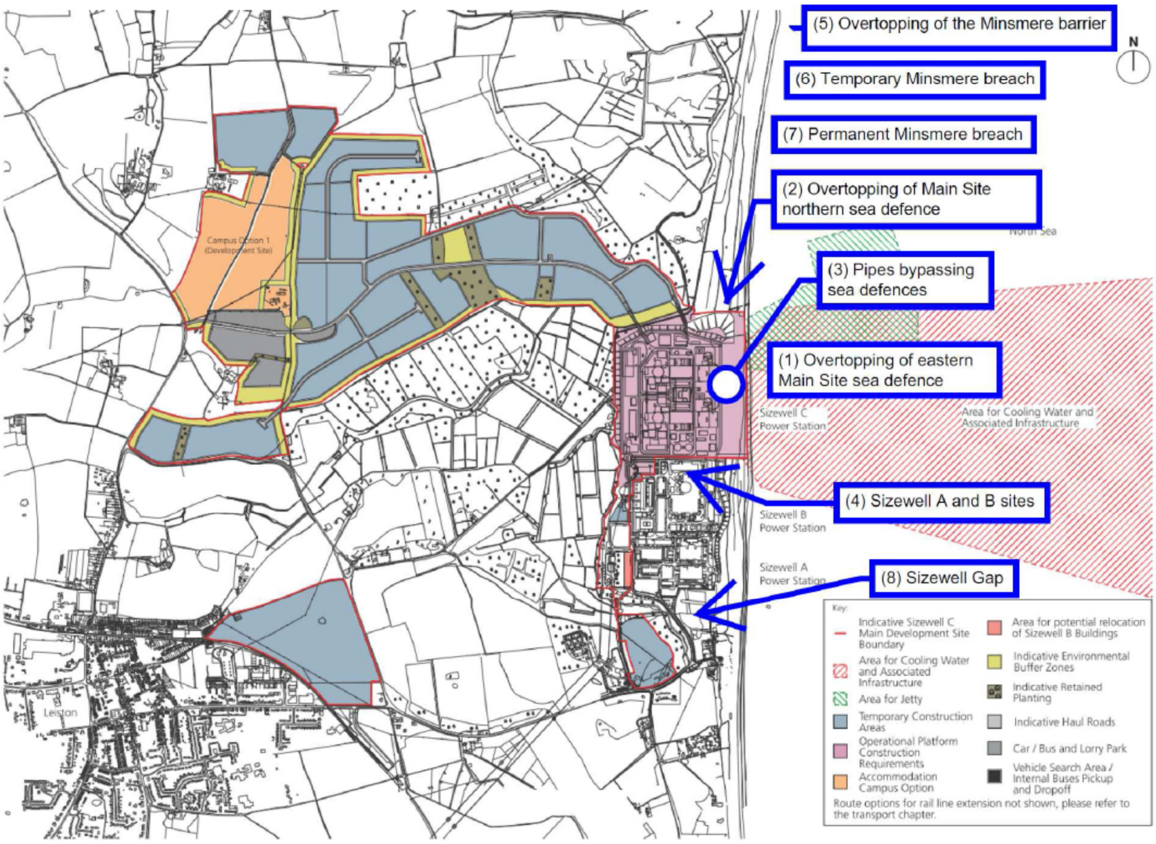


Figure 1: Potential coastal flood routes

Route 4: Overtopping of Sizewell A and B

Figure 2 shows the frontage along Sizewell A and B. The profile contains a shingle beach and vegetated sand dunes. Figure 3 shows a vertical wall in front of steps at Sizewell A.

Figure 2: At Sizewell A and B frontage (lower part)

Figure 3: At Sizewell A and B frontage (upper part)

Route 5: Overtopping of the Minsmere Barrier

Figure 4 shows an extensive shingle beach and sand dunes which separate Minsmere Nature Reserve from the North Sea.

Figure 4: Minsmere Barrier

Route 8: Overtopping of Sizewell Gap

Figure 5 shows Sizewell Gap where overtopping waves may result in sea water passing into the Sizewell Belts. Again, the potential flood route contains substantial shingle material and irregular sand dunes.

Figure 5: Sizewell Gap

Route 1: Overtopping of Sizewell C Eastern Sea Defence

We understand that the new sea defence is likely to have a main defence earth embankment with a reinforced earth landward slope. In front of the main defence, there will be a mixed shingle/sand beach with an approximately 5m bund designed to be sacrificial in the event of a storm (Figure 6).

Figure 6: Sea frontage along the proposed Sizewell C

Route 2: Overtopping of Sizewell C Northern Sea Defence

Figure 7 shows the northern frontage of the proposed Sizewell C, from where Sizewell C may be exposed to wave overtopping after potential retreat of the shoreline immediately to the north of the development.

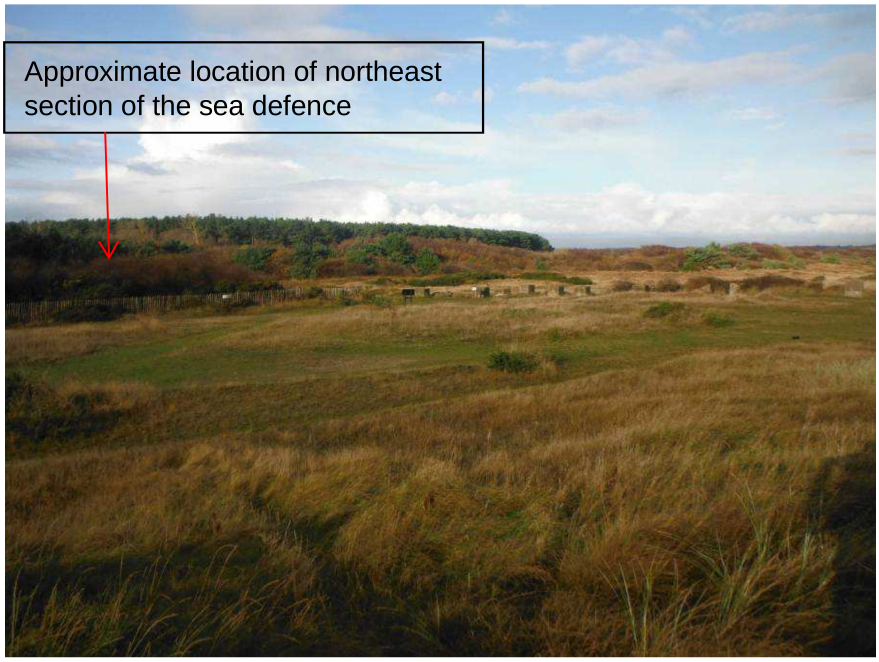


Figure 7: Northern frontage of the proposed Sizewell C

The photos demonstrate substantial shingle beach and vegetated sand dunes in all five potential flood routes involving wave overtopping. We believe those features are important to be considered in the wave overtopping analysis as they would have significant impact on wave propagation and runup before waves hit the hard sea defences. We cannot see how they can be properly represented by the EurOtop methods, and for this reason, we propose using AMAZON software for predicting wave overtopping for the Sizewell C coastal flood risk study.

We also believe that AMAZON is more appropriate for this study because the hard sea defences along Sizewell A and B have irregular profiles (such as the steps shown in Figure 3).

For the similar reasons, AMAZON has been used in the coastal flood risk studies for Dungeness and Heysham power stations, and the results have been compared with the EurOtop methods. From these two projects, we have confidence in AMAZON. We believe it is a more suitable tool and it has been substantially tested for engineering applications using a complex variety of topography, shingle beach and sand dune foreshores.

6. Recommendations

We believe that AMAZON is a suitable tool for predicting wave overtopping for coastal flood risk assessment at Sizewell at the 5 identified potential flood routes.

If the uncertainty of the AMAZON model is of concern, we recommend that AMAZON should be compared with physical models on similar cases (i.e. profiles having shingle beach, sand dune and hard defence). It is possible that such physical modelling has been completed in the past and the data may be available from laboratories (may be subject to a cost). We can send an enquiry to key laboratories in Europe on behalf of EDF Energy.

We believe there is limited value in applying both EurOtop and AMAZON at Sizewell. We have completed similar cross-checks at Dungeness and Heysham, but the difference between the results of two methods can neither validate nor invalidate each model without comparing to “true” results.

References:

van Gent (1994), The modelling of wave action on and in coastal structures, Coastal Engineering, Vol.22, pp311-339.

Hu, K., Mingham, C.G. and Causon, D.M. (1998), Numerical Simulation of Wave Overtopping of Coastal Structures by Solving NLSW Equations, Coastal Engineering, 41, 433-465.

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APPENDIX 2 - FLUVIAL MODELLING REPORT

CONTENTS

TABLES

PLATES

1 INTRODUCTION

1.1.1. This report provides an update on the schematisation of the hydraulic model of River Minsmere and Leiston Drain previously presented in August 2015 (**Appendix A**) and following model calibration completed in February 2017 (**Appendix B**). In addition, the model has been reviewed by the Environment Agency and amended in line with comments provided on 20th June 2019.

1.1.2. The report also summarises results from model runs conducted to date to assess fluvial flood risk to the proposed Sizewell C development and the potential impacts of the development to any off-site receptors.

1.1.3. This report is intended to update EDF Energy and the Environment Agency on the latest model developments, results and assumptions adopted in the detailed modelling study for the Flood Risk Assessment (FRA). A subset of model runs and results is also provided with this report to facilitate model review.

1.1.4. The outcomes presented in this report will inform the FRA as part of the Development Consent Order (DCO) application.

2 BACKGROUND

2.1 Model development

2.1.1. To inform the FRA for the proposed development at Sizewell C, a hydraulic model has been developed using industry standard hydraulic modelling software.

2.1.2. The initial 1D hydraulic model was constructed in ISIS (formerly ISIS, now Flood Modeller Pro) for the 'Flood study of River Minsmere and Leiston Drain' (Ref 1) and supplied by the Environment Agency.

2.1.3. Following internal model review and discussions with the Environment Agency, a series of changes to the model were required in order to provide sufficient level of confidence in the outputs from the hydraulic model.

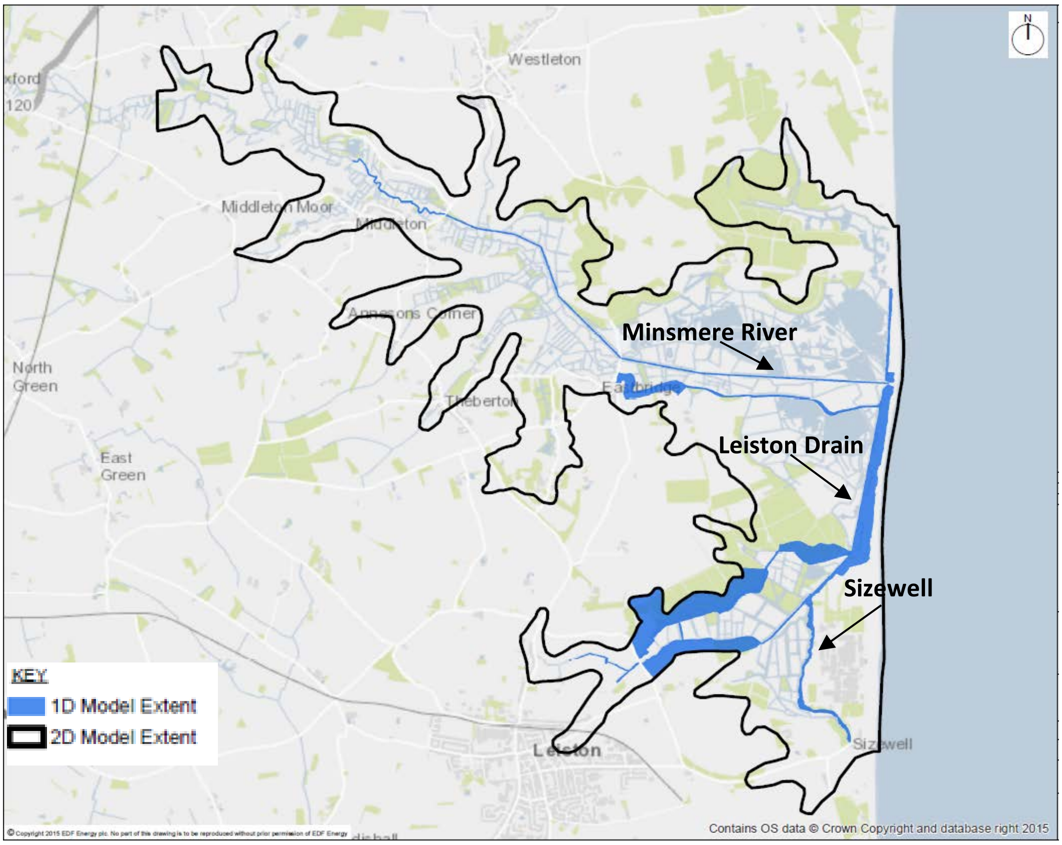
2.1.4. The key change to the initial 2013 model was to include a 2D model domain (TUFLOW) to replace most of the reservoir units, allowing a better representation of flow paths and attenuation in the low-lying floodplain. Additional surveys of the Sizewell Drain and Leiston Drain carried out in November and December 2013 were also included in the updated model. There were no further surveys carried out post initial model build in 2015.

2.1.5. Additional changes have been made to the schematisation of the low-lying reaches of the floodplain, where a direct rainfall method was applied instead of point inflows. The updated 1D-2D schematisation was then used for model calibration, validation and Sizewell C flood risk assessment.

2.1.6. **Plate 2.1** presents extents of the adopted 1D and 2D model domains.

2.1.7. Further information on the adopted approach and detailed methodology of model schematisation and updates are provided in Appendix A: 'Minsmere River and Sizewell Belts Model Schematisation Update' Report, issued to EDF on 28th August 2015.

Plate 2.1: 1D and 2D hydraulic model extents



2.2 Model calibration

2.2.1. The revised 1D-2D model has been calibrated as much as possible based on available gauge data and verified using observations of flood mechanisms and flood trash marks, although both calibration and verification were limited due to data availability.

2.2.2. Model calibration was based on three separate events; January 2003, March 2010 and January 2016. In 2012 work was undertaken to update and repair Minsmere Sluice structure, and therefore model schematisation was updated to reflect this in the model run for the 2016 event.

2.2.3. As the January 2016 event has more detailed gauging data available, especially for the Leiston catchment (new temporary loggers for which data was not available for the 2003 and 2010 events), the 2016 event was chosen for model calibration, with the other events used for model validation.

2.2.4. As a part of model calibration, sensitivity analysis was carried out on key model parameters; channel roughness, initial water level and percentage runoff and the opening of the Minsmere Sluice southern culvert. The derivation of inflows for the calibration events is described in the report 'Hydrology Review and Design Event Methodology' (Ref 2).

2.2.5. The overall conclusion was that calibration could only be executed to a limited extent because of limited data availability (temporal and spatial) and insufficient data quality.

2.2.6. The model shows good correlation with the available data and visual observations and is therefore considered sufficiently representative for the Minsmere and Leiston river systems, especially on the rising limb and first peak of the flood hydrograph.

2.2.7. The good correlation allows the model to be used for testing relative impacts of the proposed Sizewell C development, using single-peaked design events in line with current industry practice.

2.2.8. For peak flood conditions during the calibration events, the model shows an overestimation of the water level in the Leiston system by approximately 200mm. Over-estimation of flood levels is larger in the Minsmere system, which can be up to 400mm, particularly for subsequent peaks due to gradual filling/emptying of floodplain storage.

2.2.9. For gauge G5 near Leiston town, the flood levels are slightly under-estimated (100 mm).

2.2.10. Further details on model calibration are available in 'Sizewell-C Fluvial Modelling Calibration' report, issued to EDF on 23rd February 2017, provided in **Appendix B**.

2.3 Model requirements

2.3.1. The developed hydraulic model was used to provide an understanding of flood risk to the development site itself, as well as potential changes in flood risk to off-site receptors. Current and future flood risk was assessed at different phases of the development's lifespan.

2.3.2. The key points in time for the Sizewell C development are:

- 2034 – End of Construction / Start of Operation, however, 2030 (within the same climate change epoch for fluvial impact) used for assessment of construction phase flood risk;
- 2140 – Interim Spent Fuel Store Decommissioned, used for assessment through operation phase; and
- 2190 – Theoretical Maximum Site Lifetime used for assessment of impacts at end of the site lifetime.

2.3.3. To inform the FRA, a range of return periods was be run in the hydraulic model (1 in 5, 20, 100 and 1,000 years), with and without an allowance for climate change, for both the baseline and 'with scheme' model schematisations.

2.3.4. Further assessments for very low probability extreme fluvial and pluvial events was carried out as a part of the Safety Case study such as the 1 in 10,000-year and the 1 in 100,000-year return period.

3 SCHEMATISATION UPDATE

3.1 Improved model stability

3.1.1. Following the model calibration (2017), the latest model schematisation was adopted as the baseline model for further refinements and assessments through 2018/2019. Additional minor model updates were carried out to improve model stability for larger design events, including:

- Removed 3 cross-sections at the downstream boundary (Offshore1, 2 and 3) – these cross-sections were downstream of the Minsmere Sluice and were considered unnecessary for the purpose of this model;
- Removed a number of 2-node junctions that were not necessary, improved panel markers, dropped bed elevation in LEIS01_5062 at the top of the reach (slope was considered very steep during low flows, but will not influence flood extents), no changes to 2D domain;
- Removed 3 low head-loss bridges and replaced them with generalised loss units (at model nodes DRA71_1262, DRA71_1239 and DRA71_0678) – improved model stability with preserved appropriate conveyance, no changes to 2D domain;
- HX connections in the 2d_bc_hxe TUFLOW layer changed to SX connections at three locations to improve flow between extended cross-sections in the 1D model and floodplain in the 2D model domain (these locations are near cross-sections LEIS02_1209, LEIS01_2552 and LEIS01_1646);

- Removed two cross-sections, two junctions, two orifice units and spill over the orifices at connection channel between Leiston Drain Reach 1 and Reach 3 (model nodes from LEIS01_3764 to LEIS01_3764d). Distance between the upstream cross-section and the removed nodes was only 1m and hydraulics in this reach are dominated by the upstream spill unit (LEIS01_3765). Weir coefficient at this unit was lowered to 1.5 and modular limit to 0.7;
- In the current model, the Inverted syphon connecting the approach culvert from the Scott's Hall Ditch into the south chamber was represented using an orifice unit (label SCOT_0000). The orifice unit was replaced with the Inverted syphon and the dimensions of the Inverted syphon were adopted from *109417 -00.4 D.0* drawing;
- Initial conditions have been improved;
- The diameters of the culverts used to represent outfalls to the sea has been amended according to the as-built drawing.

3.1.2. In the current model schematisation, the penstock at the Minsmere South Culvert is assumed to be closed. That assumption is based on information provided by Environment Agency stating that 'this penstock is maintained in a closed position but can be operated in times of extreme flows.'

3.1.3. Since no further details have been provided on the operation of the penstock, it was not possible to determine the threshold levels for opening or closing. Therefore, the penstock has been closed. However, further testing has been carried out to assess sensitivity of the system to opening of the penstock (**section 6.5b**).

3.1.4. Flows from the Leiston Ditch, the New Cut River and the Scott's Hall Drain discharge into the chamber through orifice units/flap valves. The culverts linking each channel with its respective flap valves are currently not represented explicitly in the model, instead a coefficient within each unit was set to represent reasonable head losses through the structure.

3.1.5. The outlets from the New Cut Drain (MINS_0154oBu) into the north chamber and the Leiston Ditch (LEIS01_0000) are represented in the model as flap valves, whereas the as-built drawing suggest that they are vertical hinged gates. Although the time of closing of the gates might slightly differ to the time of closing the flapped valve in the model, this is considered not to have significant impact on the results. Further sensitivity testing could be carried out at the later stages of the project.

3.2 Aldhurst Farm

3.2.1. Aldhurst Farm Habitat Creation Scheme has been implemented to create lowland ditches and a mosaic of reedbed and open water habitat ('wetland habitat'). These wetland habitats would grade into a mosaic of acid grassland, heathland, scrub and deciduous woodland across the remainder Aldhurst Farm site.

3.2.2. The purpose of the scheme was to create habitats of similar quality and composition to those within the nearby Sizewell Marshes Site of Special Scientific Interest (SSSI) and support comparable invertebrate and rare vascular plant communities.

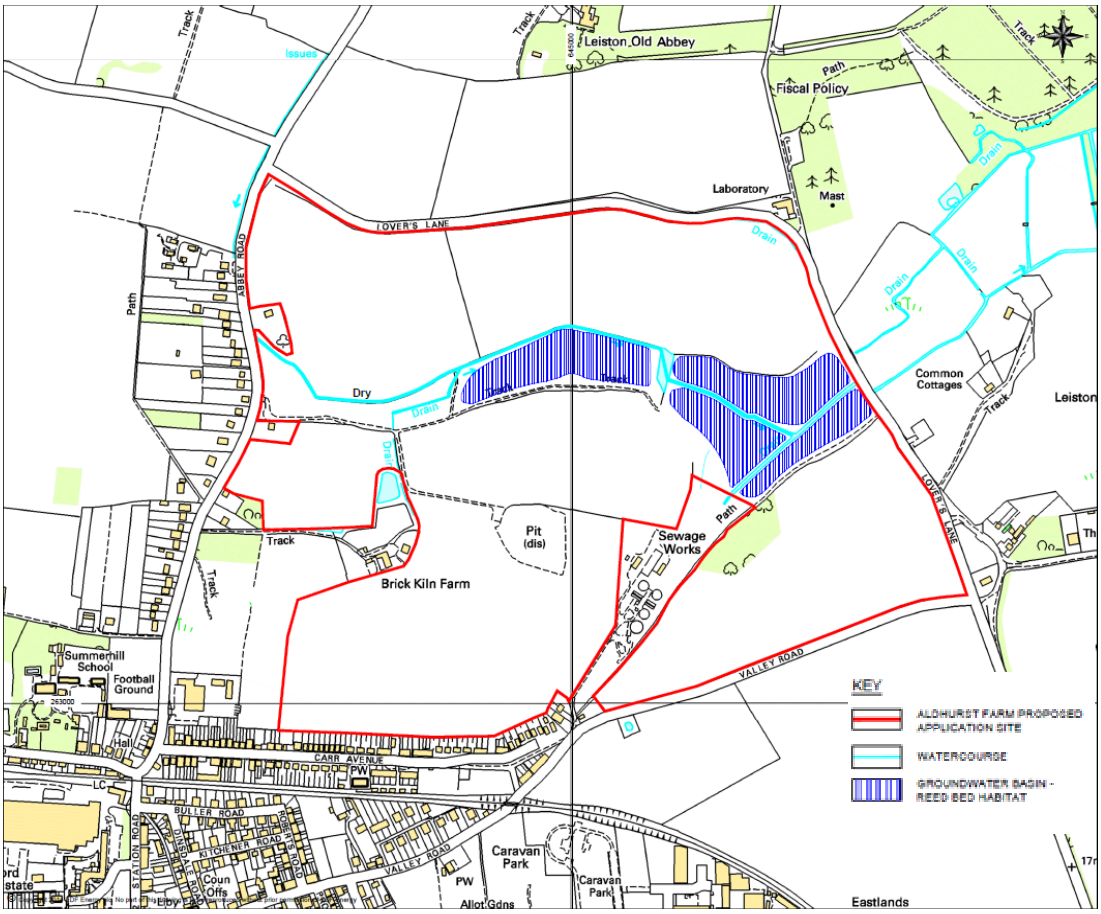
3.2.3. Excavated materials from the creation of the habitat within the wider site were reused within other areas of the site to help create the various terrestrial habitats.

3.2.4. The Aldhurst Farm now forms an extension of the Sizewell Estate, which is already managed by EDF Energy to deliver biodiversity and landscape benefits under a Higher-Level Stewardship Agreement.

3.2.5. The Aldhurst Farm site covers an area of 67hectares (ha) (**Plate 3.1**). The wetland habitat occupies approximately 6.3ha of low-lying land alongside two existing watercourses; the Aldhurst Valley Stream and a ditch receiving treated effluent from Leiston Waste Water Treatment Works.

3.2.6. Further details on the scheme and environmental screening are provided in the 'Aldhurst Farm Scheme EIA Screening Report' (Ref 3).

Plate 3.1: Extract from the Aldhurst Farm Habitat Creation Scheme EIA Screening Report Figure 3 – Wetland Habitat (Ref 3)



3.2.7. The Aldhurst Farm scheme was incorporated into the hydraulic model to assess the impact of the scheme on flood risk on-site and off-site to the Sizewell C development. The changes were incorporated to the latest baseline and 'with scheme' model schematisations.

3.2.8. The basins where included in the 2D-TUFLOW model domain by lowering the ground and assigning initial water levels as per a topographic survey of the scheme site (drawing ref. '5m grid topo survey.dwg') provided by EDF on 25th May 2018. The 'bed' levels in the basins were assigned as the water levels marked on the drawing. That was to represent the basins as being always full and therefore would offer limited storage capacity, giving slightly more conservative approach.

3.2.9. In addition, the survey data was used to define ground levels within the scheme by enforcing elevations in the DTM layer (**Plate 3.2)**. No change in roughness of the grounds at the Aldhurst Farm was assumed.

Plate 3.2: Extract from Aldhurst Farm – 5m grid topographic survey

3.2.10. Following initial baseline model runs with the Aldhurst Farm, some changes were made to improve connectivity of the basins and overall model stability. These are:

- Added connections between the basins at the Aldhurst Farm and the existing drains;
- Bottom slots at the conduits connecting Basin A and Basin C (Aldhurst Farm) turned on with max depth set to 1.0m to improve stability at initial timestep.

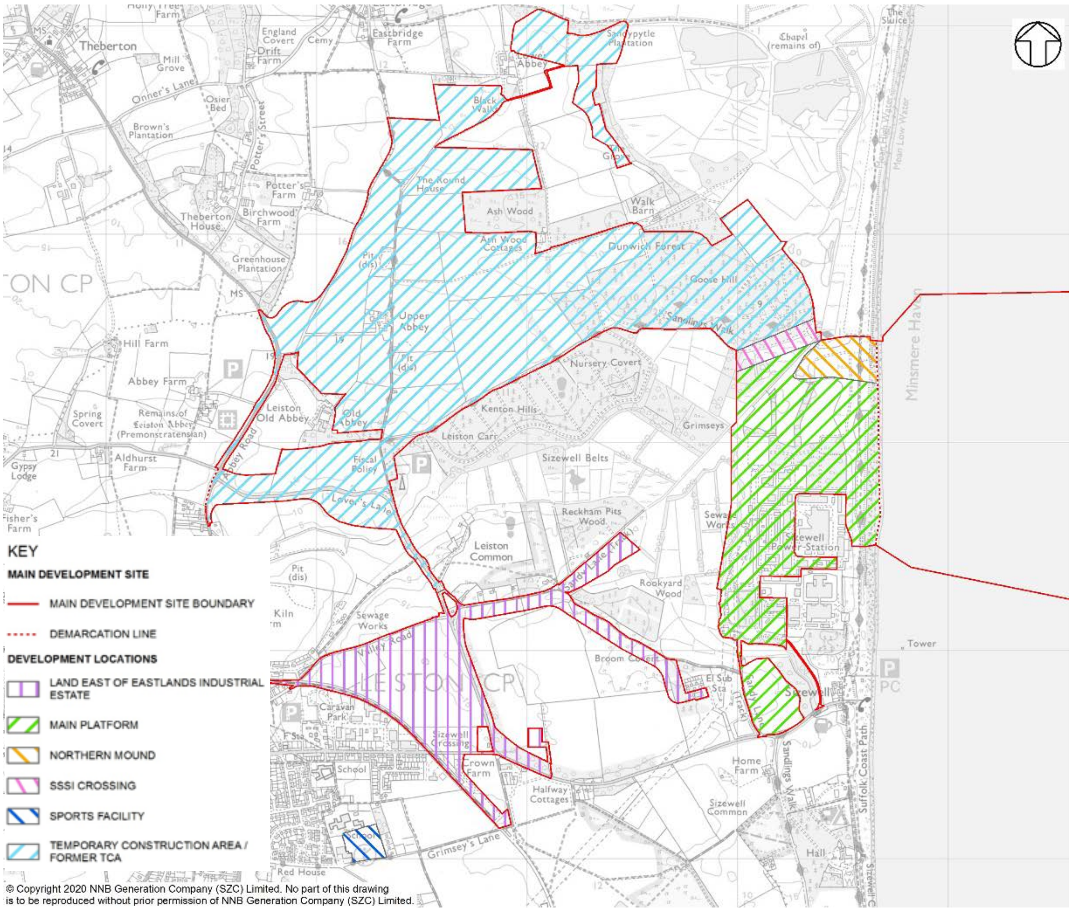
3.2.11. The drains and culverts connecting the basins to the Leiston Drain were incorporated into the 1D Flood Modeller Pro model with updated alignment of the 1D network and 1D-2D links in the TUFLOW model.

3.2.12. The width, height and invert levels of the culverts were estimated from the drawing provided by EDF Energy (drawing ref. '5m grid topo survey.dwg', provided on 25.05.2018).

3.3 Main development site

3.3.1. The proposed main development of the Sizewell C comprises three development areas (**Plate 3.3)**.

Plate 3.3: Sizewell C development sub-divisions within the redline boundary



3.3.2. Part of the development requires raising ground levels in areas of existing floodplain. Therefore, it is required as part of the FRA to assess the impact of the proposed main development site on off-site flood risk.

3.3.3. Further to the main platform, the Sizewell C projects comprises of other elements required for the construction and operation of the facilities within the different development areas (**Plate 3.4**).

3.3.4. Some of the development areas would only be present during the construction phase and others form the permanent development that would have a theoretical maximum site lifetime of up to 2190.

3.3.5. List of all components of the development is provided in **Table 3.1** and **the** indicative construction layout plan is illustrated in **Figure 3A.1** in the Description of Development.

Table 3.1: Summary of development components

Development Locations	Component Description	Temporary	Permanent
Main Platform	Main power station platform, realignment of Sizewell Drain and northern mound redevelopment.		x
	Flood defence and coastal protection measures.		x
	Beach landing facility and private access road.		x
	Fuel and waste storage facilities, including interim spent fuel and waste storage.		X
	Internal power station access roads.		X
	Operational service building, including offices, training centre, controlled access to the nuclear island, workshops, laboratories, medical and other welfare facilities.		X
	Auxiliary administration centre and storage facilities and buildings including meteorological station, conventional waste storage, transit areas.		X
	Drainage and sewerage infrastructure.		X
	A new National Grid 400kV substation.		X
	Six monopoles and four pylons to connect the conventional islands to the National Grid substation.		X
	Two nuclear islands with associated infrastructure.		X
	Two conventional islands and other associated infrastructure.		X
	Two onshore cooling water pumphouses and associated infrastructure.		X
	Marine works and associated infrastructure including cooling water structures with the fish recovery and return systems and combined drainage outfall in the North Sea.		X

Development Locations	Component Description	Temporary	Permanent
SSSI Crossing	A vehicular and pedestrian causeway crossing over Sizewell Marshes SSSI connecting the power station to the new access road to the north.		X
Construction Area	Temporary workers accommodation campus.	X	
	Leiston off-site sports facility at Alde Valley Academy shared facility		X
	Rail terminal with associated security and off-loading facilities (green rail route).	X	
	Common user facilities, including concrete batching plant and prefabrication facilities.	X	
	Construction contractors' compounds, including working areas, laydown areas, workshops and storage.	X	
	Site access and entrance hub with related parking, security, induction and temporary offices.	X	
	Car parking, bus interchange and heavy goods vehicle (HGV) holding area.	X	
	Temporary site access roads, earthworks haul roads and other temporary internal roads.	X	
	Site-wide infrastructure including drainage, lighting and environmental boundary treatment.	X	
	Spoil management including borrow pits and topsoil, subsoil and excavated material storage.	X	
	Construction electrical supply (CES) substation	X	
	Old Abbey Farm electrical substation.		X
	Upper Abbey Farm emergency equipment store and back-up generator.		X
	Car parking (including Kenton Hills improvements) and associated security buildings.		X
	Access road to the north of main platform, linking the causeway crossing with a new junction onto Abbey Road (B1122).		X

Development Locations	Component Description	Temporary	Permanent
Land East of Eastlands Industrial Estate	Caravan park for temporary accommodation and associated welfare facilities.	X	
	Stockpile areas	X	
	Freight management facilities including car park, HGV park and park and ride facilities	X	
	Temporary rail infrastructure including single railway line with sidings.	X	

3.3.6. To assess the construction phase flood risk, the development components incorporated into the model are: the main platform, the SSSI crossing, northern mound (with access road to beach landing facilities), main sea defence, access road, earth bund (acoustic bund in the temporary construction area along the temporary railway), part of the drainage infrastructure, namely water management zone 1 (WMZ1) and the temporary haul road.

3.3.7. The design levels of the included development components are given in **Table 3.2**.

Table 3.2: Summary of design levels for site components

Area	Design Levels (m AOD)
Main platform	7.3
SSSI crossing	7.3
Northern mound	10.2
Main sea defence	10.2
Access road	7.3
Earth bund	5
Water management zone 1 (bund around the zone)	5
Haul road	7.3

3.3.8. Only permanent features were incorporated in the model to assess the risks on-site and impacts or changes in flood risk off-site through operation and decommissioning phases of the Sizewell C project. The model schematisation for operation and subsequent phases includes the permanent features of the main platform, the SSSI crossing, northern mound, main sea defence, access road and haul road embankment.

3.3.9. Although some of the permanent development components would be removed in the early stages of the decommissioning phase. A conservative approach was adopted for the modelling that included all permanent components up to the theoretical maximum site lifetime.

3.3.10. Other components of the development are outside of Flood Zone 2 and 3 and modelled baseline flood extent. These components were not included at this stage of modelling, as they are unlikely to have any impact on flood risk.

3.3.11. The 'with scheme' model is similar to the baseline scenario with Aldhurst Farm and includes the main platform, SSSI crossing and the realignment of Sizewell Drain (**Plate 3.4**). Cross-sections of the realigned Sizewell Drain channel were assumed to have similar geometry to the current channel.

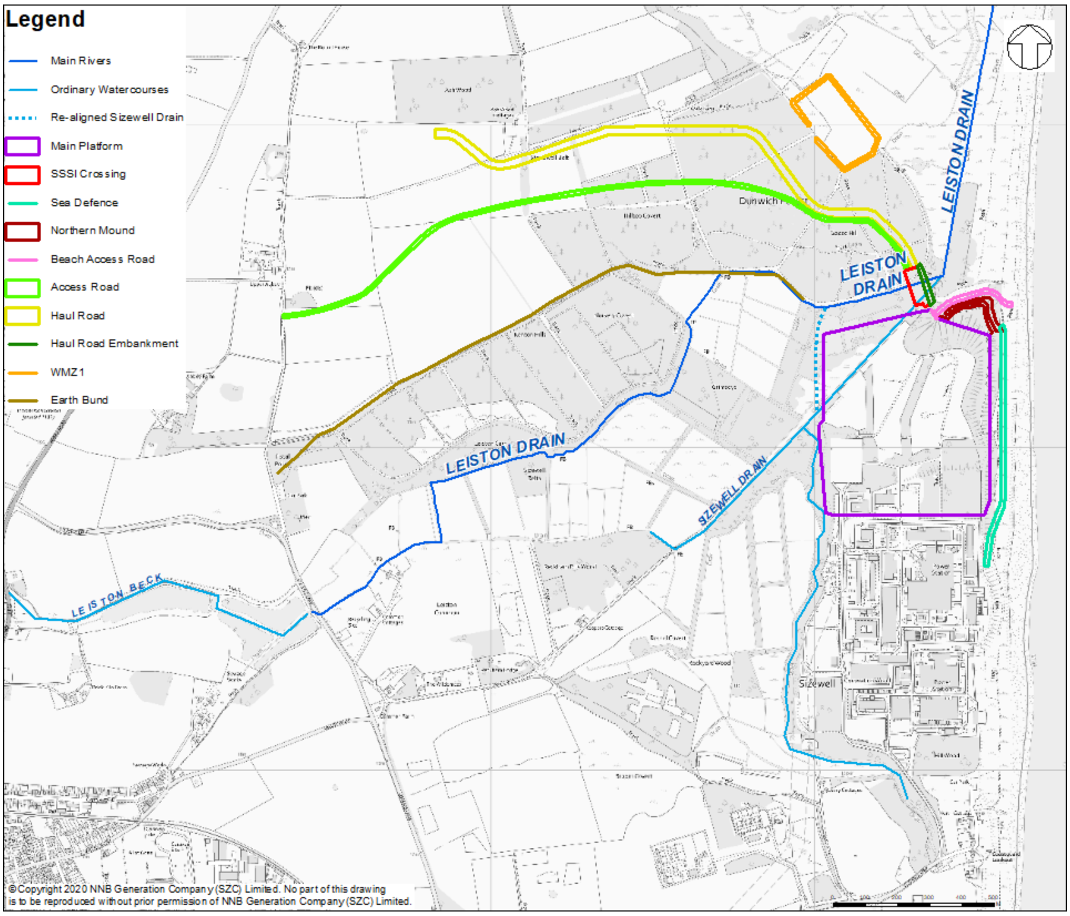
3.3.12. The outline and height of the main platform and the earth bund along the southern boundary of the temporary construction area were taken from drawing No SZC-SZ0100-XX-000-DRW-100000 (provided by EDF in July 2018) (**Plate 3.4**).

3.3.13. The SSSI crossing was added in both the 1D and 2D models. In the 1D model, the culvert opening was represented as a culvert with three conduit units with a total length of 69.5m, each 8m wide and 4.5m high. The SSSI crossing road deck level and outline were represented in the 2D model.

3.3.14. The dimensions of the culvert under the road as well as soffit and invert levels, road width and elevation were taken from drawings provided by EDF in June 2018 (No. SZC-SZC008-XX-000-DRW-100000, 100001 and 100002).

3.3.15. **Plate 3.4** shows all features of the development (temporary and permanent) adopted in the model for the construction phase simulations. Following completion of the assessment further details became available on the extent of the temporary earth (acoustic) bund, where the bund would end around the extent of the Nursery Covert and therefore would not run along the Leiston Drain as illustrated below.

Plate 3.4: Proposed 'with scheme' model schematisation



4 BOUNDARY CONDITIONS

4.1 Overview

4.1.1. Fluvial inputs into the model comprise inflow hydrographs, rainfall hyetographs and tide levels. Joint probability of fluvial events and tidal levels was carried out as outlined in Environment Agency 'Use of Joint Probability Methods in Flood Management A Guide to Best Practice Report' (Ref 4).

4.1.2. The following subsections provide some details on the model boundaries and climate change scenarios used in this assessment. Further detail on derivation of the boundary conditions is provided in the Sizewell C Hydrology Review Report (Ref 2).

4.2 Inflow boundaries

4.2.1. All FEH boundaries including the main inflow at Middleton (**Plate 4.1**), Leiston catchments and other incremental sub-catchments included in the 1D and 2D models were prepared for 10 different storm durations, which were later tested in the model to derive the critical storm duration for use in further assessment (see **section 6.1**).

4.2.2. Rainfall hyetographs for the same events were applied to the 'lowland' system in the downstream parts of the catchments in the 2D-TUFLOW model domain as shown in **Plate 4.2**.

Plate 4.1: Inflow hydrograph at Middleton for 1 in 100 year return period event and 121 hours storm duration

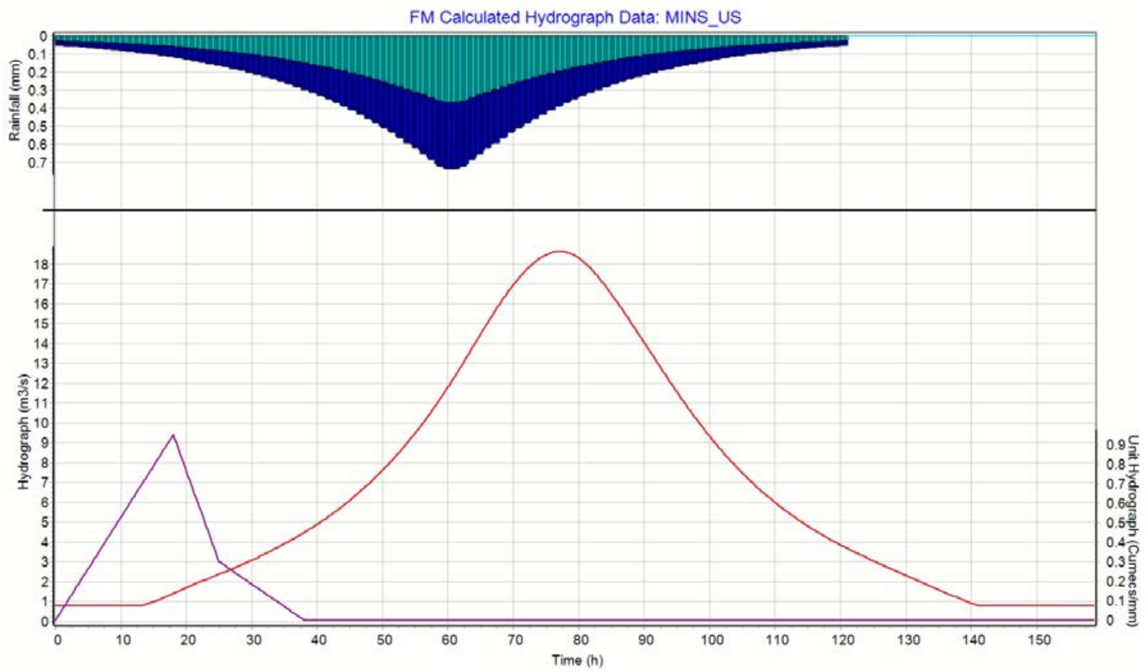
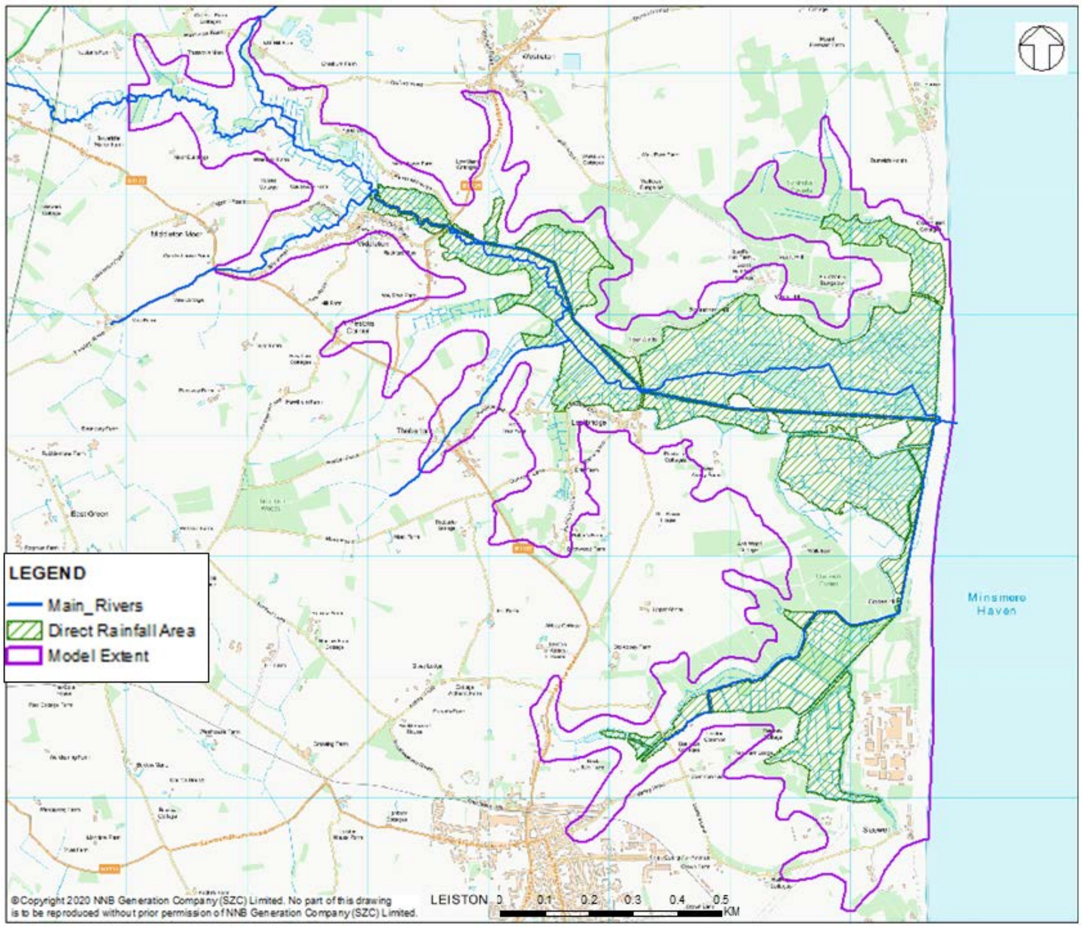


Plate 4.2: Direct rainfall area applied in the 2D-TUFLOW model



4.2.3. Rainfall inputs to the 'Lowland' system (from gauge data) have been prescribed as a 50% Percentage Runoff (PR) for the floodplain upstream of Eastbridge and 90% PR downstream of Eastbridge. These percentages were previously determined from analysis of the floodplain elevation and slope within the lowland area, more details are provided in the 'Sizewell C Fluvial Modelling Calibration Report' (Appendix B).

4.2.4. Downstream of Eastbridge, the floodplain was expected to be waterlogged. Therefore, it would exhibit a high percentage runoff, whereas upstream off this location, due to the higher (although still relatively gentle) gradient, a lower percentage runoff was expected.

4.2.5. **Plate 4.3** and **Plate 4.4** show the locations of the 1D and 2D model inflows respectively.

Plate 4.3: Location of 1D Flood Modeller inflows

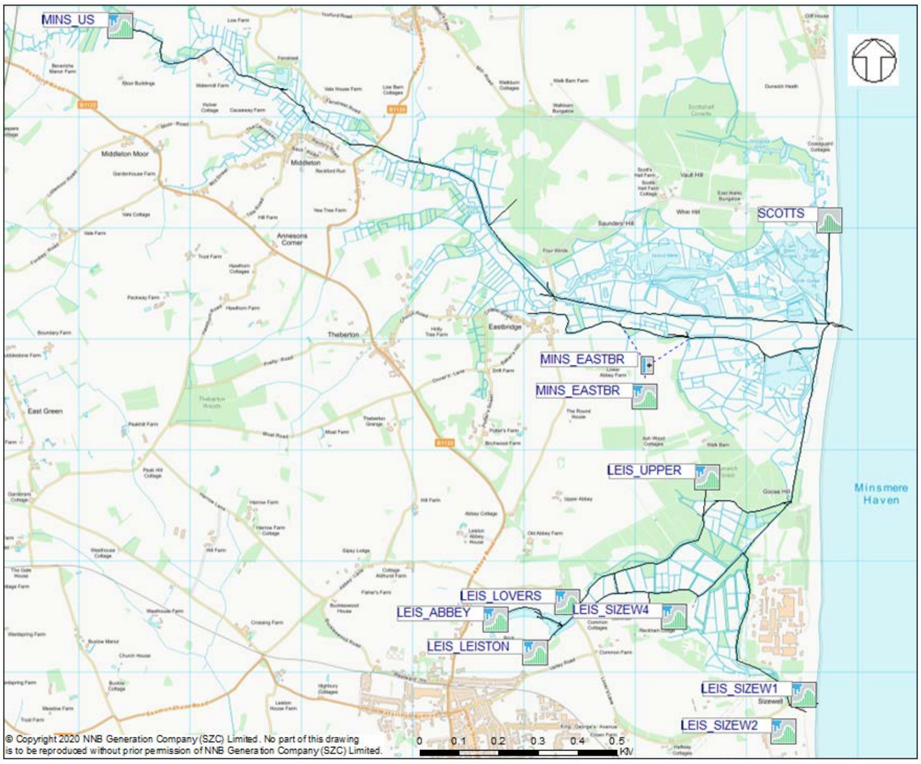
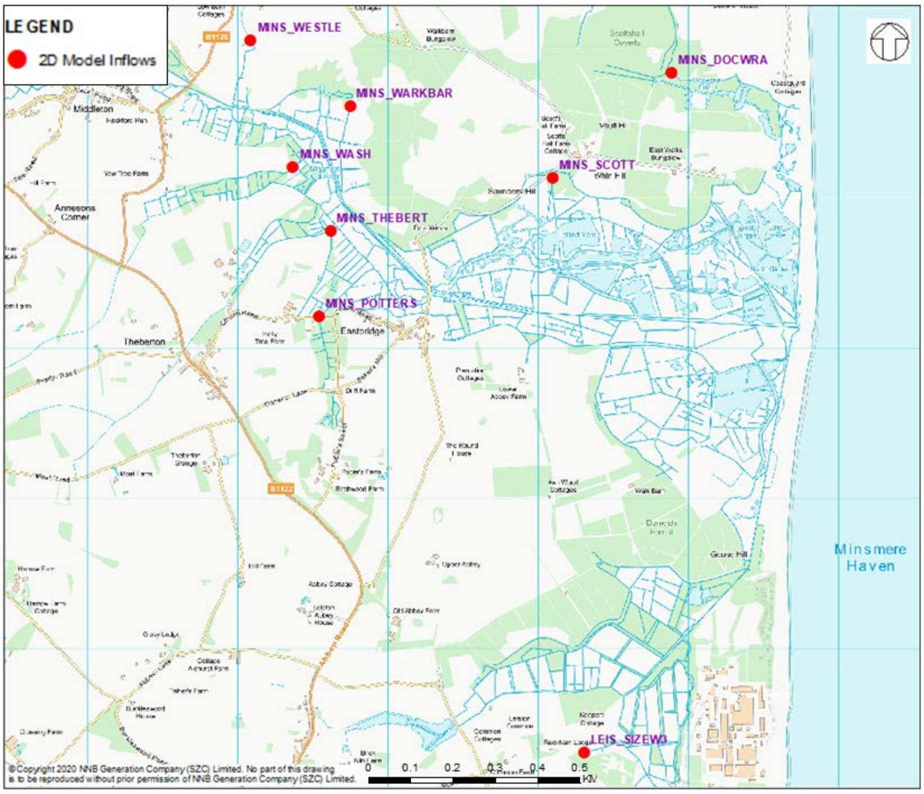


Plate 4.4: Location of 2D TUFLOW inflows



4.3 Tidal boundary

4.3.1. The hydraulic model of the Minsmere and Leiston systems required the tide level definition at the downstream boundary, where water is discharged into the sea via Minsmere Sluice.

4.3.2. Following the approach used for the calibration modelling, tidal levels at Minsmere were derived based on harmonic data for Lowestoft, obtained from Admiralty Tide Tables for 2017 (Ref 5), using a suitable relationship for transformation of tide levels from Lowestoft to Minsmere Sluice.

4.3.3. This transformation was derived based on high and low tide data for Lowestoft and Sizewell B stations (February 2009 to December 2012 supplied by CEFAS on behalf of EDF). Following discussions with CEFAS, the admiralty harmonics at Minsmere sluice were found not correct. Therefore, the adjusted Lowestoft tide levels were used as more appropriate for this study.

4.3.4. More details on the transformation method are provided in Sizewell C Fluvial Modelling Calibration Report (**Appendix B**).

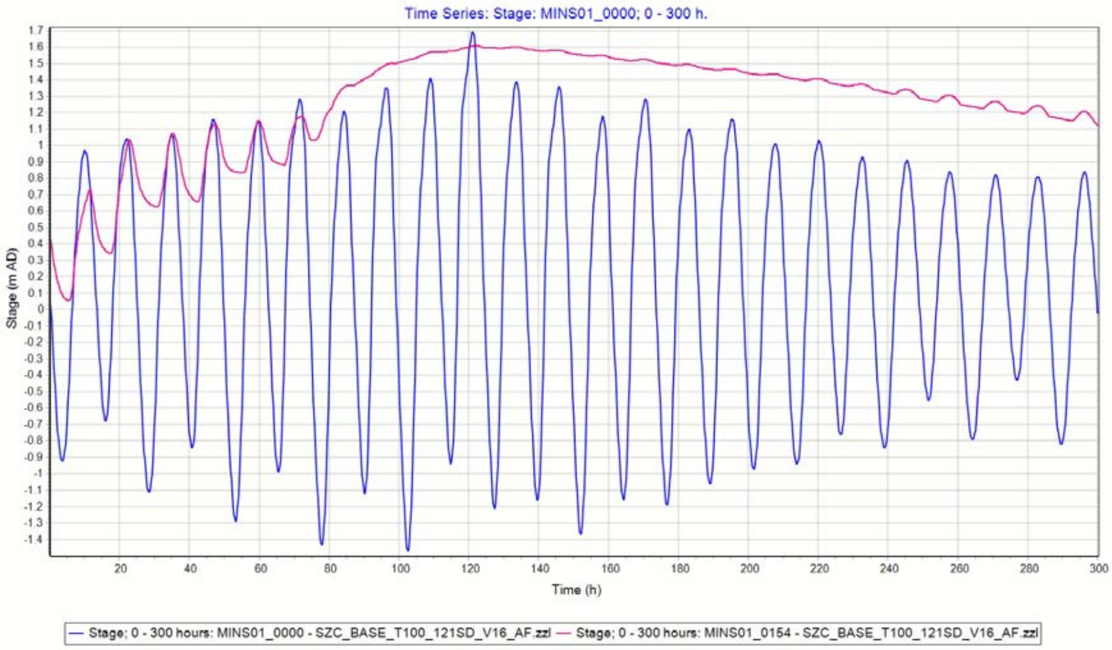
4.3.5. The timescale to generate the tide cycle was chosen with tide levels approximately at MHWS of 0.90m AOD at Lowestoft based on Admiralty Tide Tables (2017) and 1.05m AOD translated to Minsmere Sluice. The extreme tide levels for considered return period events were derived from Environment Agency Coastal Flood Boundary Dataset (CFBD) for UK updated in 2018 for point ID 4761, including confidence interval (Ref 6).

4.3.6. A joint probability of fluvial flows and tide levels was conducted with the focus of extreme fluvial flows. For example, for a 1 in 100-year joint probability event, a 1 in 100-year fluvial flow was used with the corresponding tide level. The assessment showed the marginal return periods for tide events between 2 years and 50 years are the same. Therefore, the same derived timeseries water levels were used for these scenarios.

4.3.7. The design tide level timeseries was derived in line with the Environment Agency guidance using Lowestoft surge shape (Ref 7). Further details on derivation of the tidal boundary is provided in the hydrology report (Ref 2).

4.3.8. The derived tide timeseries for the present day 1 in 100-year event with 121 hours storm duration is presented in **Plate 4.5**. The peak tide level, determined from the joint probability assessment, was aligned with the peak flow upstream of the Minsmere Sluice (model node MINS01_0154) based on the sensitivity testing carried out for critical storm duration (discussed in **section 6.1**). There might be some slight variation in the timings of the peak for other return period events or climate change epochs, however the 121 hours was determined to be appropriate for most scenarios. Further sensitivity testing and adjustment of the timings might be carried out at the later stages of the Sizewell C project.

Plate 4.5: Tidal model boundary for 1 in 100 year event



4.4 Climate change scenarios

4.4.1. Climate change allowances for increasing the future fluvial flows, rainfall intensity and sea level rise were applied in the Sizewell C fluvial model for assessment of future flood risk.

4.4.2. The Environment Agency's climate change allowances guidance (Ref 8) was used in conjunction with Environment Agency guidance: 'Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities' (Ref 9) and UK Climate Projections (UKCP18) published in November 2018 (Ref 10).

4.4.3. For increasing the fluvial flows, the 'higher central' and 'upper end' climate change scenarios were applied as the Sizewell C project is classed as 'Essential Infrastructure' in accordance with NPPF guidance (Ref 11). No extrapolation was applied for the epoch beyond 2100 (Ref 9).

4.4.4. The climate change allowance guidance (Ref 8) considers flooding of nuclear installations as an extreme consequence for which the scale of the flooding impact may extend far wider than the immediate locality of the flooding incident. The guidance advises to test an extreme case of climate change allowance to consider the potential residual flood risks. For that purpose, the H++ scenario was applied.

4.4.5. The adopted climate change allowances for key development phases are presented in **Table 4.1**.

Table 4.1: Climate change allowances for fluvial flood risk

Development phase	Year	Climate change scenario	Climate change allowance
End of construction / commissioning	2030	Upper End Allowance / H++ Scenario	+25%
End of operation	2090	Higher Central / Upper End / H++ Scenario	+35% / +65% / +80%
Interim spent fuel store decommissioned	2140		
Theoretical maximum site lifetime	2190		

4.4.6. In line with the Environment Agency guidance (Ref 9) for catchments above $5km^2$, adopted fluvial allowances were also applied for increase in peak rainfall intensity to the direct rainfall area within the model.

4.4.7. For the tidal model boundary, sea level rise allowances were applied in line with joint advice from the Office of Nuclear Regulations and the Environment Agency (Ref 12).

4.4.8. The UKCP18 RCP8.5 95th percentile 21st century projections were used up to 2125 and the exploratory projections were used beyond 2125, as advised by the Environment Agency (Ref 13). The derived cumulative sea level rise allowances (relative to 2017 base year) for the key development phases are presented in **Table 4.2**.

Table 4.2: Derived cumulative sea levels rise allowances (2017 base year) for Sizewell C key development phases

Development phase	Year	Climate change allowance (m)
End of construction / commissioning	2030	+0.094m
End of operation	2090	+0.867m
Interim spent fuel store decommissioned	2140	+1.761m
Theoretical maximum site lifetime	2190	+2.591m

4.4.9. Further details on derivation of climate change allowances are provided in the Hydrology Report (Ref 2) and 'UK Climate Change Projections 2018 - Review and Proposed Response' report (Ref 14).

4.4.10. Following completion of this fluvial flood risk assessment in October 2019, in December 2019 the Environment Agency has published updated guidance on climate change allowances for flood risk assessments (Ref 15). This has updated sea level rise allowances to reflect the latest climate change projections (UKCP18). The sea level rise allowances in that guidance are based on the UKCP18 RCP8.5 95th and 70th percentiles and provide an average figure for each scenario and therefore it is considered that the adopted allowances for the Sizewell C study (UKCP18 RCP8.5 95th percentile) are slightly more conservative.

4.4.11. The allowances for peak river flow and peak rainfall intensity in 'Flood risk assessments: climate change allowances' (Ref 15) have not be updated yet to reflect the changes based UKCP18 results. This is because high resolution rainfall projections were only published recently (September 2019) and research is still underway to assess the impact of the rainfall projections in UKCP18 on peak river flow. It is anticipated that Environment Agency would publish updates to these allowances in late 2020.

5 MODEL PARAMETERS AND STABILITY

5.1 Run parameters

5.1.1. All simulations were run using Flood Modeller version 4.3.0 and TUFLOW version 2017-09-AA-iDP-w64. The model was run to simulate 300 hours, which allowed suitable time for the hydrograph to pass through the catchment.

5.1.2. A fixed time-step of 2 seconds was applied to the 1D model and a time-step of 4 seconds was applied to the 2D model. These time-steps were chosen as they provided model stability and are appropriate given the 8m cell size of the 2D model domain grid.

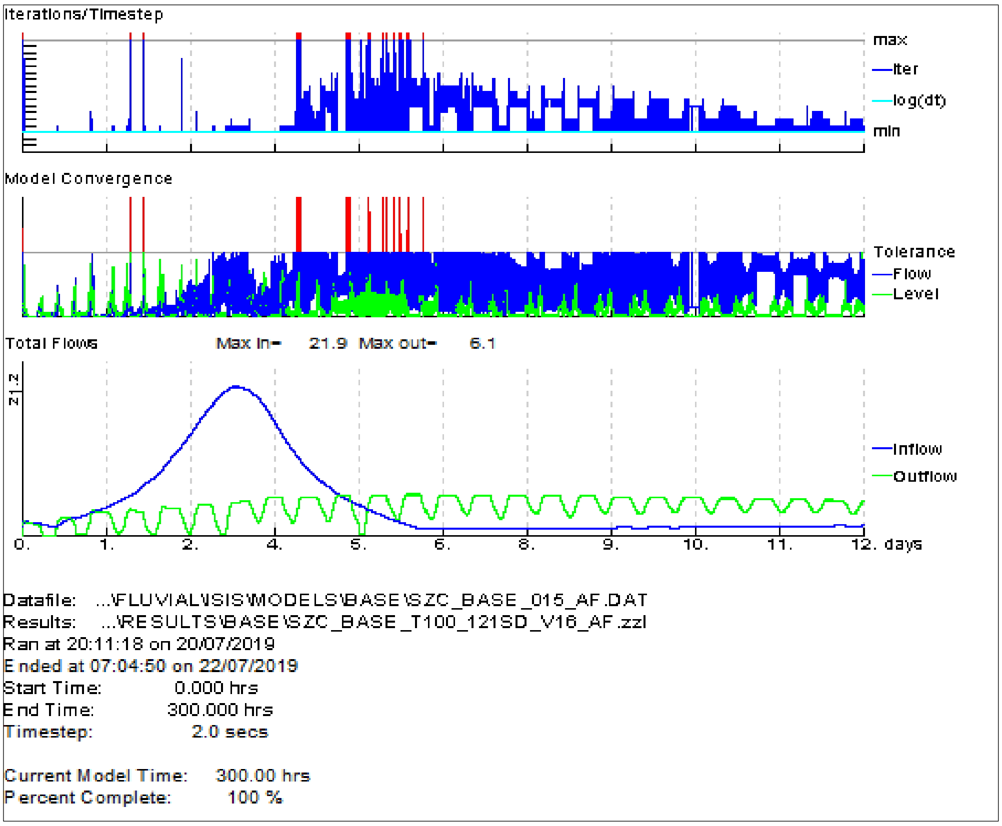
5.1.3. The following parameters were altered in the 1D model to improve overall stability and confidence in results:

- To prevent model failure due to the channel drying out at low-flows, the automated Preissmann slot option has been activated;
- The spill threshold value was changed from 0.000001 to 0.0001 to stabilise spill flow;
- The orifice linearisation head was changed from 0 to 0.01 to prevent oscillations at low head differences in the orifice unit;
- The Maxitr was raised from 6 to 17. Increasing the Maxitr allows for more calculations to be performed. In general, the larger the model the larger Maxitr should be set to improve model convergence;
- The Top Slot Height, specified global value for total depth of conduit top slot, measured from its opening to its top. The default value is zero. However, the Top Slot Height was defined as 7m in the model;
- All other 1D parameters were unaltered from default values.

5.2 1D model stability

5.2.1. All model simulations completed with some non-convergence. The guidelines for model acceptance recommended that non convergence should not be seen within two hours of the peak. The checks conducted using the present day 1 in 100-year return period event (1%AEP) show that limited poor convergence occurs from 38.6 hours to 42.94 hours and from 98.0 hours to 116.8 hours. The peak occurs at 77.25 hours, which is well outside of the 2-hour target window. The convergency plot for the 1 in 100-year return period is shown in **Plate 5.1**.

Plate 5.1: Convergence plot for 1 in 100-year event (present day)

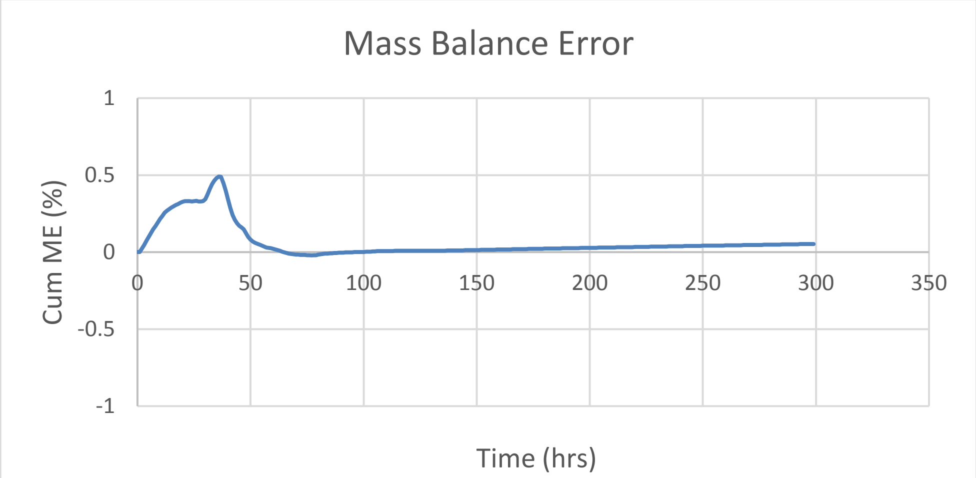


5.3 2D model stability

5.3.1. The numerical convergence has been checked through the examination of the mass balance time series within the TUFLOW output (MB2D.csv). The numerical convergence is considered very good if cumulative mass balance errors are less than 1%.

5.3.2. The mass balance output for the present day 1 in 100-year event is shown in **Plate 5.2**. The results of the mass error show that the model is within the expected tolerance. The maximum cumulative mass balance errors for the scenarios with climate change allowance (i.e. 1 in 100-year with 35%, 65% and 80% climate change allowances) is above the +1% expected tolerance, however that is limited to the first few hours of the simulation and therefore is considered not to have impact on overall model results.

Plate 5.2: 2D model mass balance plot for 1 in 100-year event.



6 MODEL RESULTS

6.1 Critical storm duration

6.1.1. Following changes to model schematisation to improve model stability, the baseline model was run for four scenarios with different storm duration-intensity of 5 hours, 49 hours, 73 hours, 97 hours, 121 hours and 145 hours. The results were compared to derive 'the worst' storm duration which results in highest peak water levels across the model and therefore highest fluvial flood risk.

6.1.2. The resulting flood depths and extents were compared for all tested storm durations, the timings of water going out of bank in the 1D model and the spilling into the flood plains in the 2D model.

6.1.3. It was confirmed that for the 1 in 100-year present-day scenario, the 121 hours storm duration is the worst in terms of flood risk, although the differences between the scenarios were very small.

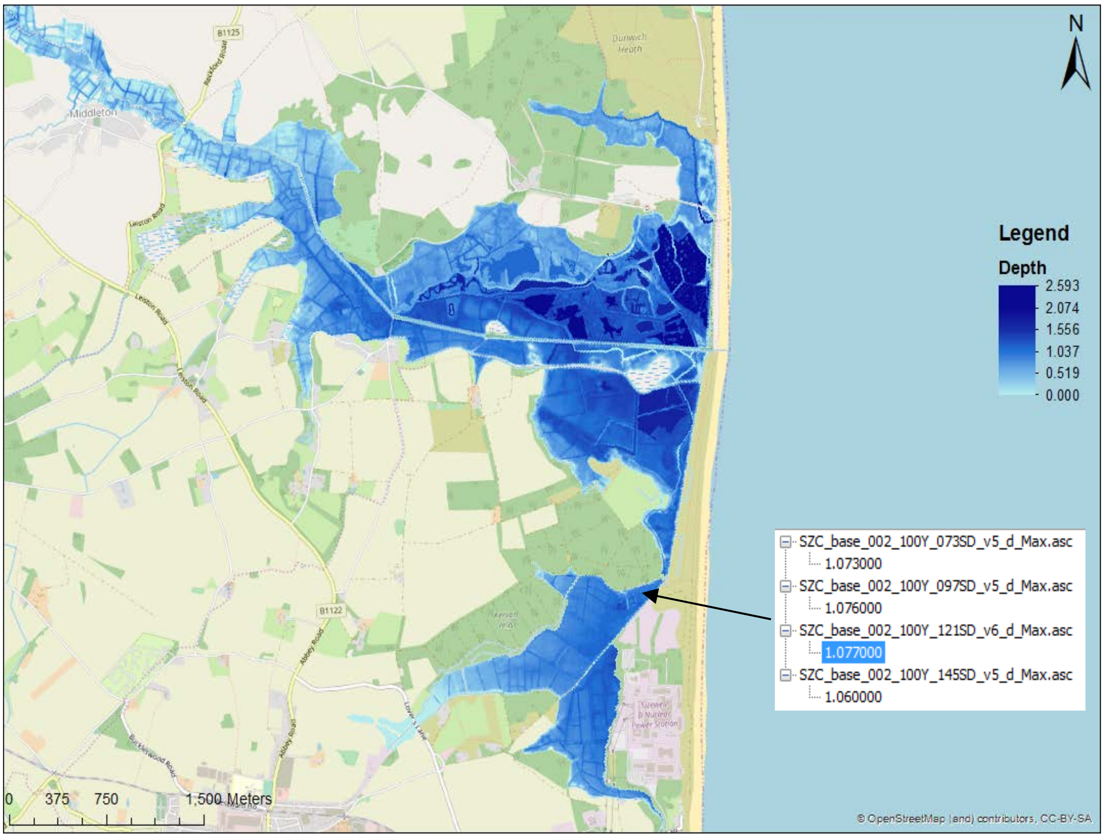
6.1.4. **Plate 6.1** presents flood depth map for a selected point in the vicinity of proposed location of the main platform.

6.1.5. **Table 6.1** presents comparison of maximum modelled water levels for the 1 in 100-year return period event and all considered storm durations for selected model nodes at key locations within the fluvial system. These locations are near Eastbridge, upstream of the Minsmere Sluice for watercourses and near the SSSI crossing.

Table 6.1: Comparison of maximum water levels for 1 in 100-year return period event and considered storm durations for selected model nodes at key locations

1D Model Node	Maximum Stage for 1 in 100-year return period event with respective storm duration (m AOD)					
	5 hrs	49 hrs	73 hrs	97 hrs	121 hrs	145 hrs
MINS01_2628u	1.417	1.501	1.594	1.597	1.597	1.580
MINS01_0154	1.284	1.484	1.582	1.585	1.589	1.574
SCOT_0000	1.308	1.490	1.588	1.591	1.593	1.577
DRA701_0000	1.257	1.496	1.592	1.595	1.596	1.580
SIZE01_0000	1.258	1.496	1.592	1.596	1.597	1.580
LEIS01_1646d	1.258	1.496	1.592	1.596	1.596	1.580
LEIS01_0000	1.256	1.490	1.588	1.591	1.593	1.577

Plate 6.1: Flood extent and depths for the 1 in 100-year return period event and four storm duration scenarios



6.2 Baseline model results

6.2.1. Following results from the critical storm duration model runs, the 121-hour event was used for further assessment of flood risk for the baseline scenarios that includes Aldhurst Farm and the 'with scheme' model schematisation that includes the Sizewell C main development components.

6.2.2. The baseline model was simulated for a range of return period events, 1 in 5-year, 1 in 20-year, 1 in 100-year and 1 in 1,000-year, with and without climate change allowances of 25%, 35%, 40%, 65% and 80%. Results from the model runs with 40% climate change allowance, which represent the ‘2050s’ epoch (Ref 8), were not presented in this report as they don’t represent a key point in time of the Sizewell C development (**Table 4.1**).

6.2.3. **Plate 6.2** presents timeseries results of water levels at Leiston Drain immediately downstream of the confluence with Leiston Ditch, 1D model node LEIS01_1646d (downstream of the proposed main platform and the SSSI crossing), for the 1 in 100-year return period event for the present-day and with 25%, 35%, 65% and 80% climate change allowance scenarios.

Plate 6.2: Timeseries of water level at Leiston Drain, downstream of the main platform location (LEIS01_1646d) – 1 in 100-year return period (present-day and with 25%, 35% 65% and 80% climate change)

6.2.4. **Plate 6.2** shows that maximum water level for the 1 in 100-year event with 80% allowance for climate change is less than 2.2m AOD.

6.2.5. The results show that in the present-day scenario, the majority of the proposed main platform area is not at risk of flooding. Only the north-west corner of the proposed platform location is within the western floodplain of the Leiston Drain. For the model runs with climate change allowance, the low-lying area behind the coastal defence and the northern mound, i.e. low-lying area where the platform would be located gets flooded.

6.2.6. **Plate 6.3** to **Plate 6.5** show flood depths for the baseline scenario for the 1 in 100-year with 35%, 1 in 100 year with 65% and 1 in 1,000-year with 65% return periods respectively.

Plate 6.3: Flood depth for baseline scenario for the 1 in 100-year return period event with 35% climate change allowance

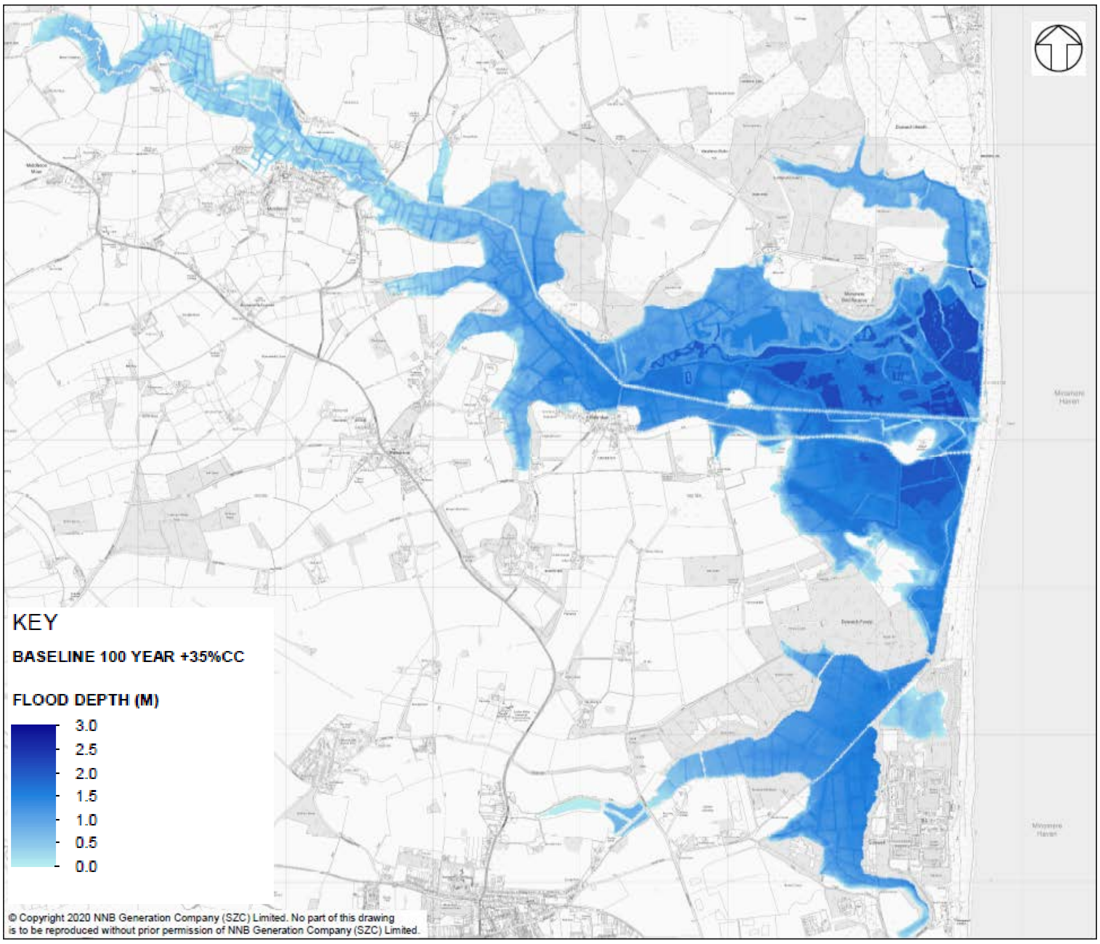


Plate 6.4: Flood depth for baseline scenario for the 1 in 100-year return period event with 65% climate change allowance

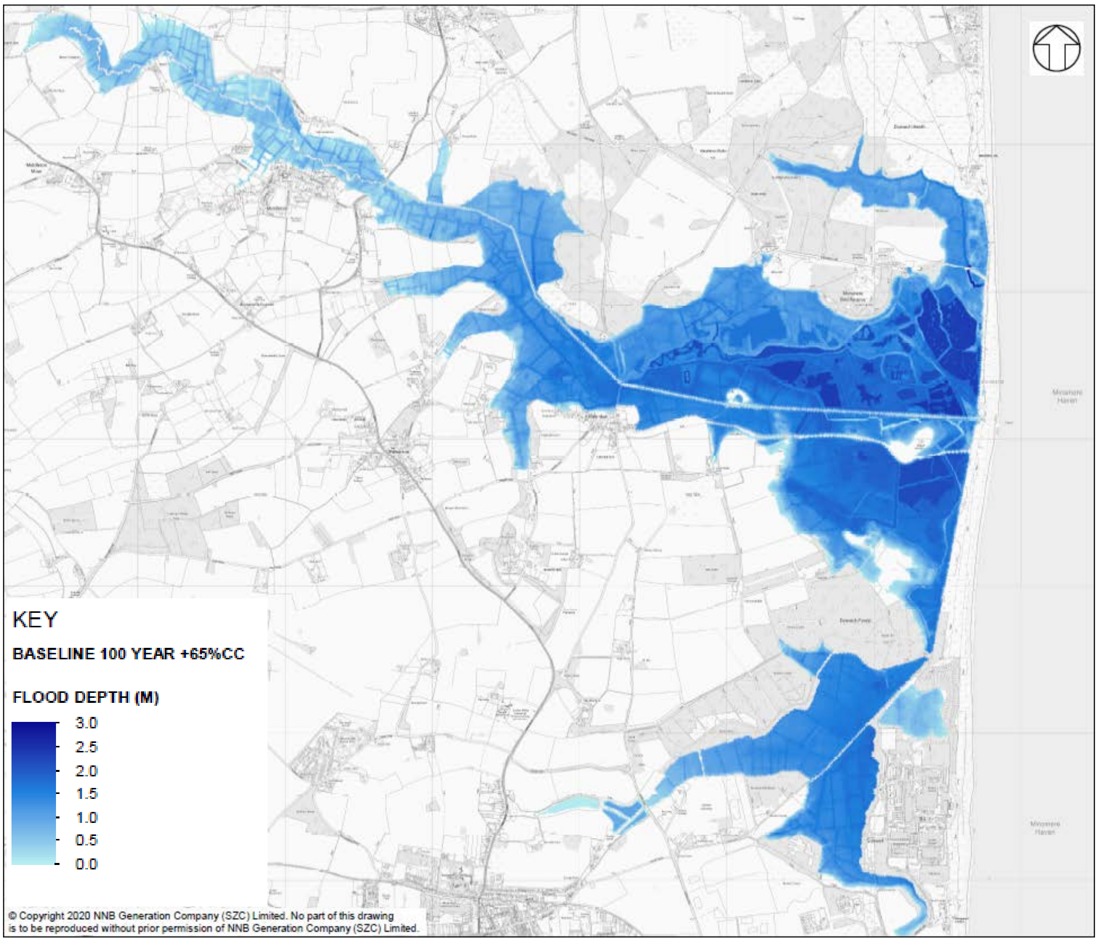
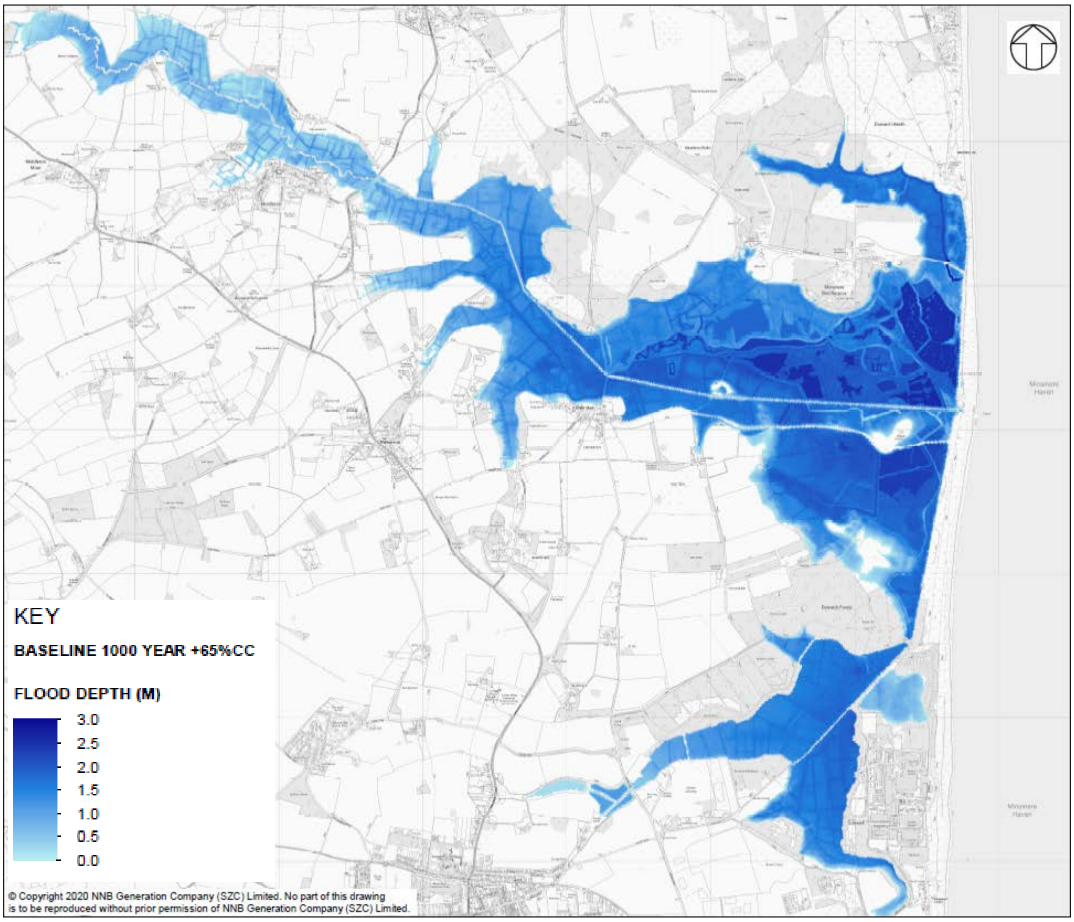


Plate 6.5: Flood depth for baseline scenario for the 1 in 1,000-year return period event with 65% climate change allowance



6.2.7. The number of properties at risk for the baseline scenario is summarised in **Table 6.2** for all considered return period events and climate change allowances.

6.2.8. **Table 6.2** shows for the present-day scenario 5 residential properties located in Leiston area are at risk for the 1 in 100-year event with 35% climate change allowance, while 8 properties are at risk for the 1 in 1,000-year event with 35% climate change allowance. The total numbers of properties at risk, including non-residential properties, are 10 and 14 for the respective return periods.

6.2.9. With regard to flood hazard rating, two of the residential properties are within the 'Low hazard' class, two within 'Danger for some' and one property is at 'Danger for most' class.

Table 6.2: Number of properties at flood risk – baseline scenario

Return Period	Number of properties at risk of flooding		
	Residential	Non-residential	Total
5 year present-day	0	1	1
5 year +25%CC	0	1	1
5 year +35%CC	1	1	2
5 year +65%CC	4	4	8
5 year +80%CC	5	5	10
20 year present-day	1	1	2
20 year +25%CC	1	1	2
20 year +35%CC	3	4	7
20 year +65%CC	5	5	10
20 year +80%CC	6	5	11
100 year present-day	1	4	5
100 year +25%CC	4	5	9
100 year +35%CC	5	5	10
100 year +65%CC	6	6	12
100 year +80%CC	7	6	13
1,000 year present-day	5	5	10
1,000 year +25%CC	6	6	12
1,000 year +35%CC	8	6	14
1,000 year +65%CC	9	8	17
1,000 year +80%CC	9	10	19

6.2.10. The full list of properties affected (NRD dataset within model domain) with corresponding maximum flood depth, velocity and hazard rating for all considered scenarios are collated in a spreadsheet provided in **Appendix C** with a summary table in the first tab.

6.2.11. **Plate 6.6** and **Plate 6.7** present flood hazard maps for the baseline scenario for 1 in 100-year return period event with 35% and 65% allowances for climate change respectively.

6.2.12. The presented hazard maps indicate the majority of the proposed main platform area is at flood risk with varying hazard ratings between 'Low' and 'Danger for All' (**Plate 6.6** and **Plate 6.7**).

6.2.13. The most significant area of risk within the area of the main platform is to the north-west of the existing Sizewell Drain. This is at 'danger for most' for both the 1 in 100-year event with 35% and 65% climate change allowance scenarios. However, for the platform area to the east of Sizewell Drain, the hazard rating is mostly 'danger for some' in the 1 in 100-year event with 35% climate change and rises to 'danger for most' for the 1 in 100-year event with 65% climate change allowance.

6.2.14. None of the proposed Sizewell C development components are within the 'Danger for all' flood hazard rating extent.

Plate 6.6: Flood hazard map for baseline scenario for 1 in 100-year with 35% climate change return period

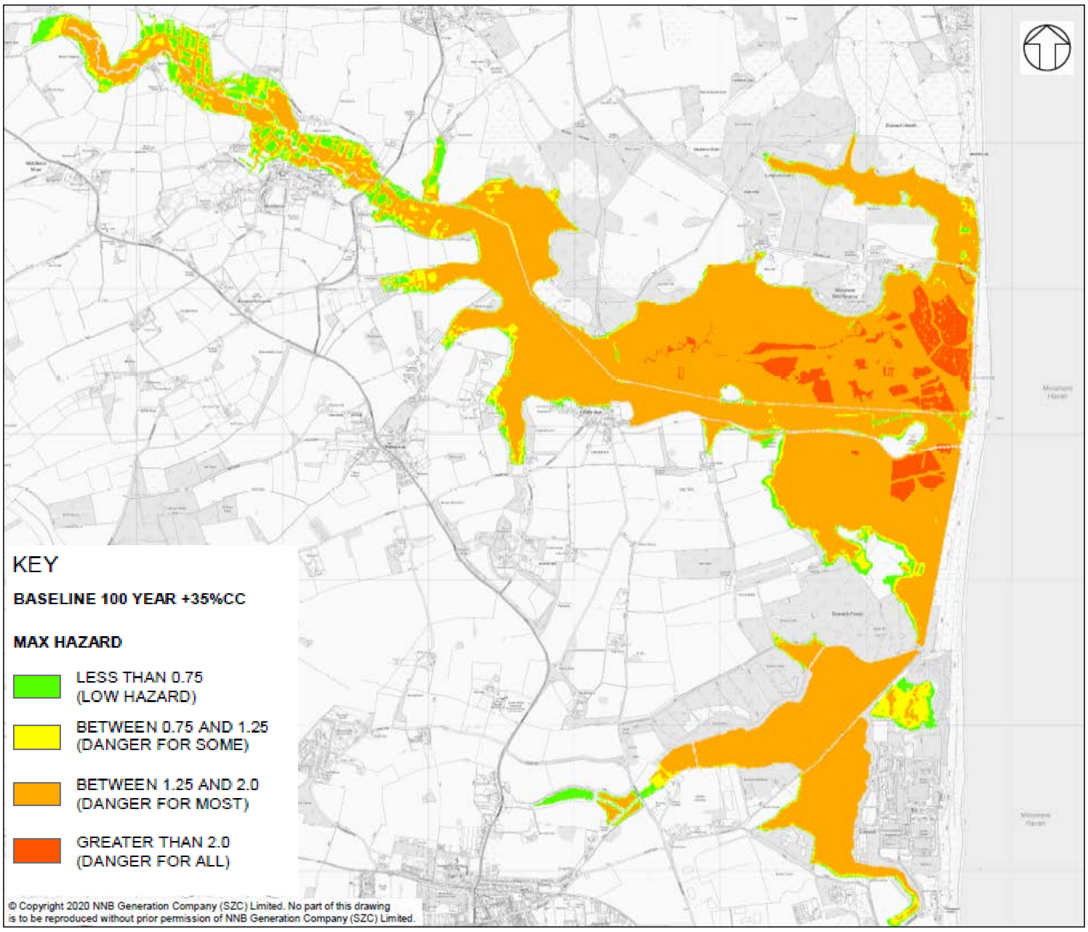
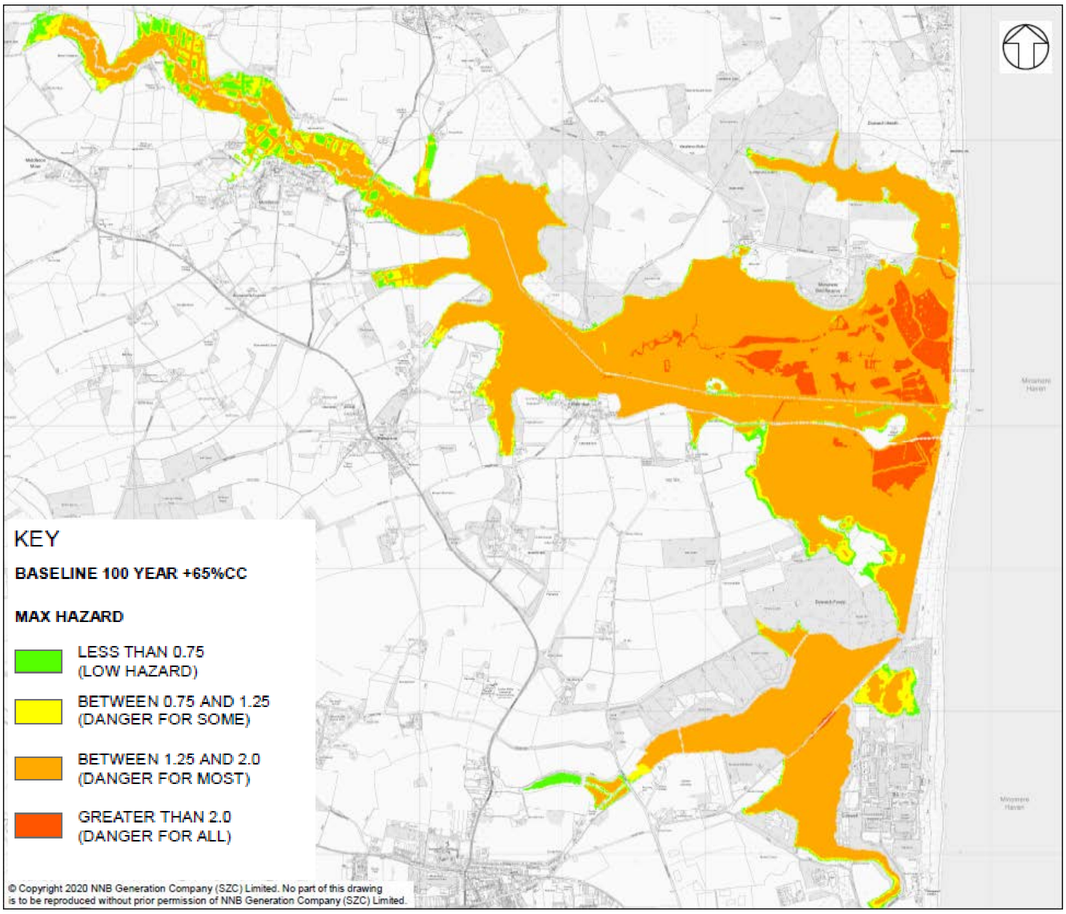


Plate 6.7: Flood hazard map for baseline scenario for 1 in 100-year with 65% climate change return period



6.2.15. Figures illustrating flood depth, hazard and velocity maps for the baseline scenario for 1 in 5-year, 1 in 20-year, 1 in 100-year and 1 in 1,000-year return periods with climate change allowances of 35% and 65% are provided in **Appendix C**.

6.2.16. The maximum velocity figure in **Appendix C** for the 1 in 100-year return period event with 35% climate change allowance illustrates that for the majority of catchment the maximum velocity is below 0.25m/s with some areas in the upper part of the Minsmere catchment of velocity up to 0.5m/s and very small only localised areas of velocity up to 1m/s. The velocities within the area of the proposed Sizewell C development are up to 0.25m/s.

6.2.17. A full set of 1D and 2D model results and processed grids for all considered scenarios for the baseline and 'with scheme' schematisations is available in a digital format.

6.3 With scheme

6.3.1. The "'with scheme'" model with Sizewell C main development components was simulated for 1 in 5-year, 1 in 20-year, 1 in 100-year and 1 in 1,000-year, with climate change allowances of 25%, 35%,65% and 80%.

6.3.2. **Plate 6.8** presents timeseries results of water levels from the with scheme model runs at Leiston Drain immediately downstream of the confluence with Leiston Ditch, 1D model node LEIS01_1646d (downstream of the proposed main platform and the SSSI crossing), for the 1 in 100-year return period event with 25%, 35%, 65% and 80% climate change allowance scenarios.

Plate 6.8: Timeseries of water level at Leiston Drain, downstream of the main platform location (LEIS01_1646d) – 1 in 100-year return period (with 25%, 35% 65% and 80% climate change)

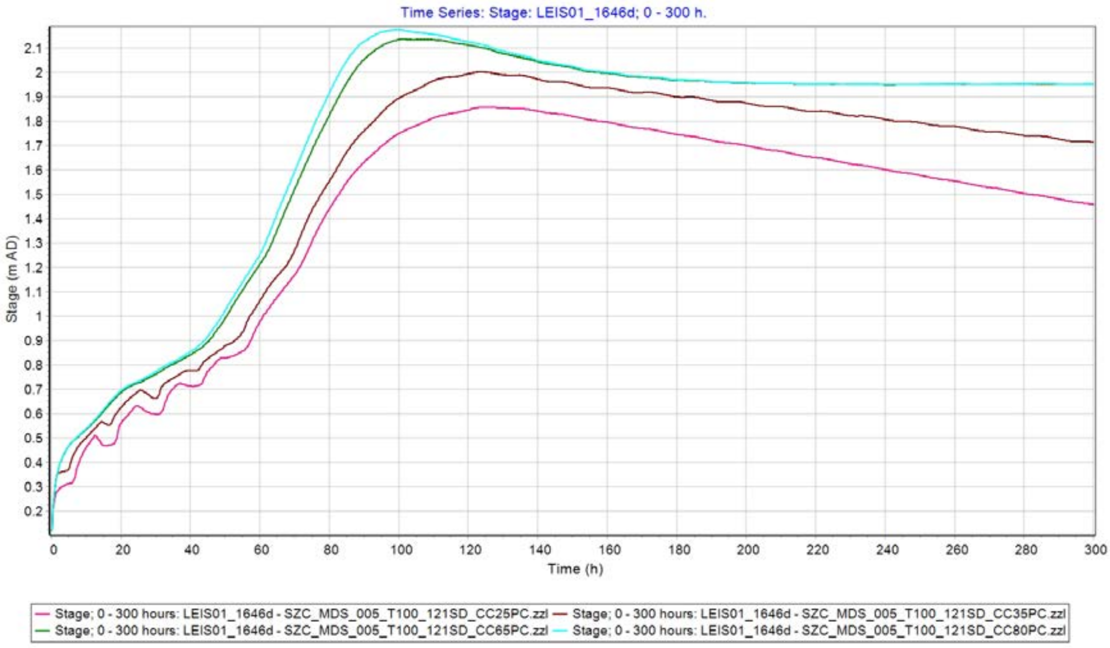


Plate 6.9 and **Plate 6.10** show maximum flood depths for the 'with scheme' scenario for 1 in 100-year return period events with 35% climate change and 1 in 100-year with 65% climate change respectively.

6.3.3. The results show the main platform and the SSSI crossing areas are not at risk of flooding from the considered extreme fluvial events. The flood extents for the two presented scenarios are very similar with increase in flood depth for the 1 in 100-year event with 65% climate change allowance.

Plate 6.9: Flood depth for 'with scheme' scenario for the 1 in 100-year return period event with 35% climate change allowance

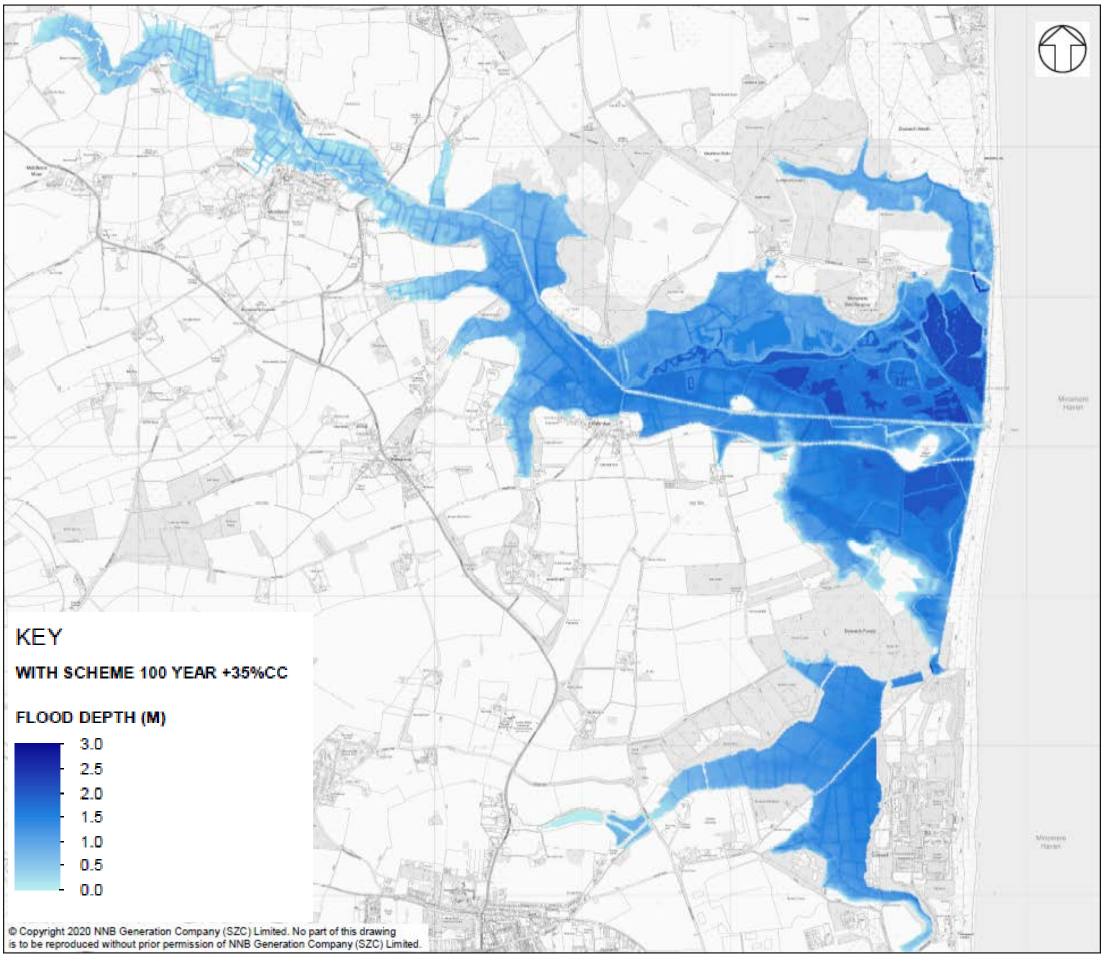
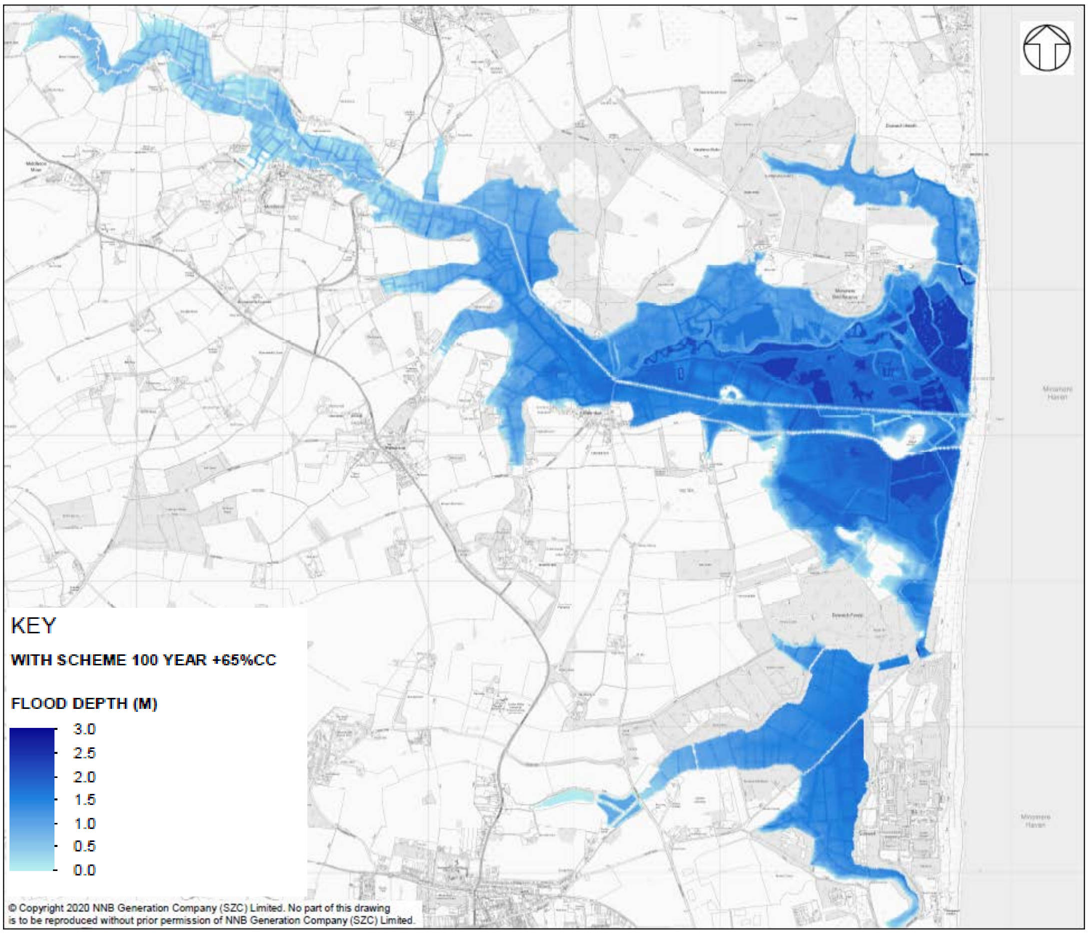


Plate 6.10: Flood depth for 'with scheme' scenario for the 1 in 100-year return period event with 65% climate change allowance



6.3.4. The number of properties at flood risk for the 'with scheme' scenario for all considered return period events and climate change allowances is summarised in **Table 6.3**.

Table 6.3: Number of properties at flood risk – 'with scheme' scenario

Return Period	Number of Properties flooded		
	Residential	Non-Residential	Total
5 year +25%CC	0	6	6
5 year +35%CC	1	6	7
5 year +65%CC	4	9	13
5 year +80%CC	5	10	15
20 year +25%CC	1	6	7
20 year +35%CC	3	9	12
20 year +65%CC	5	10	15
20 year +80%CC	6	10	16
100 year +25%CC	4	10	14

Return Period	Number of Properties flooded		
	Residential	Non-Residential	Total
100 year +35%CC	5	10	15
100 year +65%CC	6	11	17
100 year +80%CC	7	11	18
1,000 year +25%CC	6	11	17
1,000 year +35%CC	8	11	19
1,000 year +65%CC	9	14	23
1,000 year +80%CC	9	16	25

6.3.5. **Table 6.3** shows that at the end of construction phase, 4 residential properties are at risk for the 1 in 100-year event with 25% climate change allowance and 6 properties for the 1 in 1,000-year event with 25% climate change allowance. The total numbers of properties, including non-residential properties, at risk are 14 and 17 respectively.

6.3.6. For the operation phase scenarios and up to end of theoretical maximum site lifetime, the total numbers of properties at risk are 15 and 19 for the 1 in 100-year +35% climate change and 1 in 1,000-year +35% climate change events respectively.

6.3.7. A full list of properties affected (NRD dataset within model domain) with corresponding maximum flood depth, velocity and hazard rating for all considered scenarios are collated in a spreadsheet provided in **Appendix D** with a summary table in the first tab.

6.3.8. Figures illustrating flood depth, hazard and velocity for the 'with scheme' scenario for 1 in 5-year, 1 in 20-year, 1 in 100-year and 1 in 1,000-year return periods with climate change allowances of 35% and 65% are provided in **Appendix D**.

6.3.9. Since the main platform and the SSSI crossing are not flooded at any of the considered events, there is no flood velocity or hazard in those parts of the development. Other areas of the development, i.e. the temporary construction area, are located mostly outside the flood extent or on the edge of the flood extent (water management zone 1) where flood depth, velocity and hazard are low.

6.4 Comparison

6.4.1. To assess the potential impact of the Sizewell C development on flood risk to off-site receptors, a comparison has been made to determine the change in flood extent, depth, velocity and hazard throughout the development lifetime.

6.4.2. **Table 6.4** presents comparison of modelled maximum water levels for the baseline and 'with scheme' scenarios for all considered return period events and climate change allowances at the 1D model node located downstream of the proposed main platform and the SSSI crossing (LEIS01_1646d) as shown in **Plate 6.11**.

Plate 6.11: Location of the LEIS01_1646 model node

Table 6.4: Difference in maximum water levels at node LEIS01_1646d downstream of SSSI crossing.

Return Period	Climate change allowance	Max Water Level (m AOD)		Difference (m)
		Baseline	With scheme	
5 year	25%	1.325	1.332	0.007
	35%	1.449	1.458	0.009
	65%	1.847	1.859	0.012
	80%	1.912	1.925	0.013
20 year	25%	1.562	1.570	0.008
	35%	1.711	1.722	0.011
	65%	2.010	2.020	0.010
	80%	2.047	2.059	0.012
100 year	25%	1.843	1.857	0.014
	35%	1.992	2.002	0.010

Return Period	Climate change allowance	Max Water Level (m AOD)		Difference (m)
		Baseline	With scheme	
	65%	2.130	2.137	0.007
	80%	2.168	2.177	0.009
1,000 year	25%	2.129	2.138	0.010
	35%	2.187	2.196	0.009
	65%	2.287	2.297	0.010
	80%	2.311	2.323	0.012

6.4.4. The results in **Table 6.4** show for the construction phase, the maximum change in flood levels is 14mm for the 1 in 100-year return period event with 25% climate change allowance. Whereas for the operation and decommissioning phases, the maximum differences in flood levels is 11mm for the 1 in 20-year return period event with climate change allowance of 35% allowance.

6.4.5. The results for the runs with reasonably foreseeable and credible maximum climate change allowances (Upper End scenario with 65% and H++ scenario with 80% increase in fluvial flows respectively) in **Table 6.4** show the maximum difference in flood levels is 13mm (5 year event with 80% climate change), which is less than difference for the 100-year event with 25% climate change for the reasonably foreseeable scenario. This is caused by Minsmere Levels and Sizewell Belts being flooded to higher levels in both baseline and 'with scheme' scenarios, leading to lower relative differences.

6.4.6. The differences in maximum water levels and velocities for all 1D model nodes are provided in corresponding spreadsheets in **Appendix E**. There is also a spreadsheet with collated results and differences in flood depth, velocity and hazard rating for all properties located within the model extent in **Appendix E**.

6.4.7. The collated results in **Appendix E** show that overall difference in maximum stage for all model nodes is less than 30mm for up to 1 in 1,000-year return period event with 25%CC. For the most extreme events (1 in 1,000-year with 35%, 65% and 80%) difference in maximum stage is higher at the downstream end of Drain No7 (up to 150mm), however this change is very localised (limited to immediate floodplain around the watercourse) and does not affect any receptors.

6.4.8. **Plate 6.12** and **Plate 6.13** show a comparison of the timeseries water levels between the baseline and 'with scheme' scenarios at the upstream reach of the Sizewell Drain near Sizewell village (model node SEIZ01_1585) and at the Drain no.7 at Eastbridge (model node DRA701_2715) for the 1 in 100 year return period event with 35% allowance for climate change.

6.4.9. Both plates illustrate that duration of increase in flood levels is limited to the peak time and therefore the overall duration of flooding is not increased.

Plate 6.12: Comparison of timeseries water levels for the baseline (blue) and 'with scheme' (pink) scenarios at the Sizewell Drain near Sizewell village (SEIZ01_1585) for the 1 in 100-year return period with 35% climate change allowance

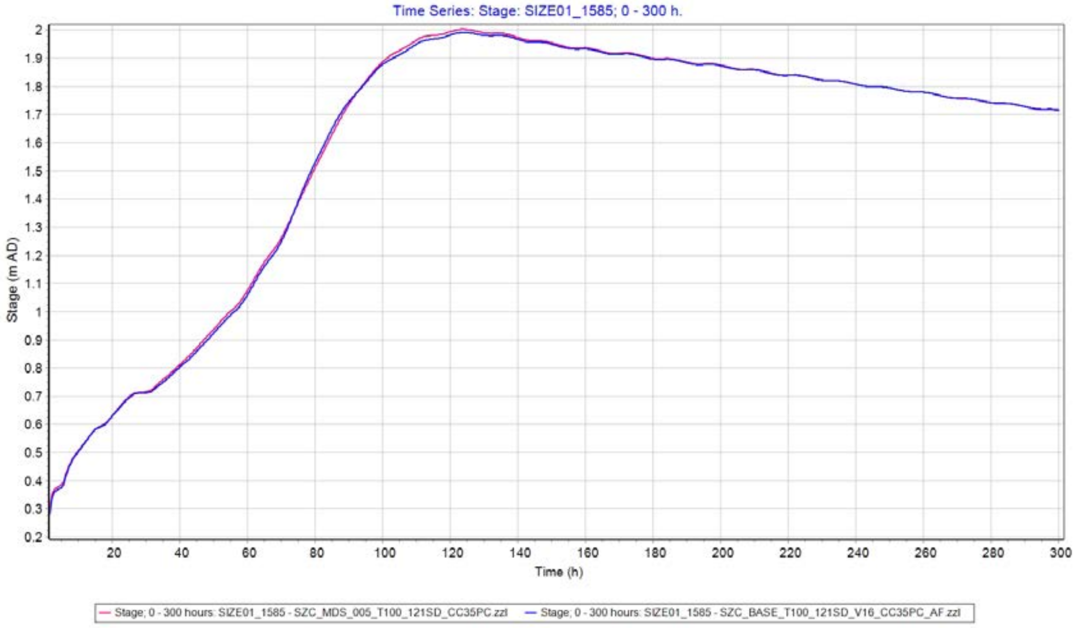
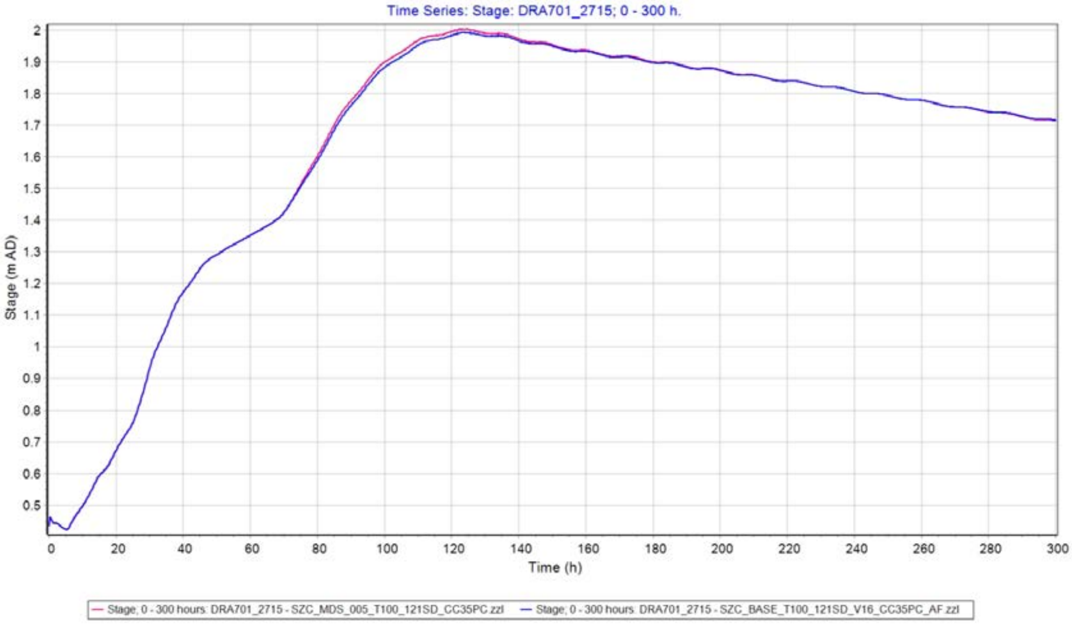


Plate 6.13: Comparison of timeseries water levels for the baseline (blue) and 'with scheme' (pink) scenarios at the Drain no.7 near Eastbridge (DRA701_2715) for the 1 in 100-year return period with 35% climate change allowance



6.4.10. **Plate 6.14** to **Plate 6.16** illustrate the differences in the maximum flood depths, 'with scheme' minus baseline, for the 1 in 100-year return period with 35%, 65% 80% allowance for climate change respectively.

Plate 6.14: Difference in maximum flood depth for the 100-year return period with 35% climate change allowance

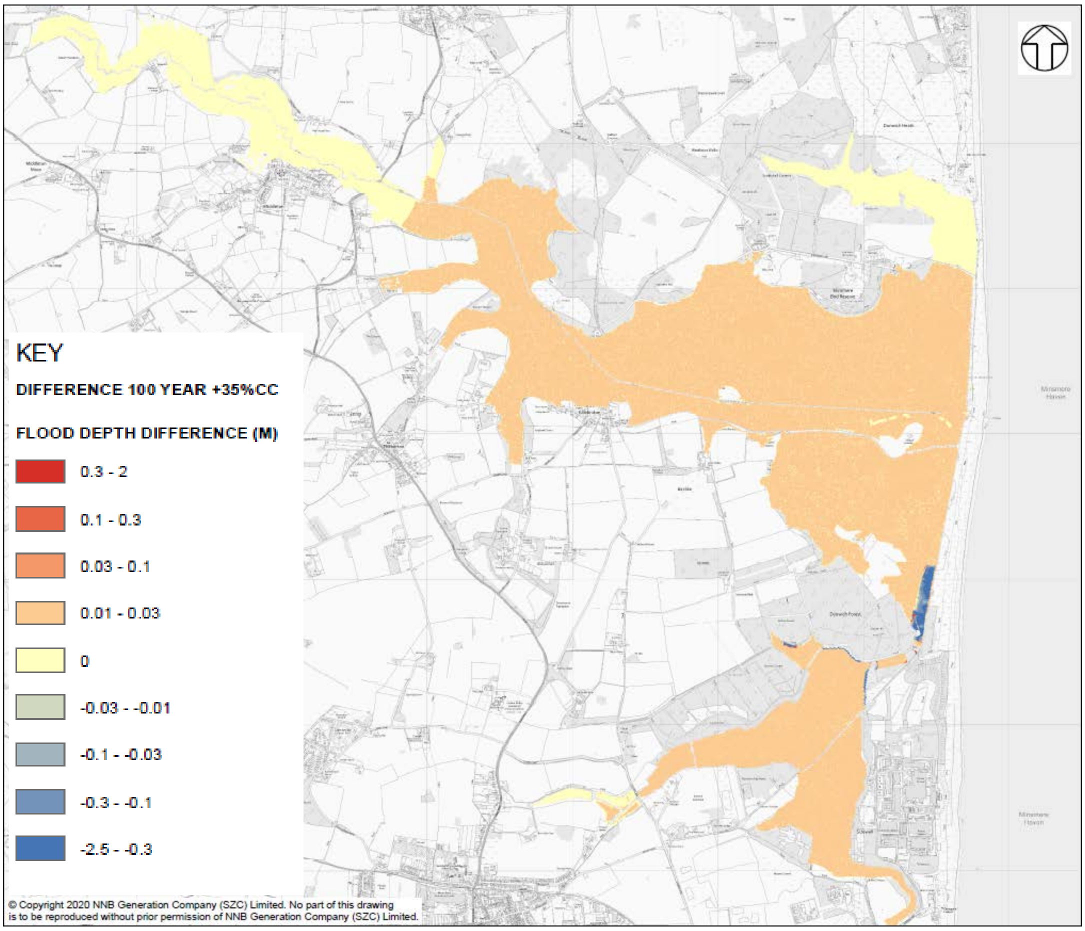


Plate 6.15: Difference in maximum flood depth for the 100-year return period with 65% climate change allowance

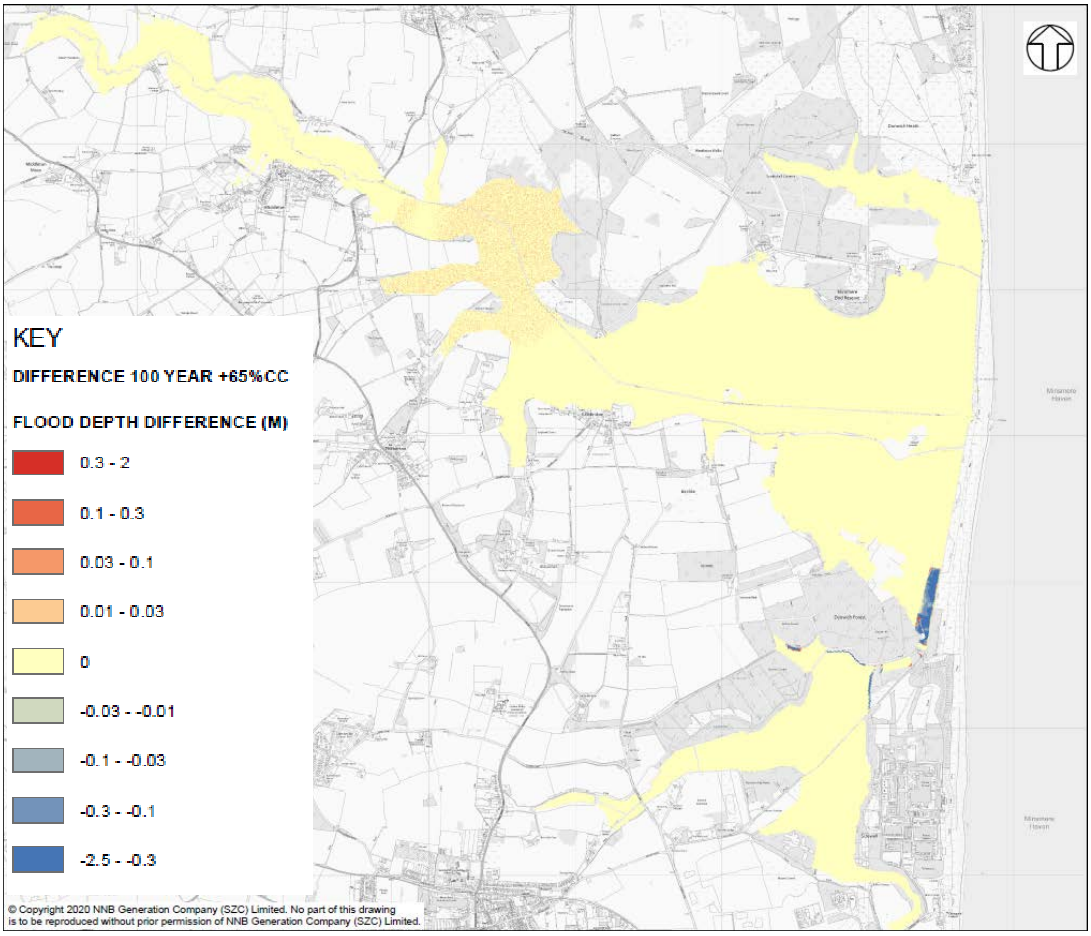
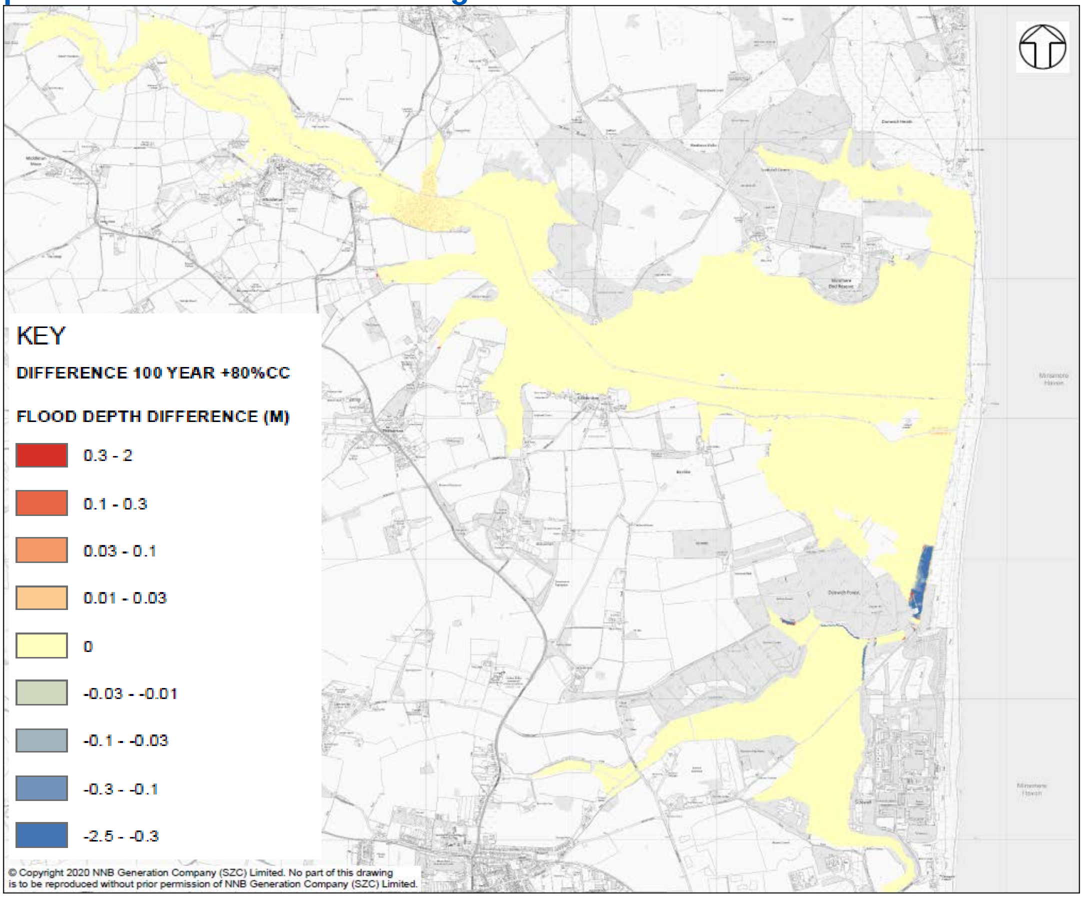


Plate 6.16: Difference in maximum flood depth for the 100-year return period with 80% climate change allowance



6.4.11. The difference plots above show that the change in flood depth is less than 30mm within Minsmere Levels for up to 1 in 100 year with 80%CC. Whereas in Sizewell Belts and the upstream parts of River Minsmere and Scott's Hall Drain catchments, the change is less than 10mm.

6.4.12. The 'high change' area on the left bank of the Leiston Drain immediately downstream of the SSSI crossing is caused by differences in representation of extended cross-sections between the 1D and 2D model and does not represent a true change in flood risk in that area. Similarly, the narrow strips of change long the north-west corner of the platform and along the earth bund to the west are caused by grid cell size and alignment of the features and do not affect overall model results.

6.4.13. Additional set of figures illustrating the difference in flood depths between the 'with scheme' and baseline scenarios for other return period events, namely 1 in 5-year, 1 in 20-year, 1 in 100-year and 1 in 1,000-year with climate change allowances of 25%, 35%, 65% and 80%, are provided in **Appendix E**.

6.4.14. **Plate 6.17** and **Plate 6.18** present difference in flood velocity maps, 'with scheme' minus baseline, for the 1 in 100-year return period event with 35% and 65% climate change allowance respectively.

Plate 6.17: Difference in velocity for the 100-year return period event with 35% climate change allowance

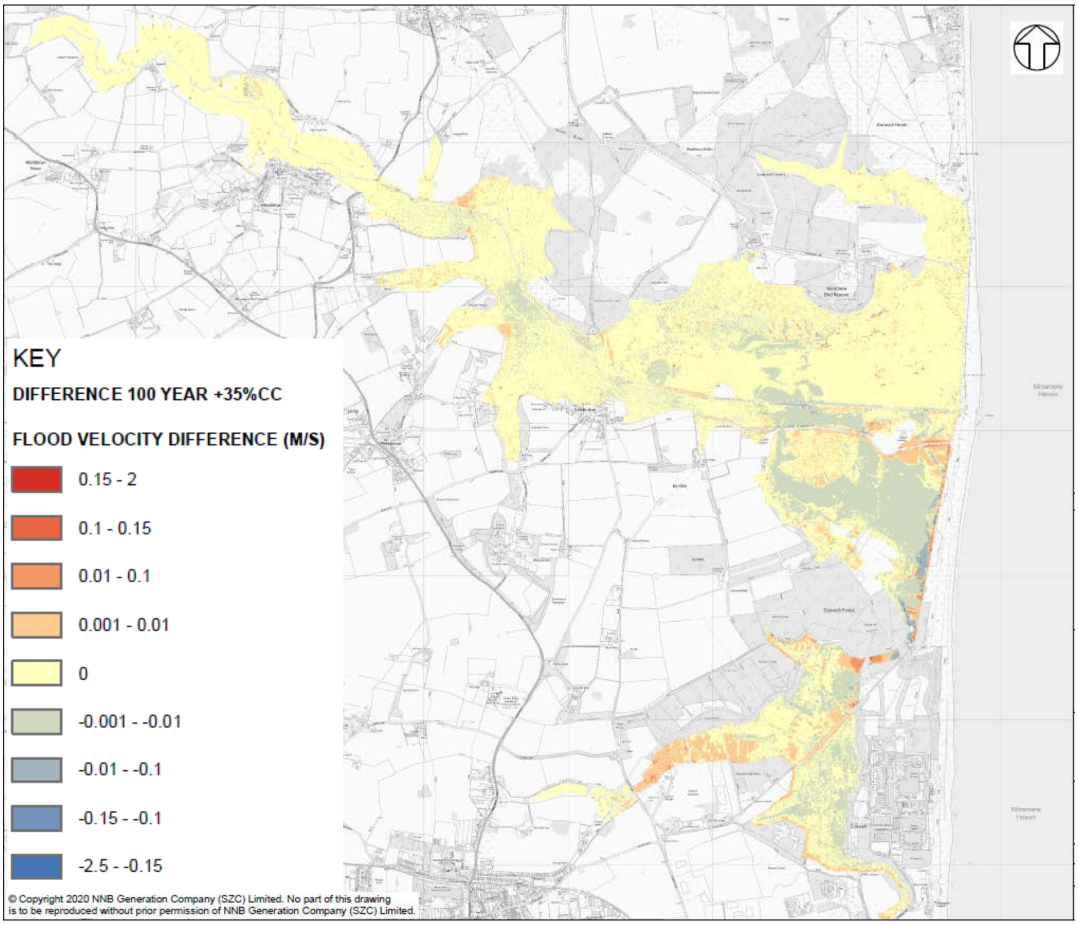
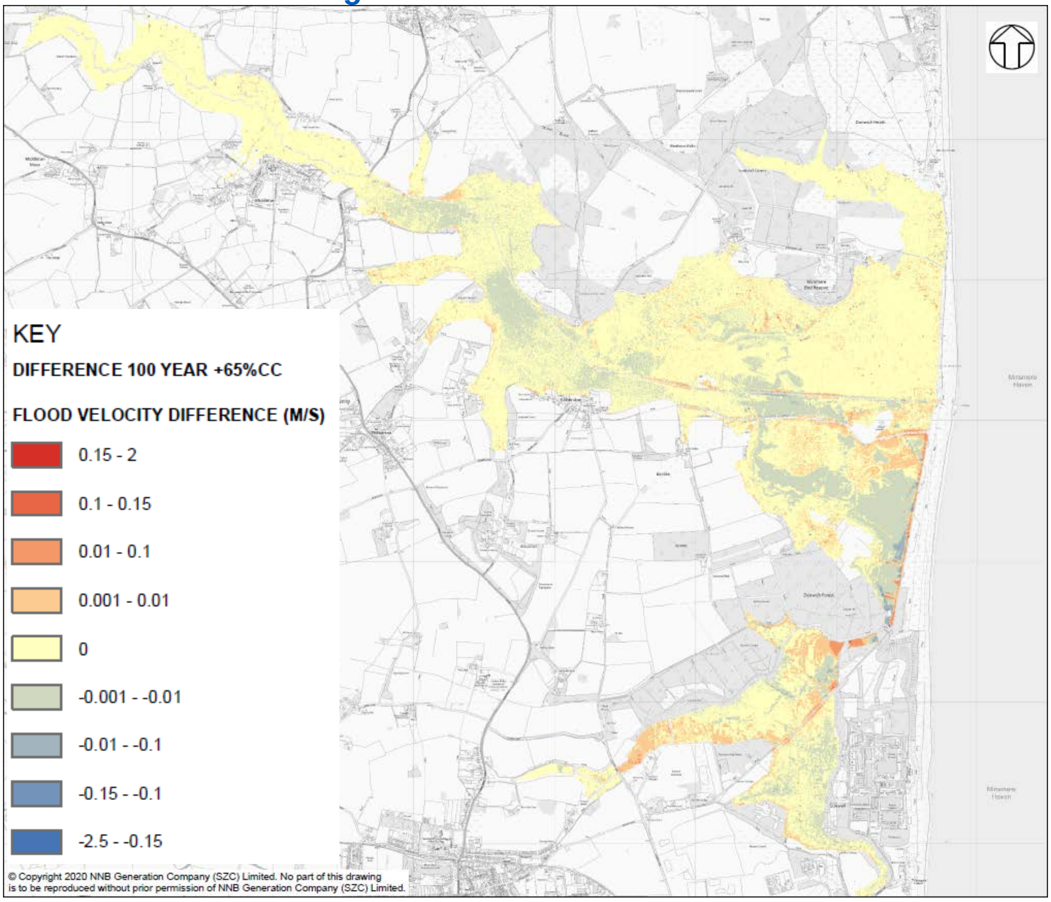


Plate 6.18: Difference in velocity for the 100-year return period event with 65% climate change allowance



6.4.15. Difference plots shown in **Plate 6.17** and **Plate 6.18,** indicate the difference in flood velocity between the 'with scheme' and baseline scenarios is less than 0.1m/s.

6.4.16. Based on the presented difference plots for flood depth and velocity, it was anticipated that there would be limited difference in flood hazard rating. The change in the hazard rating class between the 'with scheme' and baseline scenarios in **Plate 6.19** and **Plate 6.20** for the 1 in 100-year return period event with 35% and 65% climate change allowance respectively.

6.4.17. **Plate 6.19** shows that for the 1 in 100-year with 35% climate change allowance, there is only few small areas within the Minsmere Levels where flood hazard rating goes up by one class. Whereas **Plate 6.20** for the 1 in 100-year with 65% climate change allowance shows that more areas within the Minsmere Levels and upstream part of the catchment had changed hazard rating. The baseline model results indicate most of those areas are within the 'Low Hazard' rating class and would increase to 'Danger for Some'.

6.4.18. This low hazard is confirmed with the change in hazard rating assessed for the residential and non-residential properties that is presented in **Table 6.5** for the 1 in 100-year event with 35% and 65% climate change allowance. **Table 6.5** also presents the difference in maximum flood depth and velocity. Comparison for all considered return period events and climate change allowances is provided in a summary tables in spreadsheet in **Appendix E**.

6.4.19. The comparison in **Appendix E shows that** the maximum change in flood depth for all affected residential properties is less than 15mm. The total number of residential properties at flood risk has not changed due to the proposed Sizewell C development, all these are located within Leiston area.

6.4.20. The flood hazard rating has increased for four residential properties, with one for the 1 in 100-year return period event with 35%CC changing from ‘Danger for some’ to ‘Danger for most’.

6.4.21. The total number of non-residential properties at flood risk has increased by 5 for up to 1 in 1,000-year event with 35%CC and 6 for the two most extreme events. However, the flood depth for those additional properties is less than 5mm (8mm for the most extreme event) with close to zero velocity and therefore very low hazard.

6.4.22. Overall, for non-residential properties the maximum change in flood levels is 15mm with almost no change in velocity. Hazard rating class has increased from ‘Danger for most’ to ‘Danger for all’ for one non-residential property (post code IP16 4SP) for the 1 in 100-year event with 80%CC.

Plate 6.19: Difference in flood hazard rating for the 100-year return period event with 35% climate change allowance

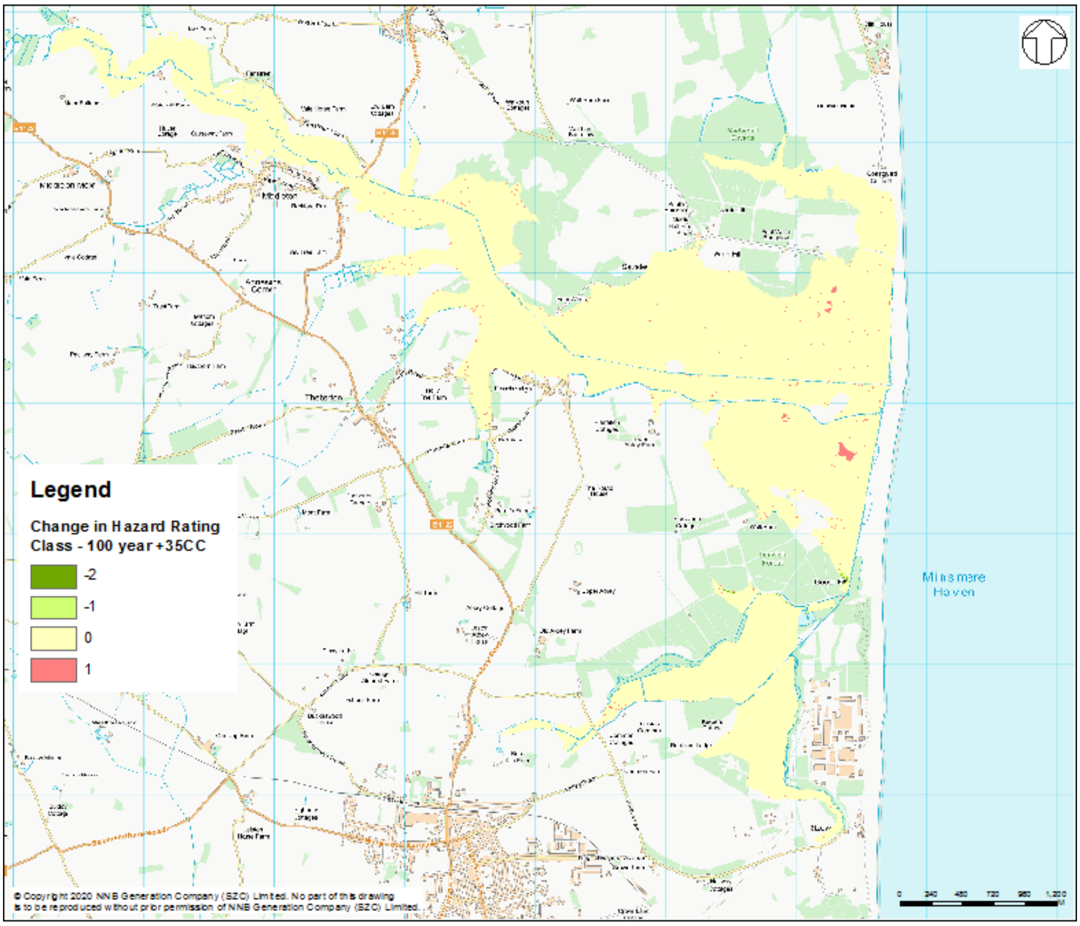
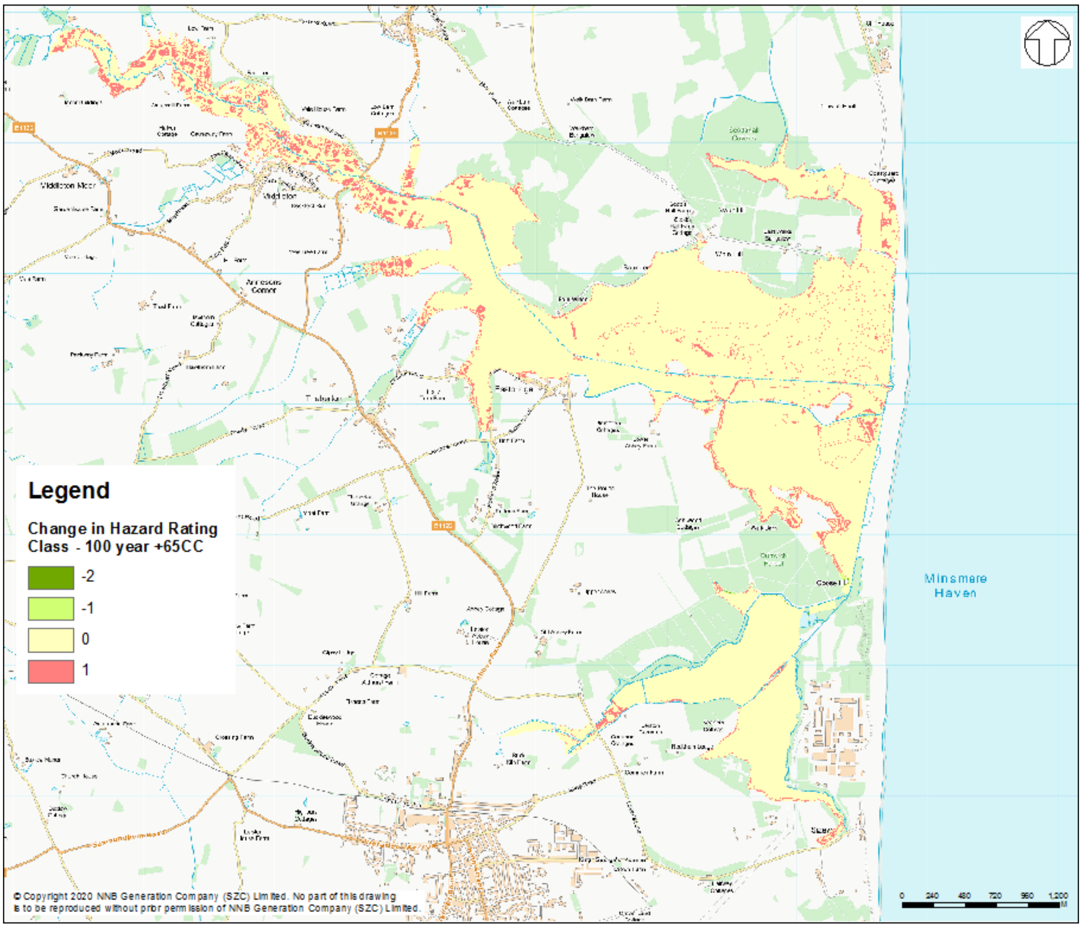


Plate 6.20: Difference in flood hazard rating for the 100-year return period event with 65% climate change allowance



6.4.23. **Plate 6.21** and **Plate 6.22** present difference in flood extent between the baseline and with scheme scenarios for the 1 in 100-year and 1 in 1,000-year events with 35% climate change allowance. There is very little difference in flood extent, mostly in the low-lying area within Minsmere Levels.

Plate 6.21. Difference in flood extent for the 100-year return period event with 35% climate change allowance

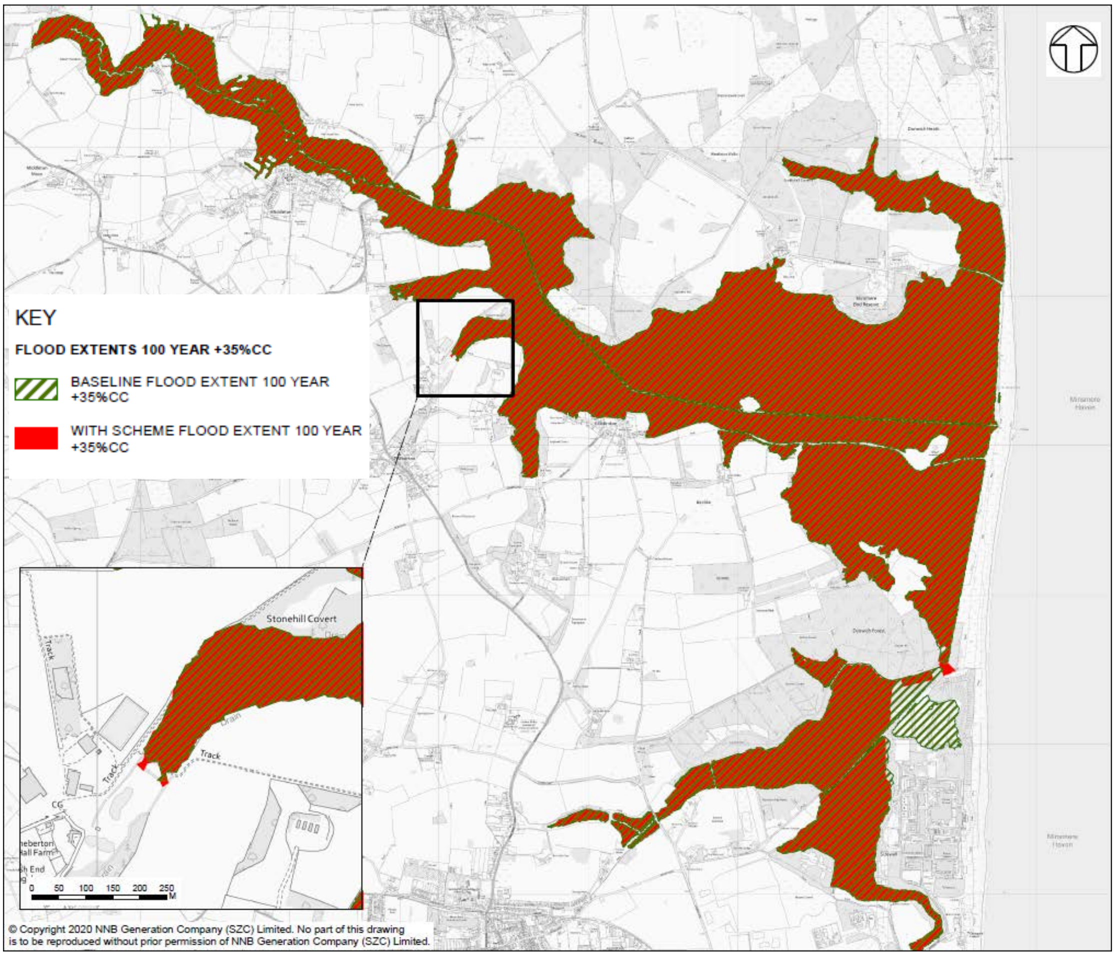


Plate 6.22. Difference in flood extent for the 1,000-year return period event with 35% climate change allowance

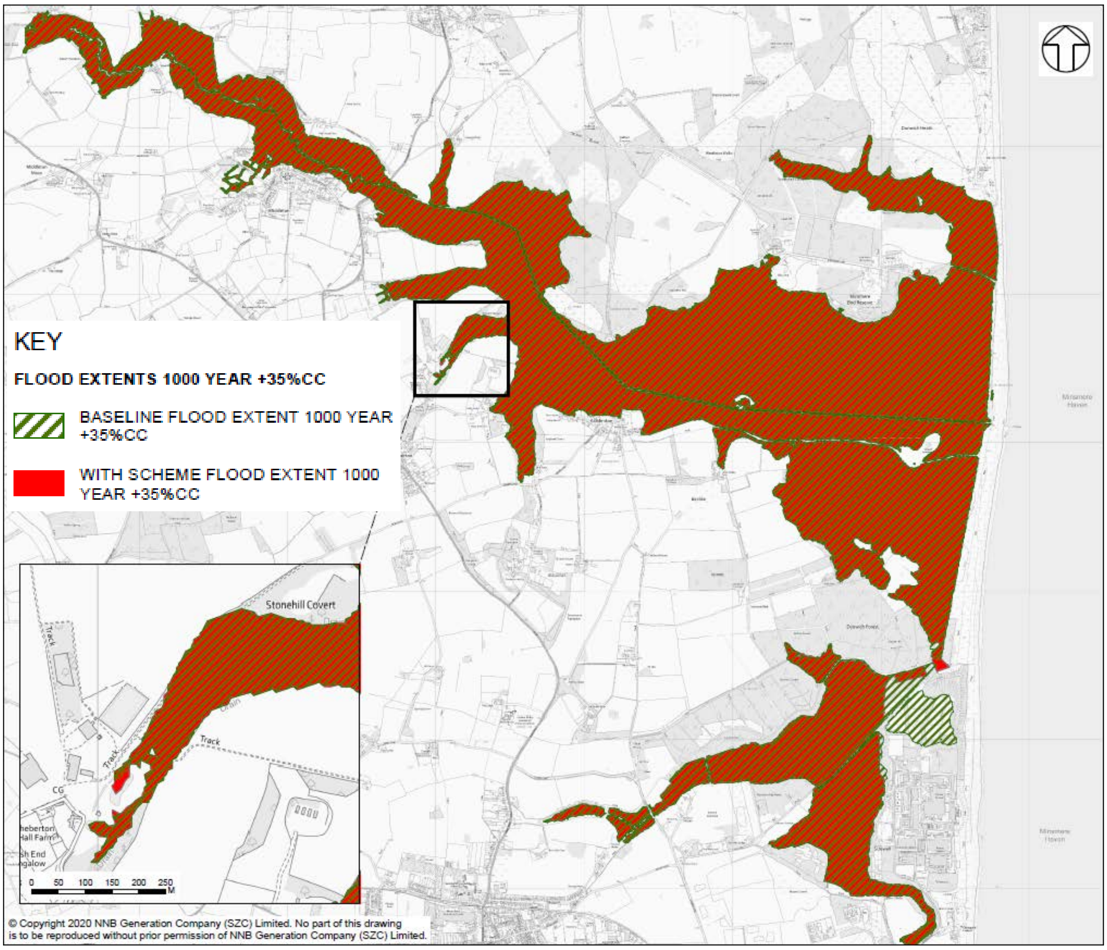


Table 6.5. Difference in flood depth, velocity and hazard rating for the affected residential and non-residential properties as a results of the proposed Sizewell C development for the 1 in 100-year event with 35% and 65% climate change

Property Type	Object ID	MCM Code	Post Code	Difference in Max Flood Depth (m)		Difference in Max Flood Velocity (m/s)		Difference in Flood Hazard Rating	
				100yr + 35%CC	100yr + 65%CC	100yr + 35%CC	100yr + 65%CC	100yr + 35%CC	100yr + 65%CC
Residential	2213858	1	IP16 4UJ	0.010	0.004	0.00	0.00	1	0
	7071690	1	IP16 4SQ	0.000	0.000	0.00	0.00	0	0
	7071691	1	IP16 4SG	0.000	0.000	0.00	0.00	0	0
	7071704	1	IP16 4SG	0.011	0.008	0.00	0.00	0	0
	7072129	1	IP16 4SG	0.000	0.009	0.00	0.00	0	0
	7072172	1	IP16 4SG	0.011	0.008	0.00	0.00	0	0
	7072173	1	IP16 4SQ	0.000	0.000	0.00	0.00	0	0
	7072175	1	IP16 4SG	0.010	0.009	0.00	0.00	0	0
	7076694	1	IP16 4UJ	0.010	0.005	0.00	0.00	0	1
Non-Residential	11947004	840	IP16 4SP	0.011	0.007	0.00	0.00	0	0
	11947019	840	IP16 4SL	0.010	0.008	0.00	0.00	0	0
	27798654	999	IP16 4UR	0.001	0.001	0.00	0.00	0	0
	28448626	999	IP17 3NR	0.000	0.000	0.00	0.00	0	0
	28449238	999	IP16 4UR	0.001	0.002	0.00	0.00	0	0
	28449806	999	IP16 4UR	0.001	0.002	0.00	0.00	0	0
	28732481	999	IP16 4SG	0.010	0.008	0.00	0.00	0	0
	28732933	999	IP16 4SP	0.000	0.000	0.00	0.00	0	0
	28732935	999	IP16 4SP	0.000	0.000	0.00	0.00	0	0
	28732936	999	IP16 4SP	0.000	0.000	0.00	0.00	0	0
	28732953	999	IP16 4SL	0.000	0.005	0.00	0.00	0	0
	28732954	999	IP16 4SL	0.010	0.008	0.00	0.00	0	0
	28733726	999	IP16 4UR	0.002	0.002	0.00	0.00	0	0
	28733727	999	IP16 4UR	0.002	0.002	0.00	0.00	0	0

Property Type	Object ID	MCM Code	Post Code	Difference in Max Flood Depth (m)		Difference in Max Flood Velocity (m/s)		Difference in Flood Hazard Rating	
				100yr + 35%CC	100yr + 65%CC	100yr + 35%CC	100yr + 65%CC	100yr + 35%CC	100yr + 65%CC
	29832142	999	IP17 3BY	0.000	0.000	0.00	0.00	0	0
	29832146	999	IP17 3BY	0.000	0.000	0.00	0.00	0	0

6.5 Sensitivity analysis

6.5.1. Sensitivity analysis is important for understanding modelling uncertainty and confirming the reliability of model results. Sensitivity tests to Manning's roughness coefficient, southern penstock opening, and loss coefficient have been undertaken as part of the Sizewell C modelling assessment.

a) Sensitivity to roughness

6.5.2. Simulations were undertaken where the channel and floodplain roughness values in the 1D model and 2D model were uniformly decreased and increased by 20% for the present-day 1 in 100-year return period event.

6.5.3. This sensitivity analysis could be used to assess the impacts of channel maintenance and seasonal vegetation variation in addition to understanding modelling parameterization uncertainty.

6.5.4. The difference in water levels between the baseline and the increase in roughness scenario shows an increase in peak flood levels of up to 0.1m in the upstream part of the Minsmere catchment (**Plate 6.23**). However, no additional properties were found to become at risk of flooding compared to the baseline scenario.

6.5.5. The results for the 20% decrease in roughness scenario show a maximum decrease in peak food level by approximately 0.03m in the Minsmere Levels and more in the upstream, steeper part of the catchment (**Plate 6.24**) in comparison to the baseline scenario.

6.5.6. Overall, the results suggest that the catchment is relatively sensitive to changes in roughness. Therefore, seasonal variations in flood levels could be expected.

Plate 6.23: Sensitivity analysis – roughness increased by 20%

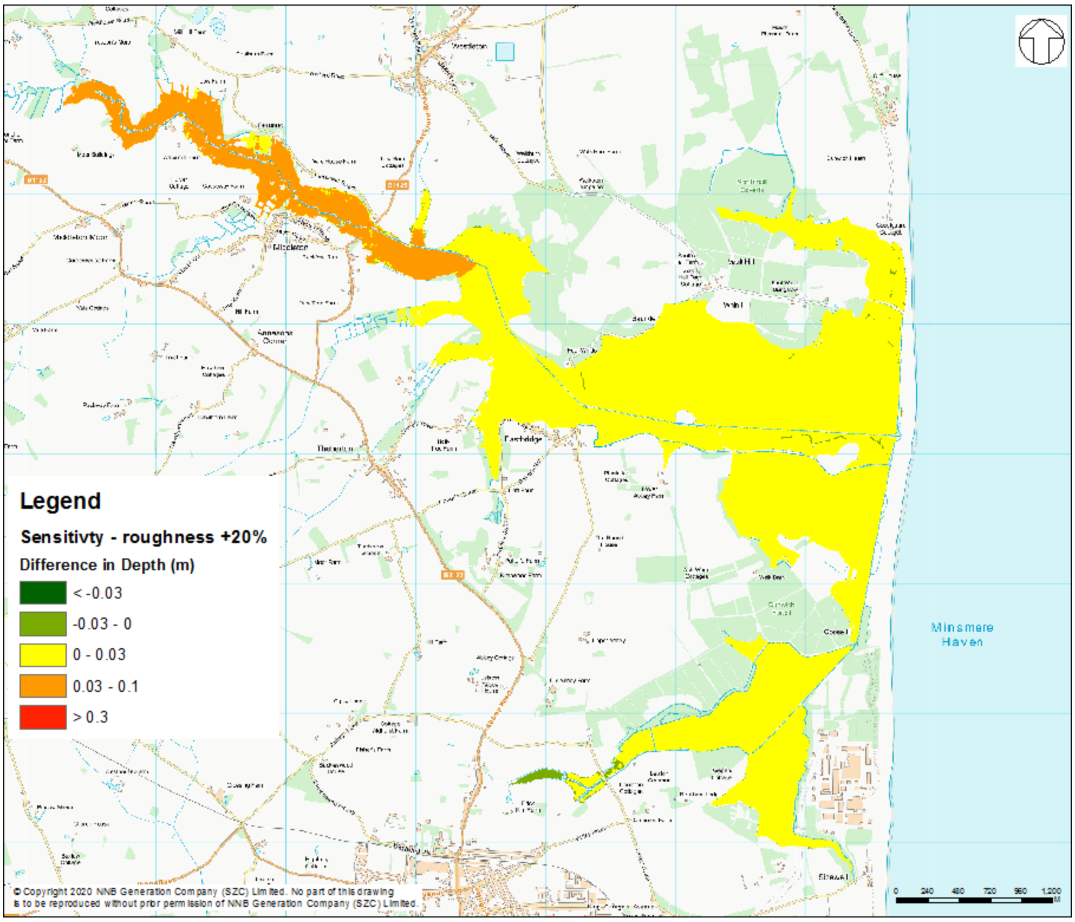
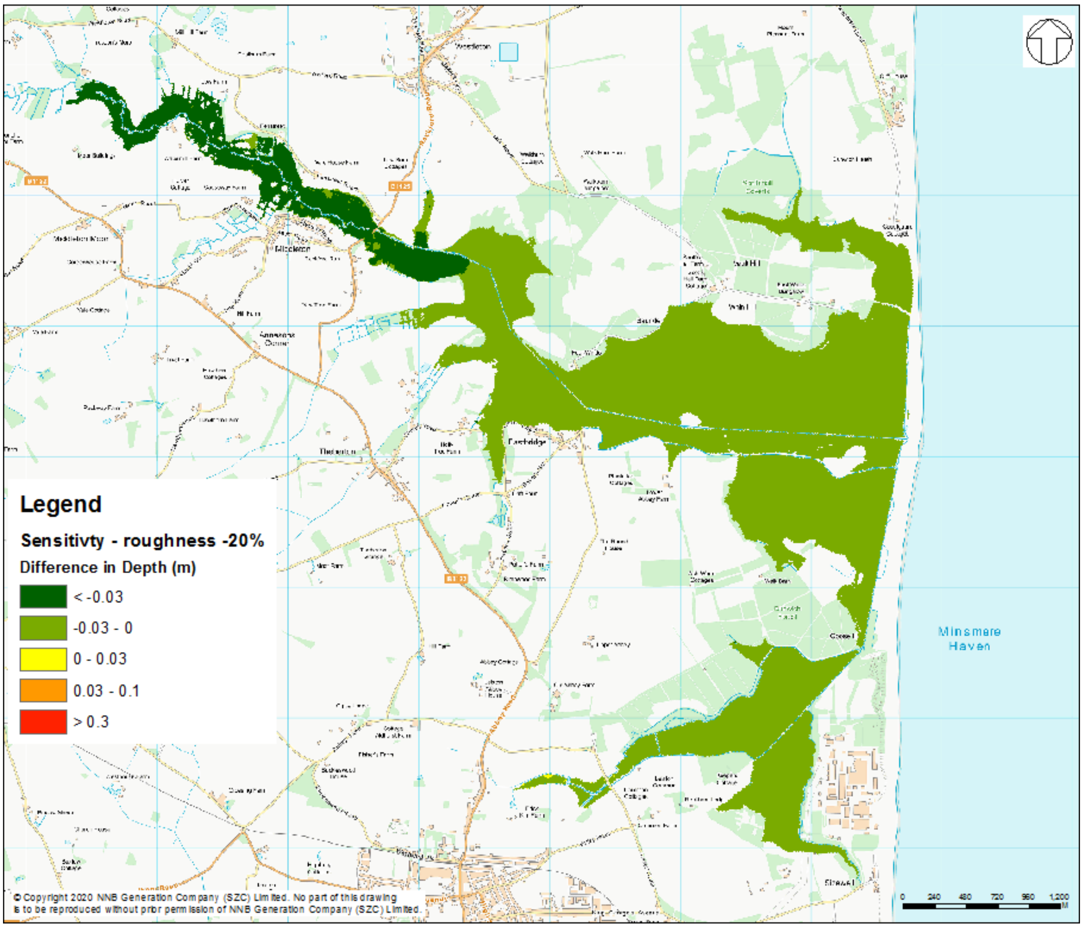


Plate 6.24: Sensitivity analysis – roughness decreased by 20%



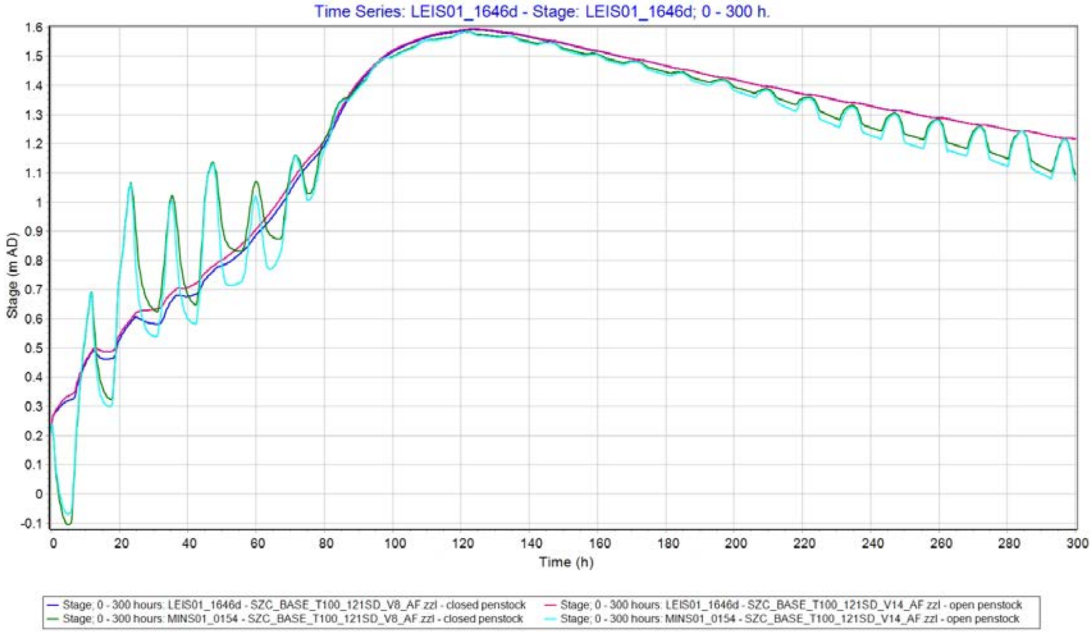
b) Sensitivity to penstock opening

6.5.7. As mentioned in paragraphs 3.1.2 and 3.1.3, the Minsmere South penstock was assumed to be closed for all modelled scenarios. The penstock was represented in the model using a flapped orifice (MINS_0154oCu) with the bore area set to the minimum of 0.001 m^2 to represent a closed position.

6.5.8. To assess impact of the closed position, a sensitivity to the penstock opening was undertaken by running the model for the present-day 1 in 100-year return period event with bore area of penstock adjusted back to its assumed size of 1m^2.

6.5.9. A timeseries plot showing the comparison of the peak water levels immediately upstream of the penstock (MINS01_0154) and downstream of the proposed SSSI crossing (LEIS01_1646d) for the closed and open penstock scenarios for the present-day 1 in 100 year event is presented in **Plate 6.25**.

Plate 6.25: Timeseries plot showing peak water levels for open and closed penstock scenarios for present-day 1 in 100-year return period



6.5.10. The model results show that by opening the penstock the peak flood level increases by a maximum of 4mm downstream of the SSSI crossing and decreases slightly by 1mm immediately upstream of the penstock. The results show that no additional properties were shown to become at risk of flooding compared to the scenario with closed penstock.

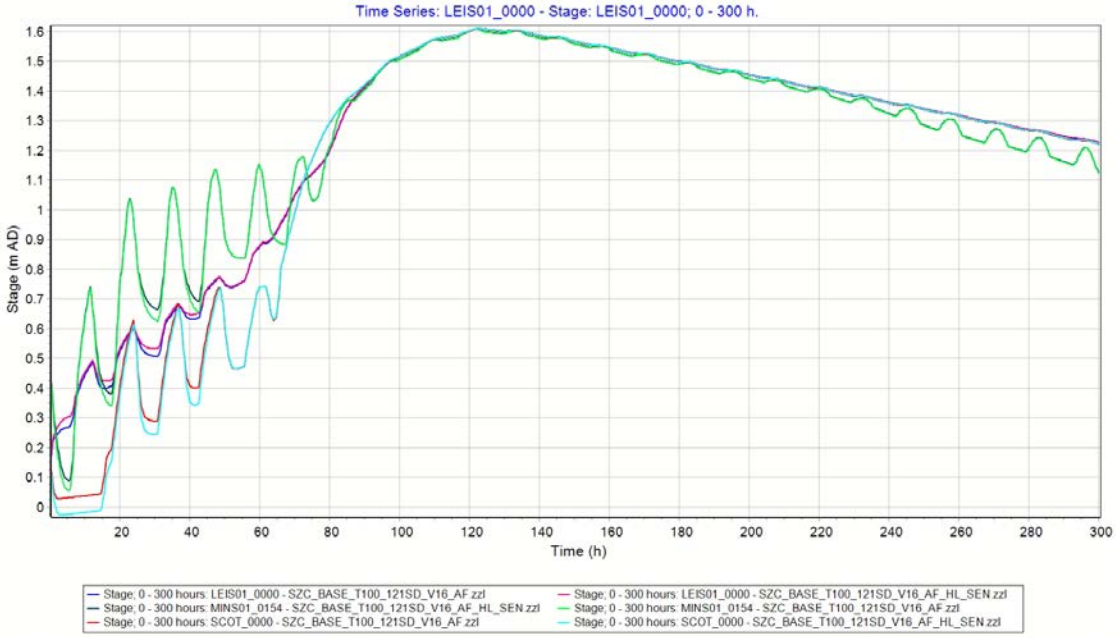
6.5.11. The overall results from the penstock opening test show low sensitivity of the model to the opening. Therefore, considering lack of details on the operation of the penstock, the closed position of the penstock was assumed appropriate for the Sizewell C study.

c) Sensitivity to loss coefficient

6.5.12. A sensitivity to loss coefficient for the Leiston Drain, the Minsmere New Cut and the Scott's Hall Drain outfalls were assessed with the coefficient decreased by 20%. The model was run for the present-day 1 in 100-year return period event.

6.5.13. The results show that decreasing the discharge coefficient by 20% has no significant impact on peak water levels, as shown in **Plate 6.26**.

Plate 6.26: Long section plot showing peak water levels for the baseline and loss coefficient decreased by 20% scenarios for 1 in 100-year event



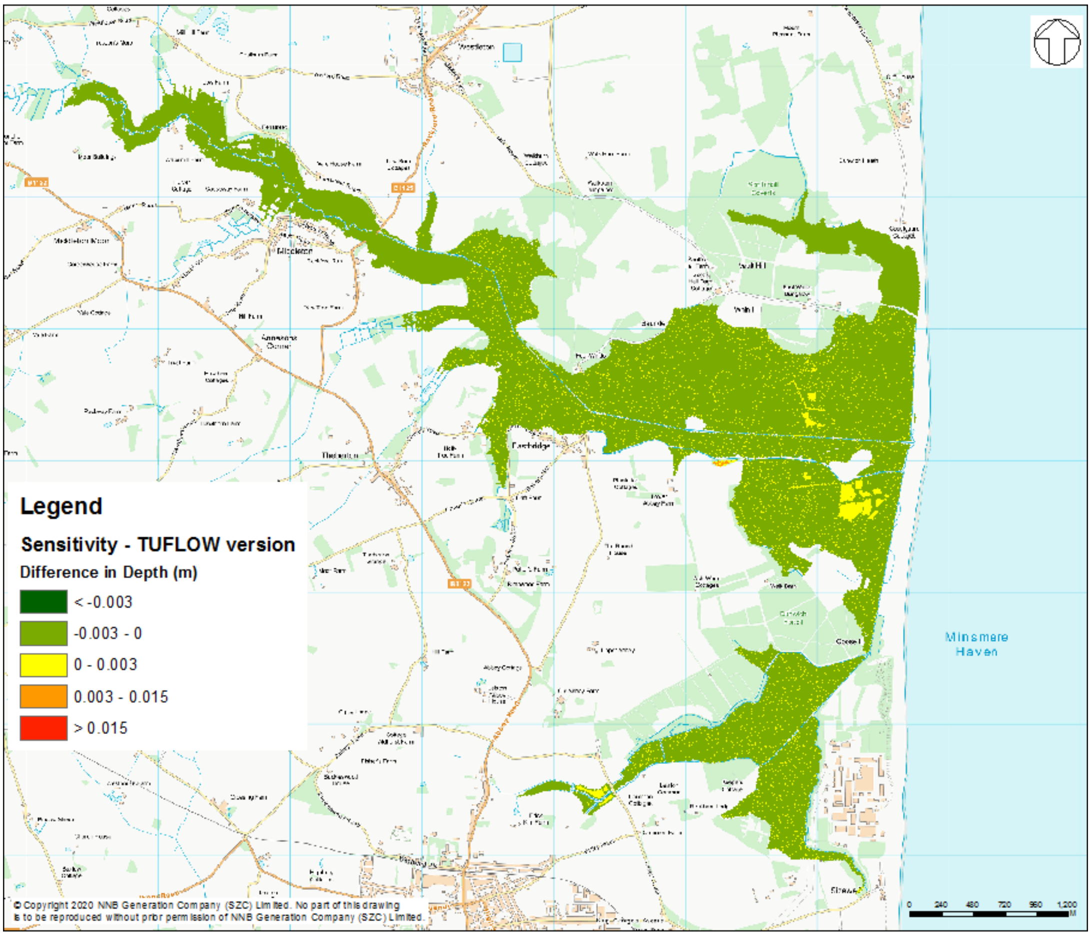
d) Sensitivity to software version

6.5.14. At the time of the model build, the 2017 version of the TUFLOW software was the most recent. Further into the assessment a new version of the software was released (TUFLOW 2018-03-AE-iDP-w64). Therefore, a sensitivity test was undertaken to assess potential changes to model outputs as a result of using the latest version of the model.

6.5.15. The model results for the present-day 1 in 100-year return period event show that running the model in latest version of TUFLOW has no significant impact in the maximum flood extent or depths (change within 3mm), (**Plate 6.27**).

6.5.16. Furthermore, no significant differences in model run-times or stability were identified between simulation with the two software versions. Therefore, to ensure efficiency, the previous version of the software was used for the final design model runs.

Plate 6.27: Difference in flood depths between the two TUFLOW software versions for the present-day 1 in 100-year return period event

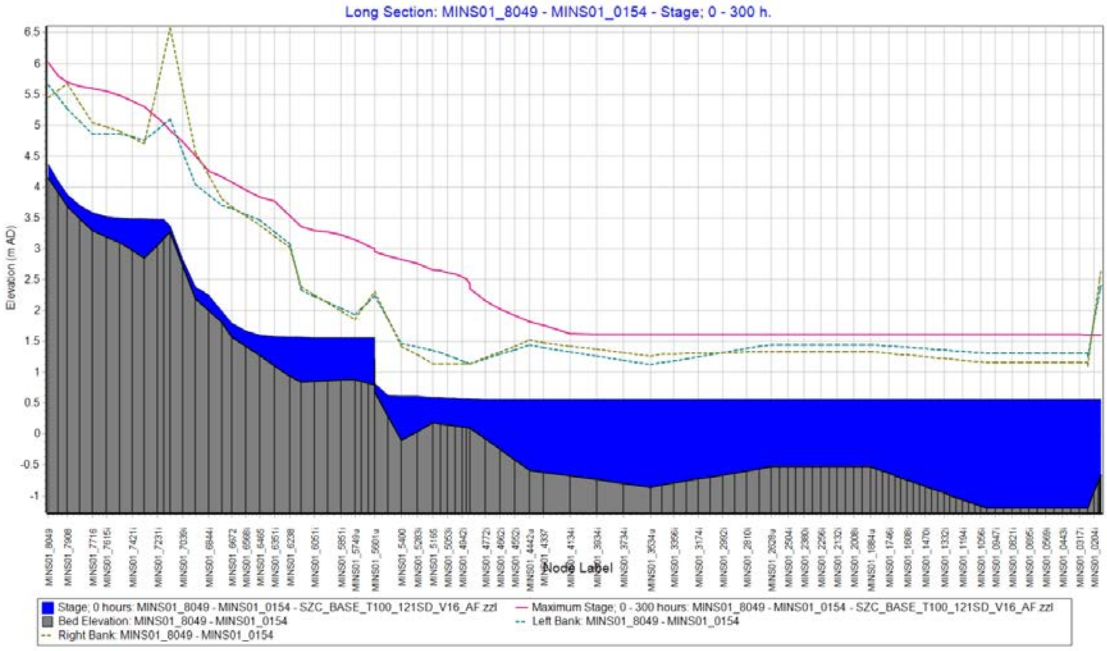


e) Initial conditions check

6.5.17. Additional checks have been conducted to confirm that the initial conditions adopted in the model are below bank levels and that peak water levels are significantly higher than initial conditions. The purpose of the check was to confirm the initial condition were appropriate and the peak water levels were not affected by the choice of initial conditions.

6.5.18. The comparison of results for the 1 in 100-year return period for a long section at Minsmere New Cut between the initial water levels (Red line) and the peak water levels (Blue line) are shown in **Plate 6.28**. The plot shows the initial water level is lower than the peak water level and bank levels. Therefore, it was concluded the initial conditions adopted for the study were appropriate.

Plate 6.28: Long section plot showing the initial water level and peak water levels for 1 in 100-year return period at Minsmere New Cut



7 LIMITATIONS

7.1.1. A 1D-2D Flood Modeller–TUFLOW approach has been used. The model achieves satisfactory numerical convergence and results are consequently considered reliable. However, some numerical non-convergence has arisen during model development. This has been addressed by the adoption of maximum iteration number to 17 and top slot height to 7. These model parameter values are within acceptable ranges.

7.1.2. The Sizewell C model (Flood Modeller–TUFLOW) was adopted from previous Environment Agency model (Ref 1). Original survey drawings used for the 2013 model build were not available to check the cross-sections and structures geometries and levels.

7.1.3. In the current model schematisation, the penstock at the Minsmere South Culvert is assumed to be closed. That assumption was based on information provided by the Environment Agency stating that '*this penstock is maintained in a closed position but can be operated in times of extreme flows*.' No new information has been made available to confirm the penstock opening. Therefore, the closed position was adopted for all model runs.

7.1.4. Bank top levels were derived from the topographic cross-section data. This approach was preferred to the alternative of using LiDAR to define the linkage between the 1D and 2D model domains.

7.1.5. In addition, the use of cross-section bank top ensures consistency in the 1D and 2D model results particularly regarding the onset of flooding. A disadvantage of using cross-section data is that a steady gradient is assumed in the bank top spill between cross-sections, such as localised high and low spots are missed in open channel rural reaches.

8 CONCLUSIONS AND RECOMMENDATION

8.1.1. The 1D-2D (Flood Modeller–TUFLOW) hydraulic model of the River Minsmere, Leiston Drain and Scott's Hall Drain developed for the Sizewell C study has been further improved using the latest available data and by making additional changes to model schematisation and parameters.

8.1.2. The Aldhurst Farm scheme has been included in the improved baseline model. The 'with scheme' model schematisation was based on the latest, at the time of the modelling, available Sizewell C development design drawings and the components of the development within the flood extents. Therefore, they could be potentially be at risk of flooding or at changing risk of flooding to off-site receptors.

8.1.3. The derived fluvial inflows have been routed through the hydraulic model. The critical storm duration of 121 hours was determined and adopted for the remaining analysis.

8.1.4. The model was simulated for a range of return period events; the 1 in 5 year, 1 in 20-year, 1 in 100-year and 1 in 1,000-yearevents, with and without climate change allowances (25%, 35%, 65% and 80%).

8.1.5. Following stability checks, the model was found to be numerically stable with limited non-convergence.

8.1.6. The baseline model results show that only a limited area of the proposed Sizewell C development is within flood extents with 11 properties at risk for the 1 in 100-year return period event with 35% climate change allowance (considered as the higher central allowance up to the end of theoretical maximum site lifetime).

8.1.7. The 'with scheme' model results show that for a range of considered return period events and climate change allowances, the maximum increase in flood levels across the Minsmere catchment is 15mm, with no significant change to flood velocity or hazard rating.

8.1.8. The highest (15mm) difference in flood levels was found for the 1 in 100-year return period event with 25% climate change allowance. For lower probability events and higher climate change allowances (including the credible maximum H++ scenario) the change in flood levels was less than 15mm due to Minsmere Levels and Sizewell Belts being inundated to higher level for both baseline and 'with scheme' scenarios making the relative difference less.

8.1.9. The total number of residential properties at risk of flooding has not changed as a result of the Sizewell C development for all of the considered return period events and climate change scenarios. The maximum increase in flood level for the residential properties is 0.014m for the 1 in 100-year return period event with 25% climate change allowance. Flood hazard rating changed from class 'Danger for some' to 'Danger for most' for four properties located within Leiston area.

8.1.10. The total number of non-residential properties increased by 5 as a result of the development for most of the considered return period events and climate change scenarios, with 6 additional properties for the most extreme 1 in 1,000-year event with 65% and 80% climate change allowances. However, for all those additional properties, the increase in flood level is less that 10mm and no change in velocity. As the flood depth and velocity for those properties is very low, they are all classified as low hazard rating.

9 REFERENCES

Ref 1 Environment Agency, Flood study of River Minsmere and Leiston Drain, Suffolk, Final Report. JBA. January 2013

Ref 2 EDF Energy. SZC FRA – Hydrology Review and Design Event Methodology. Royal HaskoningDHV. October 2019

Ref 3 Royal HaskoningDHV. Aldhurst Farm Habitat Creation Scheme EIA Screening Report. October 2014

Ref 4 Environment Agency. Use of Joint Probability Methods in Flood Management: A Guide to Best Practice R&D Technical Report FD2308/TR2. March 2005

Ref 5 UK Hydrographic Office. Admiralty Tide Tables, NP 201A-17 Tide Tables UK & Ireland Vol 1A. 2017

Ref 6 Environment Agency. Coastal Flood Boundary Conditions for UK: Update 2018 SC060064/TR6: Technical Summary Report, May 2019

Ref 7 Enivronment Agency. Coastal flood boundary conditions for UK mainland and islands, Project: SC060064/TR4: Practical guidance design sea levels. February 2011

Ref 8 Environment Agency – Flood Risk Assessments: Climate Change Allowances, February 2016 (updated February 2019): https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances#history

Ref 9 Environment Agency. Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities. April 2016

Ref 10 Met Office. UK Climate Projection 2018. November 2018. https://www.metoffice.gov.uk/research/approach/collaboration/ukcp/index

Ref 11 Ministry of Housing, Communities and Local Government. National Planning Policy Framework. London: The Stationery Office, February 2019.

Ref 12 Office for Nuclear Regulation and Environment Agency. Use of UK Climate Projections 2018 (UKCP18) by GB Nuclear Industry. March 2019

Ref 13 Environment [redacted] UKCP18 Report (AE/2015/119103/02) addressed to [redacted] On behalf of Sizewell C Co). 20 May 2019

Ref 14 EDF Energy. UK Climate Change Projections 2018 - Review and Proposed Response, Revision 2. Royal HaskoningDHV. October 2019

Ref 15 Environment Agency – Flood Risk Assessments: Climate Change Allowances, December 2019: https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

APPENDIX A: MINSMERE RIVER AND SIZEWELL BELTS MODEL SCHEMATISATION UPDATE REPORT

RHDHV, August 2015

Note

HASKONINGDHV UK LIMITED
RIVERS, DELTAS & COASTS

To	:	[redacted]
From	:	Royal HaskoningDHV
Date	:	28th August 2015
Copy	:	
Our reference	:	PB1452/Baseline-Calibration_V004/301907/Pbor
Subject	**:**	**River Minsmere and Sizewell Belts Model Schematisation Update August 2015**

Purpose

This technical note provides an update to the schematisation of the hydraulic model of the River Minsmere and Leiston Drain last presented in January 2015. The model will be utilised to determine fluvial and coastal breach flood risk to Sizewell C (SZC), as well as potential impacts of the development on off-site receptors, as part of the Flood Risk Assessment (FRA) that is being prepared for the proposed new nuclear power station.

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1 Background

1.1 Purpose of this Technical Note

This technical note discusses the principles and approach used in constructing the baseline and calibration hydraulic model of the River Minsmere catchment (which includes the Leiston, Sizewell and Scott's Hall Drains and IDB Drain #7). The purpose of the note is to detail the methods and data that have been adopted.

1.2 Previous Discussions

A Technical Note[1] and presentation regarding the outlined model schematisation was previously produced and discussed with a number of parties at a meeting held in Ipswich on 30th January 2015.

At the meeting held on 30th January 2015, a number of comments and actions were raised, some of which impact the schematisation of the model. The actions (for all parties) salient to the schematisation/development of the model are listed in Table 1.

Table 1 - Actions Arising from 30th January Meeting Specific to the Model Schematisation

Action Ref.	Description	Originator	Actionee (Completion)	Relevance to Modelling Schematisation
7	EA to provide copy of Kemp report on 1993 flooding.	RB	ND (6/3/2015)	Report will help the understanding of flood mechanisms (hence features to be included in the model schematisation) and calibration of the model
10	EA to provide copy of BBV report on Minsmere Sluice.	RB	ND (12/3/2015)	Report will help the understanding of flood mechanisms (hence features to be included in the model schematisation) and calibration of the model
11	RHDHV to consider comments raised on modelling approach, including not double counting storage, ensuring z-lines are correct, etc., as model is developed.	AD	LG (3/2/2015 - ongoing)	Proposed schematisation may change to address comments during model construction.
13	EA to confirm matters relating to Minsmere Sluice including operation of New Cut penstock, levels of weir and Scotts Hall Drain, and operational experience of blockages.	Various	ND (12/3/2015)	Data is required to set specific structure geometries and determine operational procedures (and sensitivity tests) for calibration/baseline/construction/development scenarios

Of the above, Action 11 has been addressed (and is discussed herewith) while the data received from the other actions has been utilised in determining the model schematisation.

[1] Minsmere and Sizewell Belts Modelling Schematisation V7, Royal HaskoningDHV, January 2015

1.3 Model Requirements

The completed modelling will provide an understanding of flood risk at specific stages of the site's development. For the purposes of the Flood Risk Assessment (FRA), the hydraulic model will be required to represent a range of return periods (1 in 20 (functional floodplain), 100, 200 and 1,000 years both with and without an allowance for climate change) and storm durations (to be determined) for all of the scenarios to be considered. The scenarios that will be assessed include the following geometries:

- Baseline (pre-development) geometry
- Construction phase geometry (number of phases to be confirmed)
- Operational phase geometry

The model is still in the calibration/baseline stage of construction but it will be used to determine flood risk for both fluvial and coastal (following a potential breach in the flood defences north or south of the power stations) inundation for the above scenarios; the locations of potential coastal breaches are currently being developed.

2 Data

Numerous data sets have been considered in the study; Table 2 summarises the main data sets that have been utilised in the development of the hydraulic model.

Table 2 - Main Data Sets Used in the Model Construction

Data Source	Application of Data
DTM and DSM LiDAR	Both filtered (DTM) and unfiltered (DSM) LiDAR has been utilised to generate the topography of the hydraulic model. The data has been utilised to generate the 2D domain topography.
Current Environment Agency hydraulic model and reporting	1D model data has been utilised to represent Minsmere New Cut, Leiston Drain, Sizewell Drain and Scott's Hall Drain including their associated structures. The model has been supplemented with additional survey items listed below.
Storm Geomatics survey data (December 2013 and May 2015).	Data used to supplement 1D and 2D constituents of the model.
Minsmere Sluice As Built Drawings (109417-xxxxAB)	As built drawings have been utilised to update the hydraulic model. The drawings have specifically been used to update geometry data of the various culverts into Minsmere Sluice.
KWT International Minsmere Phase 2	Scotts Hall Drain 'flap' geometry details to supplement As Built Drawings data
Coney Bank Drawings	Scheme design drawings have utilised to depict the geometry of the embankment, crest elevation and culvert through the structure

3 Proposed and Current Schematisation

3.1 Summary of Approaches

The schematisation of the current model has been derived through an understanding of the catchments (gained from previous studies and data including those received as actioned in Table

1), a number of site visits and liaison with EDF Energy, the RSPBthe Sussex Wildlife Trust and the East Suffolk IDB.

Table 3 provides a comparison of the proposed and the current hydraulic model schematisation (which provides background to how Action 11 from the meeting held on 30th January 2015 has been addressed). The changes and assumptions that have been made are discussed in more detail later in this Technical Note.

Table 3 – Summary of Proposed and Current Model Schematisation

No	Proposed Baseline Schematisation Representation	Changes/Assumptions
1	Extension of Leiston Drain upstream of Lovers Lane in order to ascertain flood risk to the floodplain and Anglian Water assets both pre and post development. A channel topographic survey (Storm Geomatics December 2013) has already been undertaken to provide this data.	No changes to proposed method.
2	Additional 1D channels within the Sizewell Belts to improve accuracy of flow paths and schematisation of the IDB drainage network and improve calibration between known and modelled flow routes to be constructed utilising Storm Geomatics data.	No changes to proposed method.
3	Improved schematisation from reservoir units to extended cross sections along the left hand bank of Leiston Drain upstream of the confluence with Sizewell Drain where the floodplain on one bank rises steeply (the current schematisation is simplified).	This has not changed in the majority of the reach however there is one area in which the floodplain has been modelled in 2D. This is located on the left hand bank of the Leiston Drain upstream of Dunwich Forest (Goose Hill).
4	Updated schematisation to link ISIS model channel sections with 2D model domain (TUFLOW) rather than 1D reservoir units (to improve reliability of the model results).	The concept has not changed however there have been changes to the extent of the 1D and 2D model and therefore the locations of the links have also changed.
5	Steep sided valley sides modelled in 1D to ensure narrow flow routes (1 or 2 cell width) are not encountered (ensuring enhanced model stability).	Not changed. The model has been tested to only model the 'in-bank' channel in 1D. However, this testing showed that this is unstable and therefore the previous stated schematisation has been deemed appropriate for use.
6	2D domain to include Coney Cross Bank and floodplain to the north previously not included in the 1D model (ensuring that the full catchment is modelled).	No changes to proposed method.
7	IDB Drain #7 (that flows in to the Leiston Drain south of Minsmere New Cut) modelled in 1D to better represent volumetric capacity of the system and better represent flow routes and capacity (not including these would make it hard to calibrate the 1D element of the model in the Sizewell Belts as the capacity would be under estimated and the model would over predict water levels for in bank flows).	No changes to proposed method. However, the availability of data is less than required due to access not being granted to survey the watercourse in its upper reach. Access to the land is not expected to be granted and therefore assumptions/simplifications are required.

No	Proposed Baseline Schematisation Representation	Changes/Assumptions
8	Field drains of similar width to the grid size (>8m are deemed similar) in the River Minsmere floodplain will be represented in the 2D model as zlines (lowered linear features in the model topography) in order to better simulate flood mechanisms and improve calibration. They will not be modelled as 1D elements as they are too numerous to model and would cause instabilities due to large difference in the volume of water being exchanged between them and the floodplain. Connections between these drains such as pipes and small culverts will be added to the model with assumed dimensions (following discussions with the IDB and RSPB) where survey data does not exist. It is intended that only those drains deemed significant in terms of drainage mechanisms (therefore possibly not all those shown) shown in the associated Figure will be represented in this way. It is not intended that field drains in the Sizewell Belts will be modelled in a similar manner. There are no connectivity issues (through embankments) in the Sizewell Belts, therefore the same approach is not necessary; there are a greater number of 1D elements in this area of the model (which gives an appropriate level of detail) and there are no physical barriers to flow between these 1D elements unlike in the Minsmere floodplain.	This concept has been utilised in the current model. However the number/extent of the drains is not as previously presented (they are less numerous).
9	In the low-lying 'flat' reaches of the floodplain, it is appropriate to add flow (from rainfall) to the floodplain rather than direct into the main channels. This flow will therefore be distributed along the zlines representing the field drains. This will ensure that water is attenuated in the drains and floodplain (as occurs in reality) and therefore the rate of rise in water levels within the 1D elements of the model will not be overestimated (which will in turn help calibration of the model).	This schematisation has been changed from point/distributed inflow along the zlines to direct rainfall over the low lying floodplain.
10	A 10m grid resolution will be utilised striking a balance between model run time and level of detail. The 2D model domain is predominantly rural floodplain; there is no need to model flow routes between buildings and other obstructions at a sub 10m resolution.	A number of tests have been performed during the model construction in order to test model stability, run times and variation in outputs. An 8m grid has been utilised which strikes a balance between these characteristics.
11	Schematisation upstream of East Bridge (River Minsmere) to remain unchanged in recognition of its remoteness from the development and to optimise model run times (less 2D domain and therefore faster run times).	The model 2D domain has been extended upstream of East Bridge to (and beyond) the extent of the 1D model. This addresses comments/concerns raised over the extent of the 2D model and potential instabilities at 1D/2D interfaces across the road at East Bridge.
12	Embankments crossing the floodplain that impound or attenuate flow will be included within the 2D model. These have been identified and included in a topographic survey brief assuming that access will be granted to the required areas (RSPB site). Structures through these embankments will be surveyed and modelled, however it is believed they will allow limited conveyance (these features will help to improve model validation as modelled flood mechanisms can be verified against RSPB or IDB accounts of typical flood routes and mechanisms).	Due to the bird breeding season, several of these embankments have not been surveyed. However these are due to be surveyed in autumn 2015. Currently the model utilises filtered LiDAR bank elevations. The embankments are different to the data used in the parallel Coastal studies as they are landward side of the coastal defences.
13	Minsmere sluice will be modelled in its present (refurbished) condition as per drawings of recent works carried out on the structure[2].	No changes to proposed method

[2] Minsmere Sluice and Embankment Works Various Drawings (019417_0001 to 0046) , Black and Veatch (2012 to 2013)

3.2 Baseline Schematisation Approach

As the model will be required to accurately simulate flood mechanisms that occur throughout the progression of the hydrograph, it has been necessary to strike a balance in the level of detail and the level of simplification to appropriately represent these mechanisms. The following provides a summary of the high level considerations and decisions made in the model schematisation/construction:

- Model grid extent. In order to limit potential risk of model instability at the interface of 1D and 2D across the road at Dam Bridge (and to address a potential issue raised at the previous Schematisation meeting) the model upstream of Dam Bridge has been changed from 1D only to a linked 1D-2D model.
- Model grid size. The 2D grid size utilised in the model will affect the schematisation of the model and the (perceived) ability to accurately model small scale drainage features. During the model build, an assessment was made as to the effect of the model grid size on model run times. Due to the rural nature of the catchment and lack of small scale flow routes and obstructions needing to be represented in the floodplain (unlike urban environments) a 10m grid size was previously proposed to be adequate to represent flood mechanisms in sufficient detail. Model grid sizes of 4, 5, 6, 8 and 10m have been tested in an early version of the model. The findings are discussed later in this Technical Note.
- IDB Drain #7: represent it in 1D, 2D or not at all? IDB Drain #7 provides a significant proportion of the in-bank channel storage of the overall Leiston Drain catchment and provides a conduit for flood flow between the floodplain at Eastbridge to Leiston Drain (and vice versa). The model has been tested with all three of the above representations in order to determine the potential effects of each schematisation. The findings suggest that not including the watercourse (in either 1D or 2D) may make calibration of the model problematic (under estimation of channel volume) and would not simulate known flood mechanisms in sufficient detail. However, modelling it in 2D would over estimate its capacity and could therefore also lead to poor representation of mechanism. Drain #7 has therefore been modelled in 1D, utilising extended cross sections and adopting the same schematisation used in other parts of the model (extending into high ground).
- The upstream reach of IDB Dain #7 has not been surveyed due to land access issues. This reach has therefore been included as a simplified channel geometry (based on LiDAR and field observations), which is preferable to excluding the reach and not allowing for the flood mechanisms that would occur. It is recognised that the channel geometry may be under or over-estimated in the upper reach. However, this simplification is considered to be justified given the importance of ensuring that this mechanism is represented, rather than omitted from the model (and the fact that land access will not be granted therefore survey cannot be obtained).
- Field Drainage. A number of field drain outfalls (both surveyed and unsurveyed) have been included in the model. These details have been included to allow direct rainfall to be routed into the modelled 1D watercourses (while channel levels are lower than those in the floodplain). This will also allow drainage and drawdown to occur as the flood peak subsides (and hence gain a better understanding of flood mechanisms and changes

once the development scenario is considered). Their inclusion will allow a more robust representation of the movement of water through the catchment.

- Representation of some of the field drainage has been undertaken using zlines. The model utilises an 8m grid therefore the capacity of the channels that these imprint on the 2D topography may over estimate storage. The number and extent of these drainage channels has therefore been kept to a minimum and the bed level of the drainage has been set artificially high to within ~100/200mm where possible of the normal ground levels (as leading up to a flood event it is considered that they would be full of water and therefore there would be no storage capacity). The field drains are included in the model to help drainage of shallow depths of water rather than to model the storage capacity of the drainage system.
- Direct rainfall[3] has been utilised in the low lying areas of the floodplain. By applying direct rainfall, the risk of double counting runoff using the 'lowland' method has been removed (as commented on in the meeting of 30th January). Direct rainfall has been applied to the model in the bottom of the valley rather than on the valley sides. This is a twofold measure as the runoff from the valley sides has been included in the FEH boundary units and secondly this will avoid potential areas of ponding water which are not connected to direct flow from main watercourses (in lower return period scenarios). As direct rainfall has been applied, tests of the outputs will be undertaken to determine if mapping should be considered above a shallow depth threshold (for example above 10 or 50mm only). The area covered by the 2d_rf rainfall boundary units is smaller in plan area than the physical area depicted in the hydrological investigation; therefore an appropriate scaling factor will be used to ensure the correct volume of water is applied to the 2D model (which will also need to take account of percentage runoff to ensure the correct volume is simulated). The scaling factors will be determined through the calibration process.

4 Detailed Methodology

A more detailed description of the 1D and 2D elements of the model is provided below.

4.1 1D ISIS Channels

The 'ISIS' software package has recently been re-named and upgraded to include a number of new features; the new product is called 'Flood Modeller Pro'. ISIS is still supported and is integral to the new product however the name 'ISIS' is being phased out. The ISIS model will be migrated to Flood Modeller Pro in the calibration process, however for the purposes of this Technical Note, the term ISIS is used.

The 1D ISIS model has been utilised to represent reaches of the Minsmere New Cut, Leiston Drain, Sizewell Drain, Scotts Hall Drain and IDB Drain #7. The location of the 1D model nodes are shown in Figure 1.

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[3] Sizewell FRA – Hydrology Update, Royal HaskoningDHV, August 2015

Cross section data has been generated using the various survey data outlined in Table 2. The width of each cross section has been determined based on its physical location and characteristics. However, these can be categorised into two distinct types:

- Where the watercourse is embanked and the floodplain is both extensive (in width) and lower than the bank elevation, the cross section has been 'cut back' to represent the channel from 'top of bank to top of bank'.
- Where the width of the adjacent floodplain is narrow or the floodplain rises into higher ground, the cross sections have been extended into high ground. This is the same method as outlined in the previous model schematisation note.

Initially all cross sections were 'cut back'. However through time and the development of the model, model mass error and hence stability has been improved by extending cross sections in the reaches where the channel is bounded by high ground.

Roughness coefficients in the hydraulic model are still to be calibrated. However the 'in bank' channel roughness coefficients implemented in the model to date range from 0.040 to 0.045, with bankside roughness coefficients of 0.050 to 0.080 being utilised to represent surfaces from grassed embankment to tree/scrub in winter conditions. These have been derived utilising the Cowan's method. Winter channel conditions have been assumed due to the seasonality of flooding in the catchment.

The current model in-bank roughness coefficients are less conservative than those of the current Environment Agency model which is documented to also represent winter conditions. There are some significant differences in roughness coefficient (~100%) between the former model and the proposed schematisation; these occur in the Leiston catchment. Table 4 shows the differences in roughness coefficient between the two models.

Table 4 - In-bank Roughness Coefficients

Watercourse	Current EA Model Manning's 'n' Coefficient	Proposed Model Manning's 'n' Coefficient
Minsmere New Cut	0.040 – 0.048	0.040 – 0.045
Leiston Drain	0.040 – 0.077	0.040 – 0.045
Sizewell Drain	0.054 – 0.086	0.045
Scotts Hall Drain	0.045	0.040
IDB Drain #7	Not modelled	0.040

Our roughness coefficients are based on field observations of the various watercourses in winter conditions assuming little/no vegetation. Although many of the watercourses can be weed/vegetation choked during the summer months, they have been observed to be devoid of vegetation in winter months. The degree of conservatism adopted in the approach to deriving the roughness coefficients will therefore change the coefficients calculated. An example of the variation in vegetation that can be observed is shown in Figure 2.

Sensitivity tests will be performed on roughness coefficient once the model development, calibration and baseline model runs are completed.

4.2 1D ISIS Model Structures

A number of structures have been included in the 1D hydraulic model which can be seen in Figure 3. Structures shown in red have been surveyed while those that are shown black have either been modelled based on estimated geometries through field observations or are to be surveyed. Circles represent structures that either allow continued flow in the 2D floodplain (under/through embankments or roads) or connect the floodplain back into the 1D model (field drains).

Structure details for the various outfalls at Minsmere sluice have been based on As Built Drawings, the dimensions/elevations of which have previously been verified and agreed with the Environment Agency[4].

The current 1D model assumes no blockage of structures or opening of the penstock on the Minsmere New Cut outfall. Sensitivity tests will be performed at a later date to determine the effects of 50% blockage of both outfalls The Environment Agency have divulged that due to recent changes to the Minsmere outfall (removal of a grates and replacing them with high funnels), it is expected that historic blockage due to sediment getting into the system will no longer occur. It has been previously agreed that a 50% blockage of both culvert will be considered sufficient to determine upstream impacts.

Flood warning procedures provided by the Environment Agency suggest that if a flood is likely to occur, the penstock at Coney Bank may be closed (water levels are monitored by the RSPB). However, the model assumes that the penstock in Coney Bank remains open and therefore the culvert through the bank has been represented to allow the routing of flow from the Coney catchment into the Scotts Hall Drain catchment. This assumption is a worst case scenario in terms of potential volume in the Scotts Hall Drain and negates the need to include details of the penstock or a set of rules for the opening/closing of the penstock.

4.3 1D ISIS Model Hydrology

Hydrological boundary conditions for the 1D model are discussed in detail in a separate Technical Note[5].

4.4 2D Model Extent and Grid Size

The model currently uses an 8m grid which strikes a balance between model run times and the level of detail required to accurately model flood behaviour. The 8m model currently takes ~10 hours to compile (80 hour event hydrograph representing a 21 hour 1 in 1,000 year return period, although the flows are yet to be confirmed) while a 5m grid model would take between one to two days to compile. The larger 10m grid was not adopted as it caused instabilities towards the peak of the flood event that has been utilised to build the model (an unconfirmed 1 in 1,000 year return

[4] Email Dated 12th Marc [redacted]
[5] SZC-NNBPEA-XX-000-REP-000066

period); the instabilities were a product of volumetric exchange being transferred from 1D to 2D and the channel storage available in the 'cut back' 1D channel sections. The extent of the model domains can be seen in Figure 4.

Where elements of the model are represented in 1D, the 2D grid has been nulled to ensure double counting of floodplain storage is not realised. It is possible to reduce the size of the 2D model domain in these locations (which may improve model run times by a small proportion). However, at present the 2D domain has not been reduced as the underlying data is required to be able show 1D water levels (using TUFLOW water level lines) in animation plots which may be required for future use.

4.5 2D Topography

The topography of the 2D hydraulic model has been derived utilising a number of data sources and techniques. The majority of the 2D model topography has been generated utilising a DTM (Digital Terrain Model) created from filtered LiDAR supplied by the Environment Agency. The model terrain has been improved (smoothed) to aid model stability utilising the TUFLOW 'Interpolate ZHC ALL' command. This command is typically used where models with "bumpy" terrain, such as that from airborne laser surveys, might benefit from using Interpolate ZHC or Interpolate ZUV[6].

It was intended that a full bank survey would be commissioned along the Minsmere New Cut downstream of Dam Bridge in order to correctly depict the elevation of the banks (interface of the 1D and 2D models). However, due to land access issues and the timing relative to the bird breeding season, it has not yet been possible to collect this data. It is currently intended that this survey will be undertaken and delivered in autumn 2015. In the absence of this data, filtered LiDAR data has been utilised to determine bank elevations along the watercourses in the model. A GIS routine has been run to determine the maximum bank elevation within 20m either side of the channel centre lines (at a 20m interval) so that detailed bank topography can be represented in the model.

On inspection of the outputs from this process, it was evident that the algorithms used in the filtering process have removed trees and other vegetation. In many locations they have also caused the embankments along the watercourses to be removed and therefore the process returned values that were not representative of the true bank elevation. Therefore a manual inspection of the filtered, unfiltered and aerial photography has been undertaken to identify and remove spurious values.

Figure 5 shows a reach of the Minsmere New Cut between Dam Bridge and Minsmere Sluice and depicts where the river bank (elevation) has been removed in the filtered LiDAR. Assuming access to the river banks can be obtained and data is collected, the model will be updated with new topographic data to ensure that the bank crests are represented correctly. If this should not be possible, we intend on utilising manually inspected Filtered LiDAR elevations to depict bank crests.

Part of the Minsmere Sluice refurbishment work undertaken in 2013/2014 by the Environment Agency included bank raising/stabilisation. The plan drawing of the scheme (109417-0010AB

[6] TUFLOW User Manual (Build 2010-10-AB), BMT WBM, 2010

General_Arrangement) and associated drawings have been used to assume bank levels as depicted in Table 5. (00xxAB represent the As Built Drawing numbers). This will be checked and amended if necessary as and when additional survey is undertaken.

Table 5 - As Built Bank Levels

Reach	Local Capping (m AOD)	Embankment Raising (m AOD)	Bank Reinstatement (m AOD)	Erosion Protection (m AOD)
Reach A	1.250 (0031AB)	1.210 (0031 AB)	Assumed 1.230 Assumed level between 1.210 (raising) and 1.250 (capping) and given its location on plan drawing Insufficient details on drawing 0031AB to determine actual elevation	None proposed
Reach B	Assumed 1.245 Assumed level between 1.240 (Reach C) and 1.250 (Reach A) Insufficient details on drawing 0031AB to determine actual elevation	Assumed 1.245 to keep same level as capping in Reach B	None proposed	Assumed 1.245 to keep same level as capping in Reach B
Reach C	1.240 (0031AB)	None proposed	None proposed	None proposed
Reach D	1.410 (0-40m) (0031AB) 1.355 (40-180m) (0031AB)	None proposed	None proposed	1.133 (0037AB) top of headwall (although this appears low compared to local capping)

For calibration purposes, if the event has occurred pre changes to the embankments, manually checked filtered LiDAR levels will be used, for events occurring after these changes, the levels above will be assumed.

The Black & Veatch 2006 report[7] (written before the bank level and Minsmere Sluice works were undertaken) and a meeting held with RSPB [redacted] suggests that the first ‘over bank’ flood mechanism that would be expected is spilling into the washland on the right hand bank of the Minsmere New cut, approximately 700-1,000m upstream of Minsmere Sluice. Due to the small capacity of the washland this will be followed by the second mechanism which is spilling on the left hand bank opposite the washland. Whilst this is the understood historical flood mechanism, inundation may no longer occur in this order due to the recent embankment works undertaken at this location. However, it is expected that the model will calibrate to show mechanisms occurring in this order for events pre-dating the changes to the bank levels.

A number of embankments crossing the floodplain have been included in the hydraulic model. The crest elevation of these have been derived from filtered LiDAR, and these have also been checked due to the effects of filtering as seen in the river embankments. The location of the raised embankments and spot levels (both shown in red) depicting the top of the river banks are shown in Figure 6.

[7] Freshwater flooding and drainage at Minsmere RSPB reserve, Black & Veatch, October 2006

4.6 2D Model Roughness Coefficients

Mastermap polygons have been used to assign roughness coefficients to the 2D model. Figure 7 shows part of the model domain and the distribution of different land uses within it. Table 6 lists the roughness coefficients used for different land uses within the model (extracted from the model .tmf materials file). Roughness coefficients have been derived to be befitting of a winter scenario as implemented in the 1D model. Figure 7 shows a selection of the Category Numbers listed in Table 6.

Table 6 - 2D Model Land Use and Roughness Coefficients (Materials File)

Category No	Manning 'n'	Classification
1	0.500	BUILDINGS
2	0.500	GLASSHOUSES
3	0.060	SLOPE
4	0.060	BOULDERS
5	0.030	WATER
6	0.070	CONIFEROUS
7	0.070	COPPICE
8	0.080	HEATH
9	0.070	MARSH REEDS OR SALTMARSH
10	0.080	NONCONIFEROUS TREES
11	0.070	ORCHARD
12	0.080	ROUGH GRASSLAND
13	0.100	SCRUB
14	0.020	STEP
15	0.020	GENERAL SURFACE - MANMADE (CONCRETE/ASPHELT)
16	0.050	GENERAL SURFACE NATURAL (GRASS) - THIS IS RELATIVLY HIGH AS THE GRASS IS NOT KEPT BUT IS IN LINE WITH 1D MODEL BANK ROUGHNESS
17	0.070	GENERAL SURFACES MULTIPLE (GARDENS)
18	0.050	GENERAL SURFACE UNKNOWN
19	0.030	FORESHORE
20	0.020	RAIL - TRACK
21	0.050	RAIL - EMBANKMENT
22	0.020	PATH - MANMADE
23	0.070	PATH - NATURAL
24	0.020	ROADSIDE - MANMADE
25	0.035	ROADSIDE - NATURAL
26	0.035	ROADSIDE - UNKNOWN
27	0.020	ROAD - MANMADE
28	0.040	ROAD - NATURAL
29	0.020	ROAD AND PATH STRUCTURES
30	0.030	WATER
31	0.030	ROCK
999	0.050	UNCLASSIFIED - PLEASE CHECK EACH DATA SET FOR 999 AND RE-NUMBER AGAINST AERIAL PHOTOGRAPHS

4.7 2D Model Boundary Conditions

The topography of the catchment and level of detail used in representing non main river drainage (field drains) within the model has necessitated hydrological inputs to be added to the 2D model domain which are then routed via the model back to the 1D modelled watercourses. Figure 8 shows the location at which flow from upland sub-catchments derived in the hydrological investigation are added to the model (as hydrographs).

Rainfall falling on the flat, low lying floodplain will exhibit a different runoff response to that from the steeper upper catchment. The previous modelling schematisation note discussed runoff from the low lying catchment would be derived utilising the lowland hydrology method, however this has subsequently been changed to a direct rainfall schematisation. This change will address the raised observation (Action 11) that the model must not double count attenuation of the hydrograph.

Direct rainfall volumes will be calculated for the areas at an elevation below 3m AOD. It is assumed that in the lowest parts of the floodplain (RSPB reserve and Sizewell Belts), due to high groundwater levels (to be determined) or standing/ponding water, will exhibit a high percentage runoff contribution (80-100%). Runoff percentages from different parts of the low lying catchment can be adjusted during the calibration process and will be taken forward into subsequent phases of the modelling.

The physical extent that the direct rainfall is applied in the model is less than that occupied by the 3m AOD contour. This is to ensure that water collects in the bottom of the valley and then as water levels rise, it will spread across and up the valley slope. Utilising direct rainfall can necessitate simplifications in mapping model outputs. Direct rainfall will ‘wet’ every model cell where the boundary unit is applied. Although a cell may become wet, the depth of flooding (for example on a slope) may be less than one millimetre and therefore the cell should not be shown to be at risk of flooding as it would be misleading. This can be overcome by mapping depth above a certain flooding threshold (we initially propose a depth of 50mm; this will be tested during calibration of the model).

The application of the direct rainfall has been assumed to help reduce the risk that this will need to be undertaken; however the model outputs will be checked and labelled accordingly if this approach is used. Figure 9 shows the extent that the direct rainfall is applied in the model.

5 Next Steps

The following summarises the next steps to be undertaken in order to provide information for the FRA:

- Calibration and verification of the model to a number of recorded flood events.
- Derive a comprehensive list of model scenarios prior to running baseline and development scenarios.
- Further topographic survey is planned (pending grant of land access); this will provide additional data for the model construction (both 1D and 2D elements). Once this data has been received, we will add it to the hydraulic model as appropriate.

- Undertake baseline, construction and post development scenarios. Once the baseline modelling is completed, up to date information on construction sequencing and the final post development form (plot plans, etc.) will be obtained from EDF, and the model will be updated to reflect this information by amending the 1D model cross sections and 2D model domain as necessary. Depending on construction sequencing, it may be necessary to assess several stages during the construction phase, as well as the fully completed (post development) scenario.
- Utilise model outputs to inform the Flood Risk Assessment. This will include preparation of a model build report describing the model build process, calibration, validation of outputs and scenario modelling, to support the full FRA reporting.

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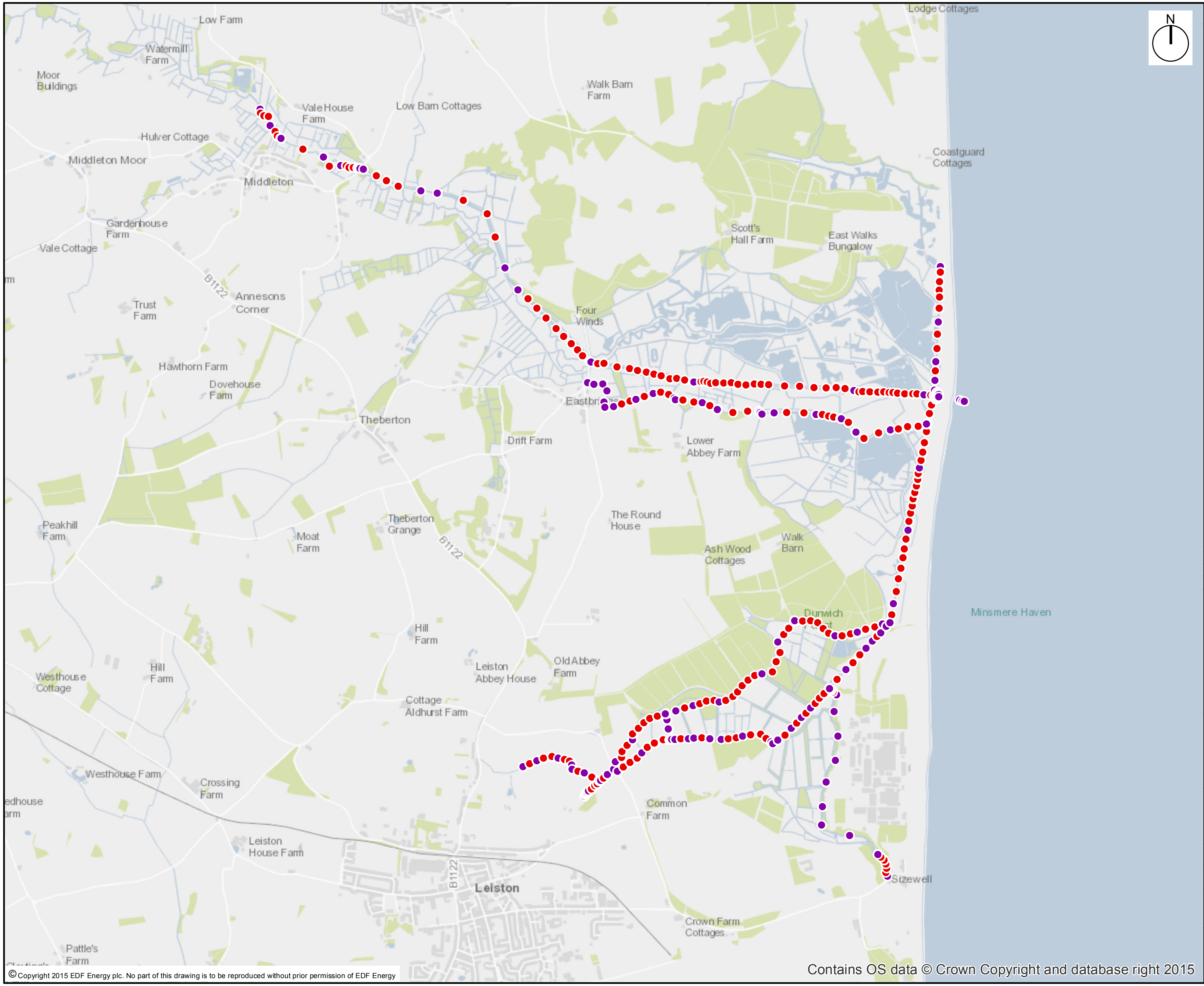
FIGURES



INTERNAL BUSINESS COLLABORATOR REFERENCE
NOT APPLICABLE FOR ALL FIGURES

NOTES

KEY
- INTERPOLATE
- RIVER SECTION

NOT PROTECTIVELY MARKED

REVISION	DATE	DRAWN	CHECKED	REASONS FOR REVISION/COMMENTS	APPROVED
01	27/08/2015	BN	LG	FIRST ISSUE	LG

EDF ENERGY
SIZEWELL C

DOCUMENT:
RIVER MINSMERE AND SIZEWELL BELTS MODEL SCHEMATISATION UPDATE AUGUST 2015

DRAWING TITLE:
ISIS MODEL NODE SCHEMATISATION

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REVISION: 0

DATE: AUG 2015
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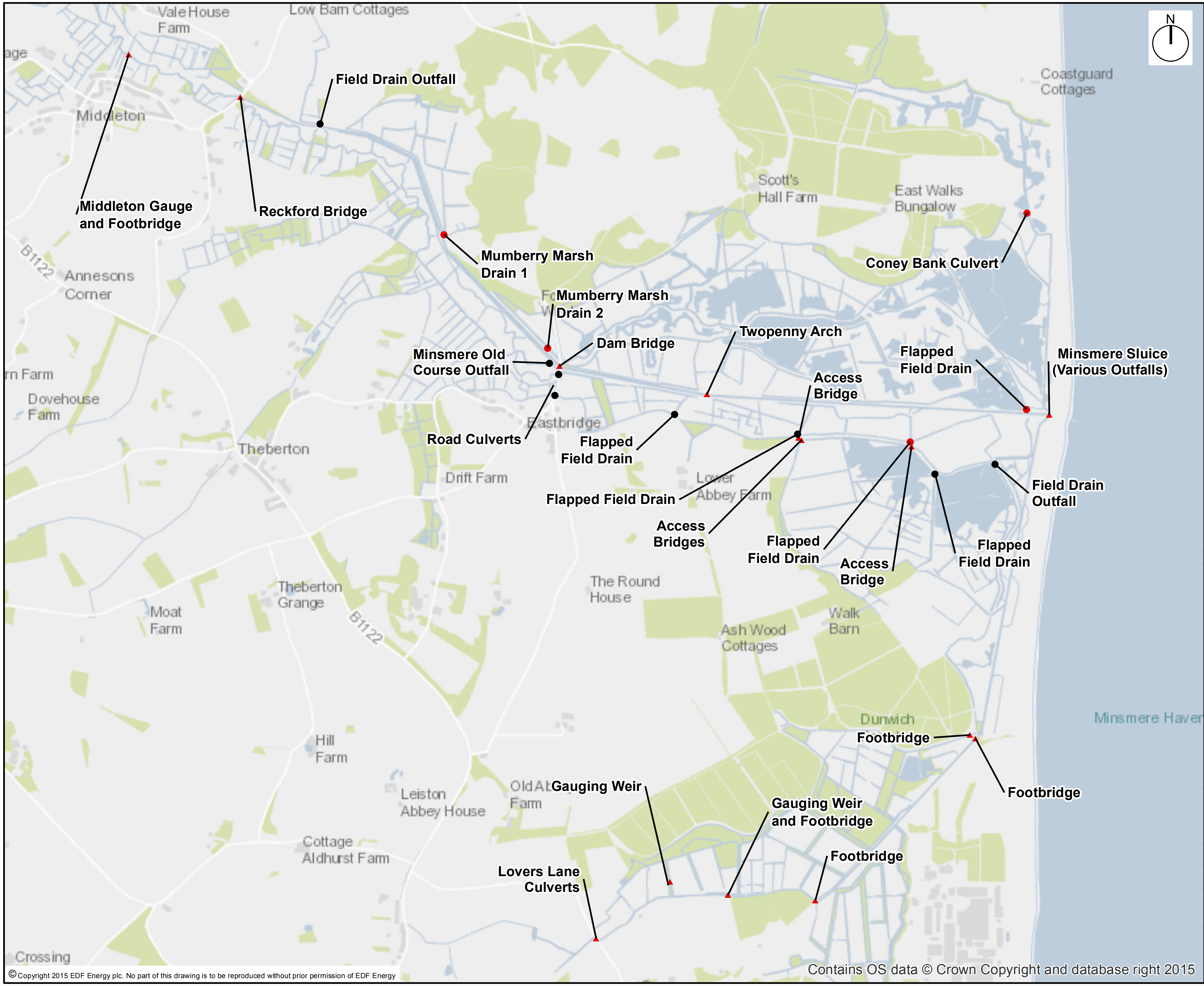
SCALE BAR
0 240 480 720 960 1,200 Meters

DRAWING SECURITY CLASSIFICATION:
☐ PROTECTIVE MARKING REQUIRED
☒ NOT PROTECTIVELY MARKED

	Summer	Winter
Dam Bridge Looking Upstream		 Source: http://www.theroostbandb.co.uk/places.html
Dam Bridge Looking Downstream		 Source: https://flic.kr/p/6ak2G

Figure 2 – Variations in Channel Vegetation at Dam Bridge



INTERNAL BUSINESS COLLABORATOR REFERENCE
NOT APPLICABLE FOR ALL FIGURES

NOTES

KEY

- Field Drain Sized From Field Observations
- Field Drain Sized From Survey
- Main River Structure From Survey

NOT PROTECTIVELY MARKED

REVISION	DATE	DRAWN	CHECKED	REASONS FOR REVISION/COMMENTS	APPROVED
01	27/08/2015	BN	LG	FIRST ISSUE	LG

EDF ENERGY
SIZEWELL C

DOCUMENT:
RIVER MINSMERE AND SIZEWELL BELTS MODEL SCHEMATISATION UPDATE
AUGUST 2015

DRAWING TITLE:
1D MODEL STRUCTURE LOCATIONS

DRAWING NO: 3 REVISION: 0

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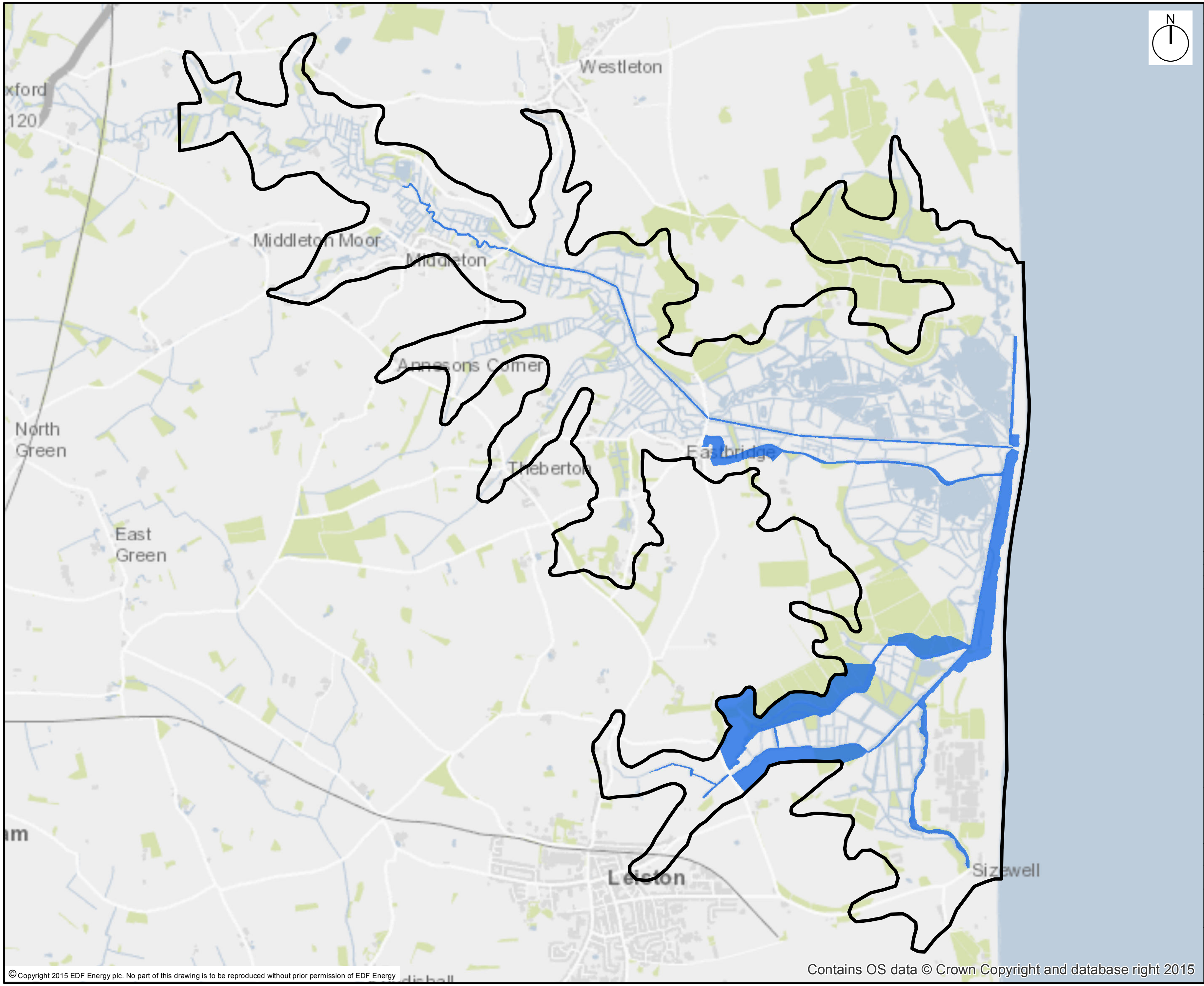
☒ NOT PROTECTIVELY MARKED



INTERNAL BUSINESS COLLABORATOR REFERENCE
NOT APPLICABLE FOR ALL FIGURES

NOTES

KEY

1D Model Extent
2D Model Extent

NOT PROTECTIVELY MARKED

01	27/08/2015	BN	LG	FIRST ISSUE	LG
REVISION	DATE	DRAWN	CHECKED	REASONS FOR REVISION/COMMENTS	APPROVED

EDF ENERGY
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DOCUMENT:
RIVER MINSMERE AND SIZEWELL BELTS MODEL SCHEMATISATION UPDATE AUGUST 2015

DRAWING TITLE:
1D AND 2D MODEL EXTENTS

DRAWING NO: 4
REVISION: 0

DATE: AUG 2015
DRAWN: BPN
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SCALE BAR
0 250 500 750 1,000 Meters

DRAWING SECURITY CLASSIFICATION:
☐ PROTECTIVE MARKING REQUIRED
☒ NOT PROTECTIVELY MARKED

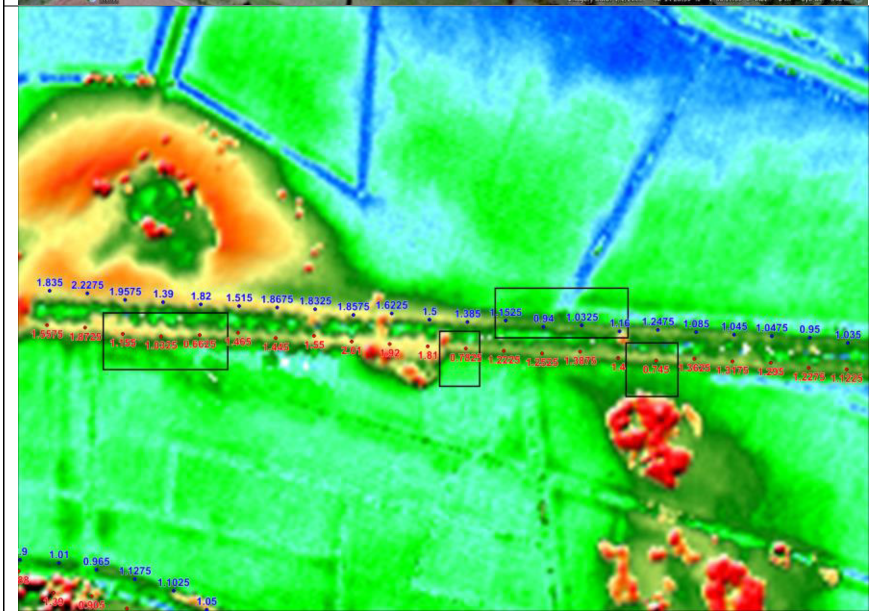
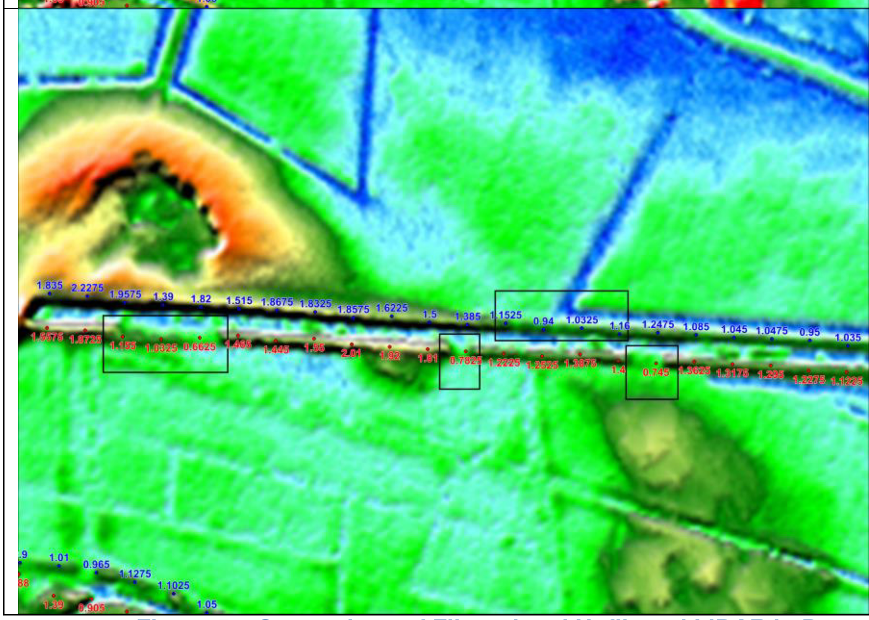
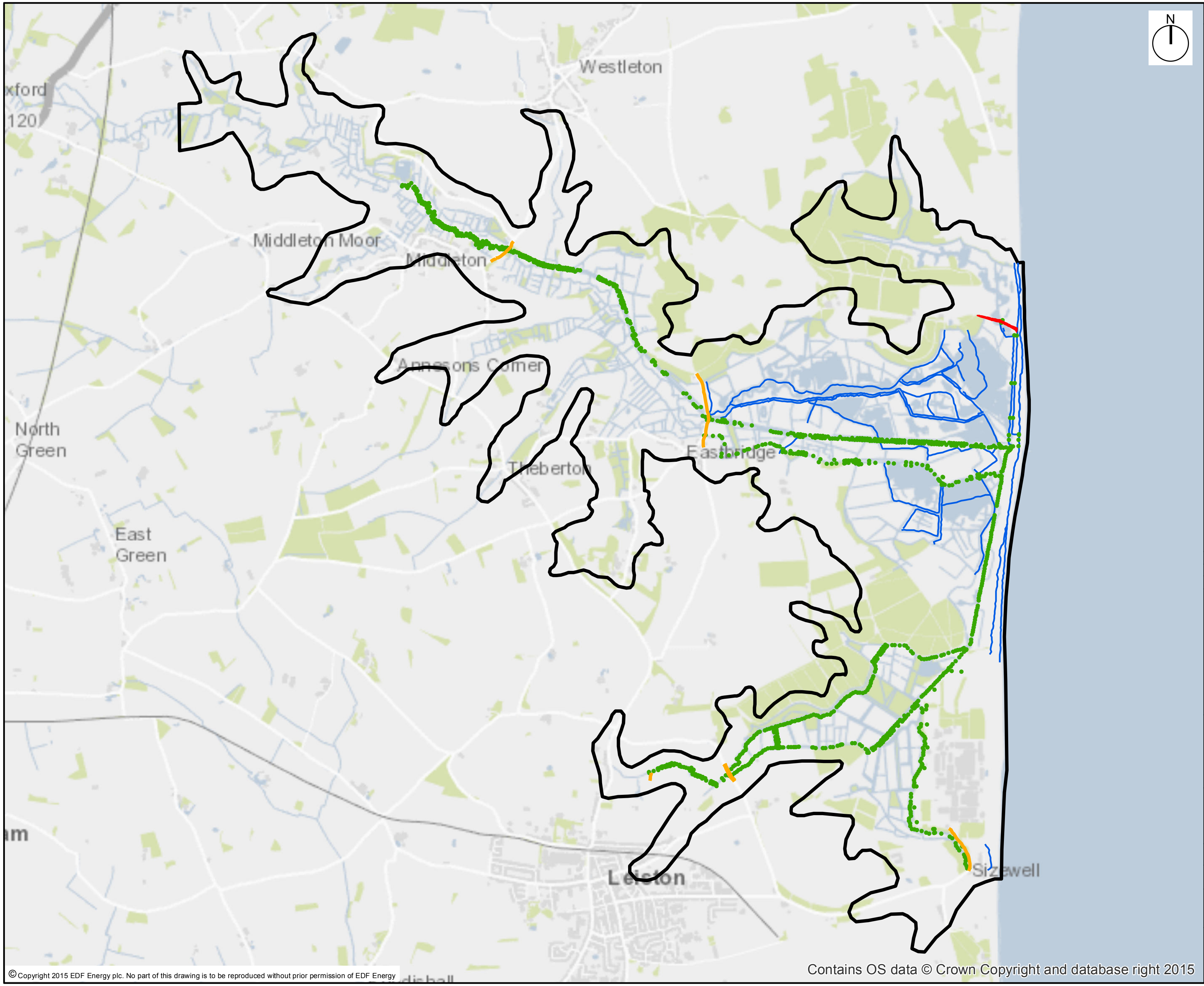
	Aerial photography (source Google Earth) showing the location of banks and trees in the floodplain
	Unfiltered LiDAR shows continuous Minsmere New Cut river banks depicted in dark green/yellow in comparison to low lying light green floodplain
	Filtered LiDAR showing non continuous Minsmere New Cut river banks (within black rectangles)

Figure 5 – Comparison of Filtered and Unfiltered LiDAR in Determining River Bank Elevations



INTERNAL BUSINESS COLLABORATOR REFERENCE
NOT APPLICABLE FOR ALL FIGURES

NOTES

KEY

- 2d Model Extent
- Raised Embankments
- Bank Elevations
- Coney Bank
- Raised Roads

NOT PROTECTIVELY MARKED

REVISION	DATE	DRAWN	CHECKED	REASONS FOR REVISION/COMMENTS	APPROVED
01	27/08/2015	BN	LG	FIRST ISSUE	LG

EDF ENERGY
SIZEWELL C

DOCUMENT:
RIVER MINSMERE AND SIZEWELL BELTS MODEL SCHEMATISATION UPDATE AUGUST 2015

DRAWING TITLE:
RAISED EMBANKMENT LOCATIONS

DRAWING NO: 6
REVISION: 0

DATE: AUG 2015
DRAWN: BPN
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DRAWING SECURITY CLASSIFICATION:
☐ PROTECTIVE MARKING REQUIRED
☒ NOT PROTECTIVELY MARKED

INTERNAL BUSINESS COLLABORATOR REFERENCE
NOT APPLICABLE FOR ALL FIGURES

NOTES

KEY

2D Roughness Coefficients

- SCRUB
- ROUGH GRASSLAND
- WATER
- MARSH REEDS OR SALTMARSH
- GENERAL SURFACE NATURAL
- HEATH
- BUILDINGS
- FORESHORE
- GENERAL SURFACES MULTIPLE (GARDENS)
- GENERAL SURFACE - MANMADE (CONCRETE/ASPHELT)
- CONIFEROUS
- GLASSHOUSES
- PATH - MANMADE
- ROAD AND PATH STRUCTURES
- ROADSIDE - MANMADE
- ROADSIDE - NATURAL
- NONCONIFEROUS TREES

NOT PROTECTIVELY MARKED

REVISION	DATE	DRAWN	CHECKED	REASONS FOR REVISION/COMMENTS	APPROVED
01	27/08/2015	BN	LG	FIRST ISSUE	LG

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SIZEWELL C

DOCUMENT:
RIVER MINSMERE AND SIZEWELL BELTS MODEL SCHEMATISATION UPDATE AUGUST 2015

DRAWING TITLE:
EXAMPLE OF 2D LAND USE (ROUGHNESS COEFFICIENT) DISTRIBUTION

DRAWING NO: 7
REVISION: 0

DATE: AUG 2015
DRAWN: BPN
SCALE: 1:15,000 @A4

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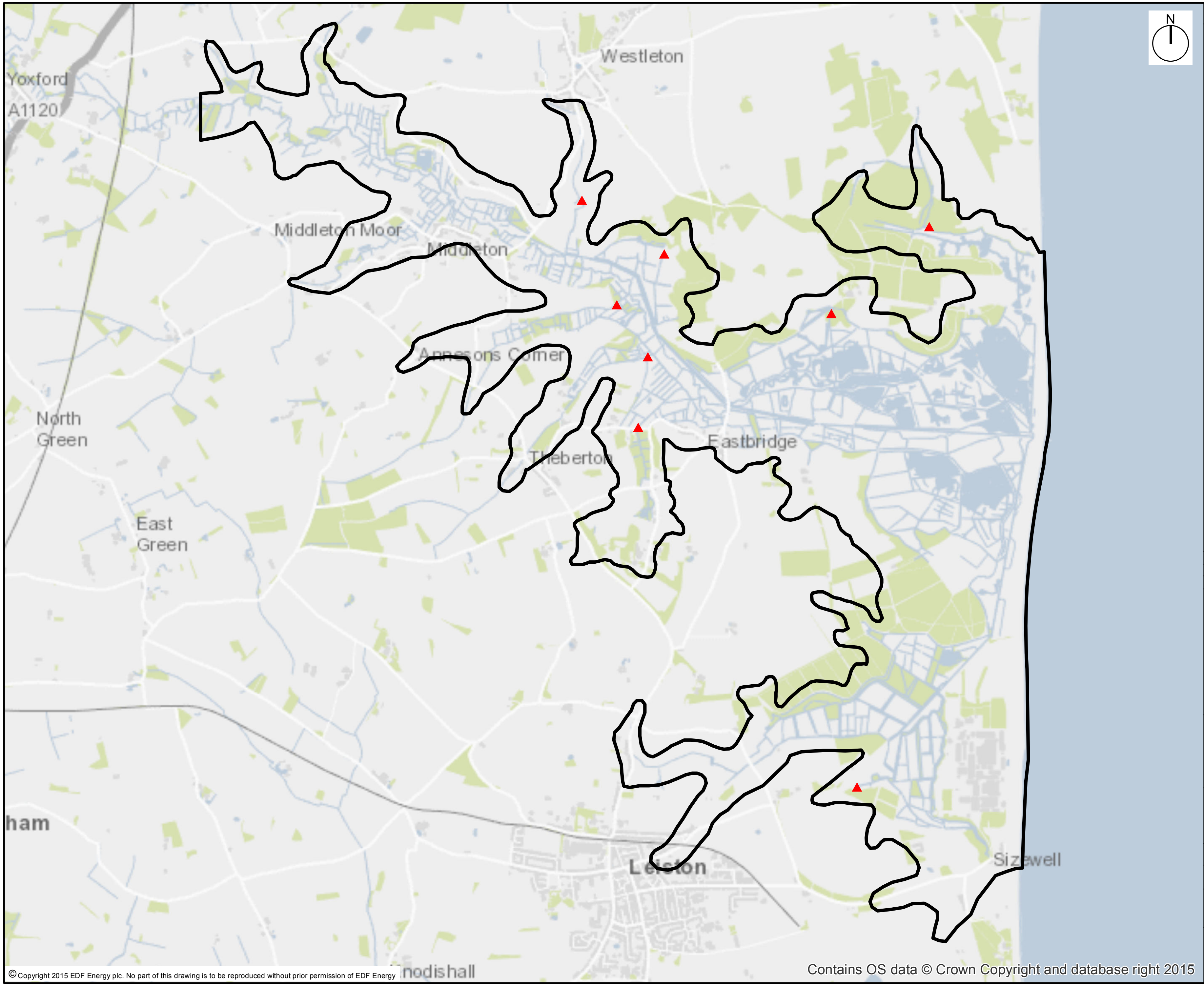
DRAWING SECURITY CLASSIFICATION:
☐ PROTECTIVE MARKING REQUIRED
☒ NOT PROTECTIVELY MARKED



INTERNAL BUSINESS COLLABORATOR REFERENCE
NOT APPLICABLE FOR ALL FIGURES

NOTES

KEY

- 2D Model Extent
- 2D Model Inflow Locations

NOT PROTECTIVELY MARKED

REVISION	DATE	DRAWN	CHECKED	REASONS FOR REVISION/COMMENTS	APPROVED
01	27/08/2015	BN	LG	FIRST ISSUE	LG

EDF ENERGY
SIZEWELL C

DOCUMENT:
RIVER MINSMERE AND SIZEWELL BELTS MODEL SCHEMATISATION UPDATE
AUGUST 2015

DRAWING TITLE:
2D MODEL INFLOW LOCATIONS

DRAWING NO: 8 REVISION: 0

DATE: AUG 2015 DRAWN: BPN SCALE: 1:40,000 @A4

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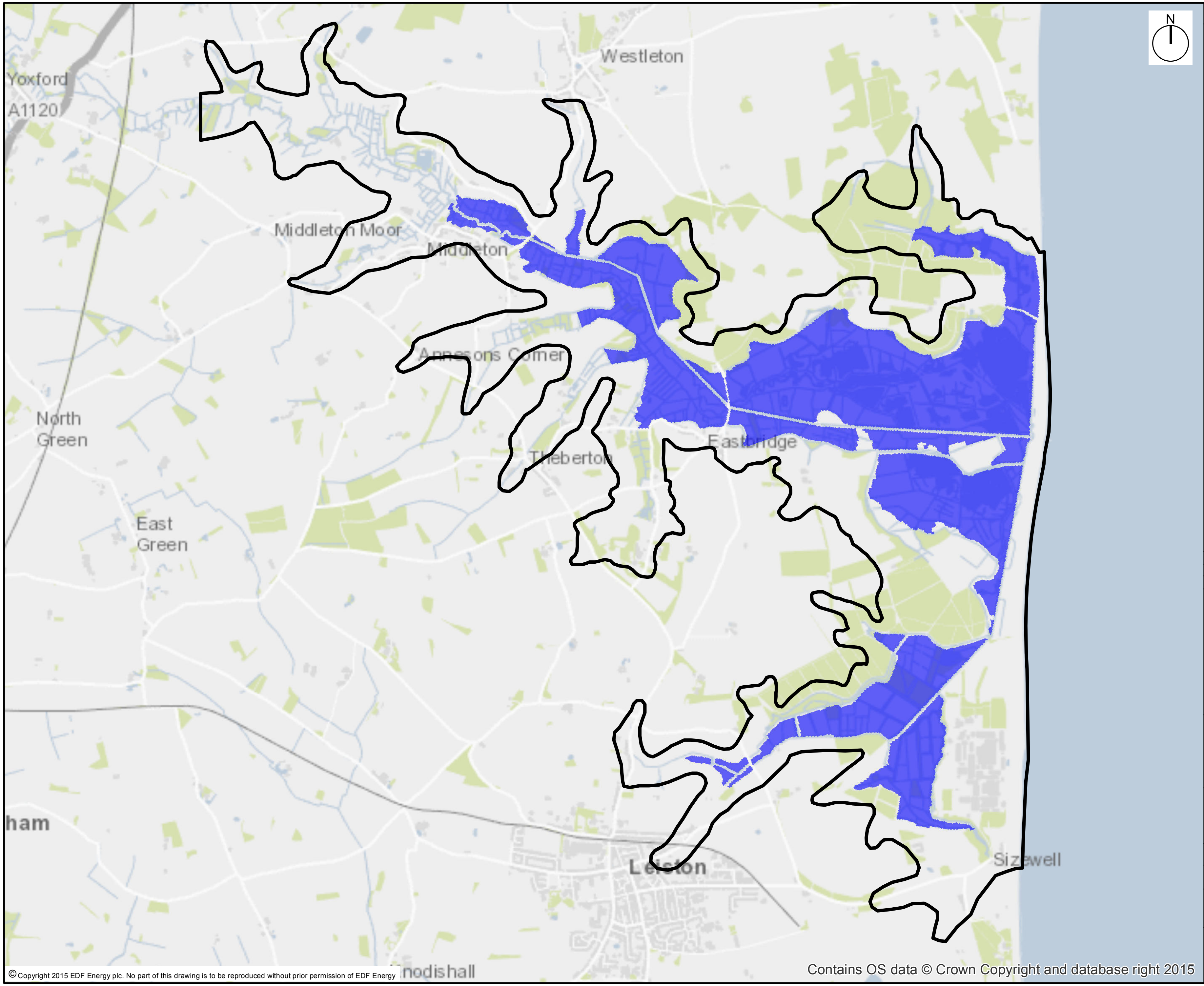

INTERNAL BUSINESS COLLABORATOR REFERENCE
NOT APPLICABLE FOR ALL FIGURES

NOTES

KEY
- 2D Model Extent
- 2D Model Direct Rainfall

NOT PROTECTIVELY MARKED

REVISION	DATE	DRAWN	CHECKED	REASONS FOR REVISION/COMMENTS	APPROVED
01	27/08/2015	BN	LG	FIRST ISSUE	LG

EDF ENERGY
SIZEWELL C

DOCUMENT:
RIVER MINSMERE AND SIZEWELL BELTS MODEL SCHEMATISATION UPDATE AUGUST 2015

DRAWING TITLE:
LOCATION OF APPLICATION OF DIRECT RAINFALL

DRAWING NO: 9
REVISION: 0

DATE: AUG 2015
DRAWN: BPN
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DRAWING SECURITY CLASSIFICATION:
☐ PROTECTIVE MARKING REQUIRED
☒ NOT PROTECTIVELY MARKED

APPENDIX B: SIZEWELL-C FLUVIAL MODELLING CALIBRATION REPORT

RHDHV, February 2017

Note / Memo

HaskoningDHV Nederland B.V.
Water

To: EDF [redacted]
From: [redacted]
Date: 23 February 2017
Copy: [redacted]
Our reference: PB1452/MMC1/301907/Pet
Classification: Open

Subject: **Sizewell-C Fluvial Modelling Calibration**

Purpose

This technical note has been prepared to outline the work undertaken and the findings of the calibration of the Minsmere catchment hydraulic model.

The model will be utilised to determine fluvial flood risk to the Minsmere catchment as a consequence of Sizewell C (SZC) construction and associated developments, as part of the Flood Risk Assessment (FRA) that is being prepared.

Despite limited availability of calibration data, the model is sufficiently calibrated and verified (by stakeholders) to provide confidence that the model fits the purpose for use in subsequent phases of the Flood Risk Assessment. This technical note provides technical background information about the calibration and verification activities conducted for the model to assess the performance of the model.

Date	Version	Reason for Issue	Checked/Reviewed
7 April 2016	V1	Draft for consultation	[redacted]
28 April 2016	V2	Draft for consultation	[redacted]
28 April 2016	V3	Draft for consultation	[redacted]
18 July 2016	V4	Final for consultation	[redacted]
06 September 2016	V5	Final calibration for consultation	[redacted]

15 September 2016	V6	Final calibration for consultation	[redacted]
28 September 2016	V7	Amended following client comments	[redacted]
23 February 2017	V8	Amended following client comments (02/12/06)	[redacted]

Abstract

The hydraulic 1D-2D model of the Minsmere catchment covers the Minsmere south catchment and Leiston catchment. The hydraulic model has been updated, semi-calibrated and validated in order to assess the impact of Sizewell C construction and associated developments to support the Flood Risk Assessment (FRA). The model has been calibrated based on available gauge data and verified using observations of flood mechanisms and flood trash marks. The model has not been fully calibrated due to limited data availability and data quality.

For the calibration, three separate events (January 2003, March 2010 and January 2016) have been selected. The model schematisation is slightly different for these runs as the Minsmere works that were undertaken in 2012 were included for the 2016 event. For each of these runs a sensitivity analysis has been conducted on three model parameters; channel roughness, initial water level and percentage runoff. Additional sensitivity runs have been done on the opening of the Minsmere Sluice southern culvert. Calibration has been conducted on the January 2016 event as for this event more detailed gauging data was available for especially the Leiston catchment, for which data was missing for the 2003 and 2010 events.

The model results show a good fit for a high channel roughness (0.065n – 0.08n) and an initial water level of 0.5m +AOD. The percentage runoff shows a best fit for the runs done with 50% instead of 40% as was also observed in the field. Model results show that the tested parameters have a relatively small influence on peak flood levels but may have a greater impact on small events. Opening of the Minsmere Sluice southern culvert decreases the water levels in Minsmere catchment and causes a small increase in water levels in Leiston drain catchment. In agreement with the Environmental Agency, it was decided to apply a uniform channel roughness of *0.08n*, an initial water level of *0.5m +AOD* and with the southern Minsmere Sluice culvert being opened as final calibration parameters. 2016 runs show that Leiston system can be calibrated to a limited extent showing good fit up to the situation where the Leiston system is affected by back flow from Minsmere. Further calibration is not possible as a result of model accuracy of the base model and limited data to calibrate it.

The overall conclusion of the model calibration is that calibration could be executed to a limited extent because of limited data availability and data quality. The model however shows good linkage with the available data and visual observations and is therefore considered representative for the Minsmere and Leiston water systems at the rising limb of the flood. For peak flood conditions the model shows an overestimation of the water level for Leiston system with about 20 cm. Overestimation of flood levels is bigger in the Minsmere system which can be up to 40 cm. For gauge G5 near Leiston town

the flood levels are slightly underestimated (10 cm). Flooding in the upstream Leiston system is mainly the result of local runoff rather than backwater from downstream.

The impact of Sizewell-C developments is expected to be in the order of 1 – 2 cm's while the model accuracy is in the order of 5 cm for low flow conditions and 10-20 cm for high flow conditions. Despite the overestimation of the flood levels, the model is able to represent the typical behaviour of the system well which makes the model applicable for the FRA. The FRA will focus on the impact of Sizewell-C and will thus be based on a comparison between the baseline and development models under varying hydrological and climatological conditions (including sea level rise). Due to overestimation of the flood levels in the calibration, a worst-case approach is proposed to assess the bandwidth of possible outcomes when running the SCZ developments under different conditions (see chapter 7: worst – case approach).

1. Introduction

This Technical Note provides detail of the work undertaken (and findings) following the publication of the Memo dated 7th January 2016[1] in relation to the calibration of the Minsmere fluvial model. The Memo and proposed way forward were also discussed in a telecom between EDF Energy, the Environment Agency, JBA and Royal HaskoningDHV on 18th January 2016.
As outlined in Section 4 of the Memo of 7th January, larger events on the Minsmere catchment have been utilised in this phase of the model calibration:

- “For hydraulic model calibration, the model will be run for the three selected calibration flood events (January 2003; March 2010; and November 2014), using the recorded flows at Middleton, observed rainfall data and the observed tidal boundary levels”.

However, although there has been significant effort to undertake and provide results for all of the above events, due to the practicalities of iterative changes to the model and the model run time of the November 2014 event (over one week to compile a 650 hour duration event), it has only been possible to undertake calibration for the January 2003 (350 hour duration) and March 2010 (300 hours) events.

Following the initial publication of the findings, EDF, Environment Agency, JBA and Royal HaskoningDHV agreed in the meeting of May 2016 to run a third event which occurred in January 2016 and provided additional data. The event has been run through the hydraulic model and the findings of this event are reported herewith. For this event data are available for the Leiston system.

This Technical Note sets out the proposed way forward following the findings of the validation and calibration of the model.

Note:
Although a full calibration cannot be executed due to data availability and quality, the total process of optimizing the model as preparation for the FRA is called “calibration”. It should be noted that the 2016 event was run with a different model schematisation as it includes the heightening of the Minsmere embankments which were undertaken in 2012. This schematisation shall be used for the Baseline model for the FRA.

[1] SZC FRA fluvial model calibration - issues and way forward v6, RHDHV 7th January 2016

2. Approach

The approach during the calibration phase of the fluvial model has been to identify and replicate (in the model) key flood mechanisms and changes in flood depths where field observations have been provided by the RSPB.

The field observations provided are gauged water levels (above the gauge datum) at Middleton gauge (supplied by the Environment Agency) and water levels (to unknown datum) within the RSPB reserve and Minsmere South Levels (supplied by the RSPB). Figure 1 shows the locations of the observations points supplied by the RSPB. As the stage data supplied by the RSPB is not levelled to metres Above Ordnance Datum (mAOD), the observed change in water depth has been used in the calibration rather than the absolute level.

To provide suitable validation and to ensure confidence in the model, it is necessary that observed flood mechanisms are replicated in the hydraulic model. Two pieces of evidence have been used to define the key flood mechanisms: Section 2.3 (Mechanisms of flooding) of the Black & Veatch 2006 Report[2]; and, observations from [redacted] of the RSPB (summarised in an email of 27th October 2014). The 2006 Black & Veatch Report states:

> *"The following account of flooding processes within the RSPB reserve is taken from personal communications with Minsmere RSPB site manage*[redacted]
>
> - RSPB note that flooding tends to occur because the Minsmere sluice gates close at high tide and there is not enough storage in the watercourse channels to contain the water coming down the river system. At times of flood or when the sluice cannot drain, water spills back from the sluice for around 400-500m and spills over the lowest areas of the banks.
> - A small washland (c81 – See Figure 2) is positioned south of the New Cut, where the bank level is artificially low. This is filled when water backs up from the sluice. However, it is too small to contain all flood events.
> - When flooding occurs, the New Cut (more so than Leiston Drain) overtops and floods into the North levels (where horses are) opposite the washland at c81. The northern bank also has several large holes in it which allows water in the northern extent of the reserve. These are scheduled for repair by the Environment Agency – It is believed that these have been undertaken by the Environment Agency as part of the Minsmere Sluice and Embankment Works in 2014-2015.
> - Floodwater can also overtop further upstream towards Eastbridge Levels which run along Drain #7. The rear gardens of houses in Chapel Road, Eastbridge have been affected by flooding in the past when Drain #7 overspills. Floodwater overtopped upstream of Eastbridge then passes down Drain #7 and can overtop the banks of Drain #7 onto the south levels over the southern bank into compartments 87, 92 and 96.
> - Once flood water has entered the washland c81, water can then overtop the southern bank of the washland (c81) and run southwards across compartment c83. This then drains into Drain #7 and then again into the southern levels via the route above.
> - Further upstream, Meadow Marsh (c40) and c41 can also be flooded by the New Cut overtopping its northern bank.

[2] Freshwater flooding and drainage at Minsmere RSPB reserve, Black & Veatch, October 2006

- Drainage of the Southern levels and the Northern levels cannot start until the water levels in the New Cut, Leiston Drain and Drain #7 are low enough to allow gravity flow.

These flood routes are shown on Figure 4."

Figure 4 from the 2006 Black & Veatch report is shown in Figure 2.

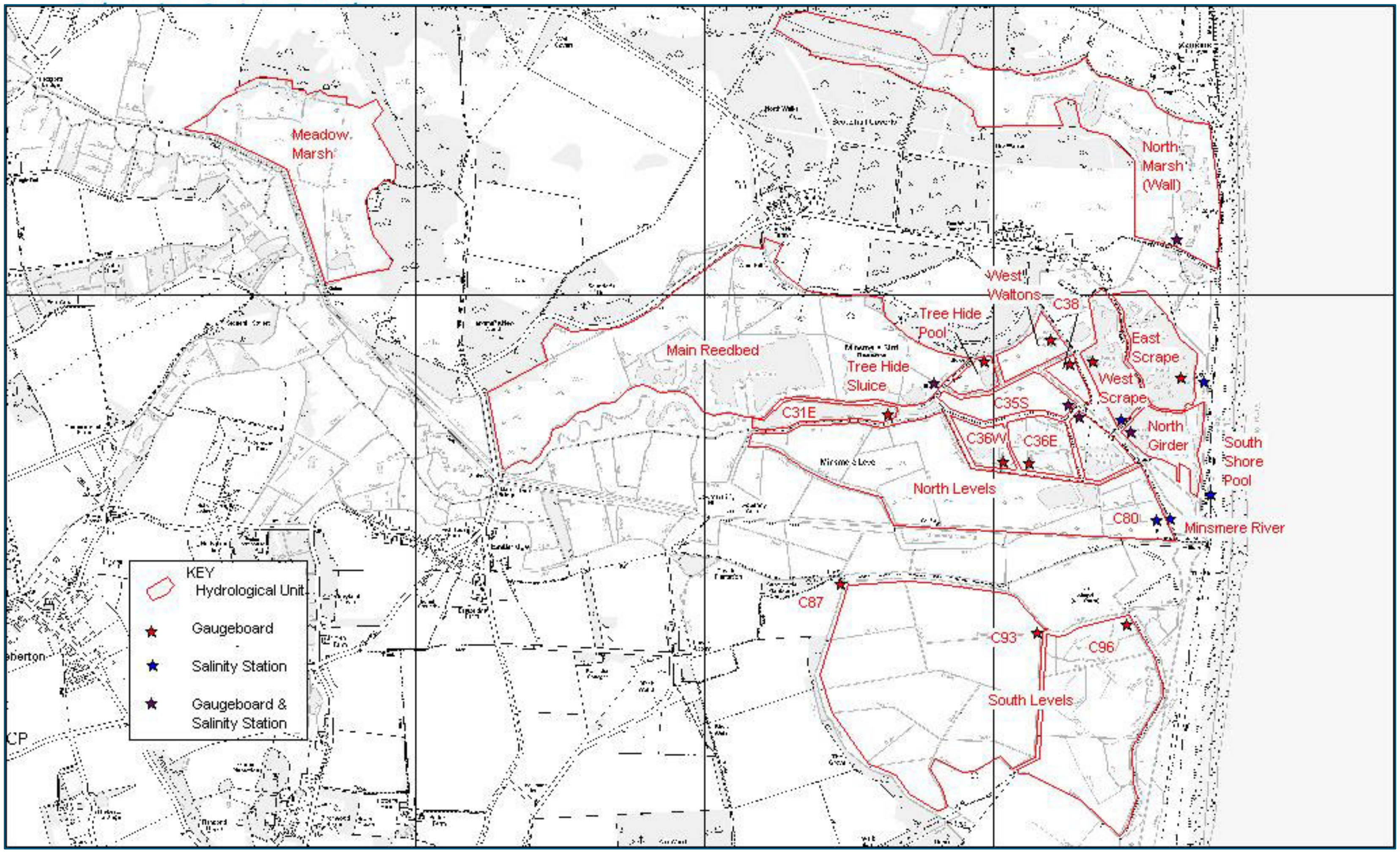

Figure 1 - Water Level Observation Levels Supplied by the RSPB

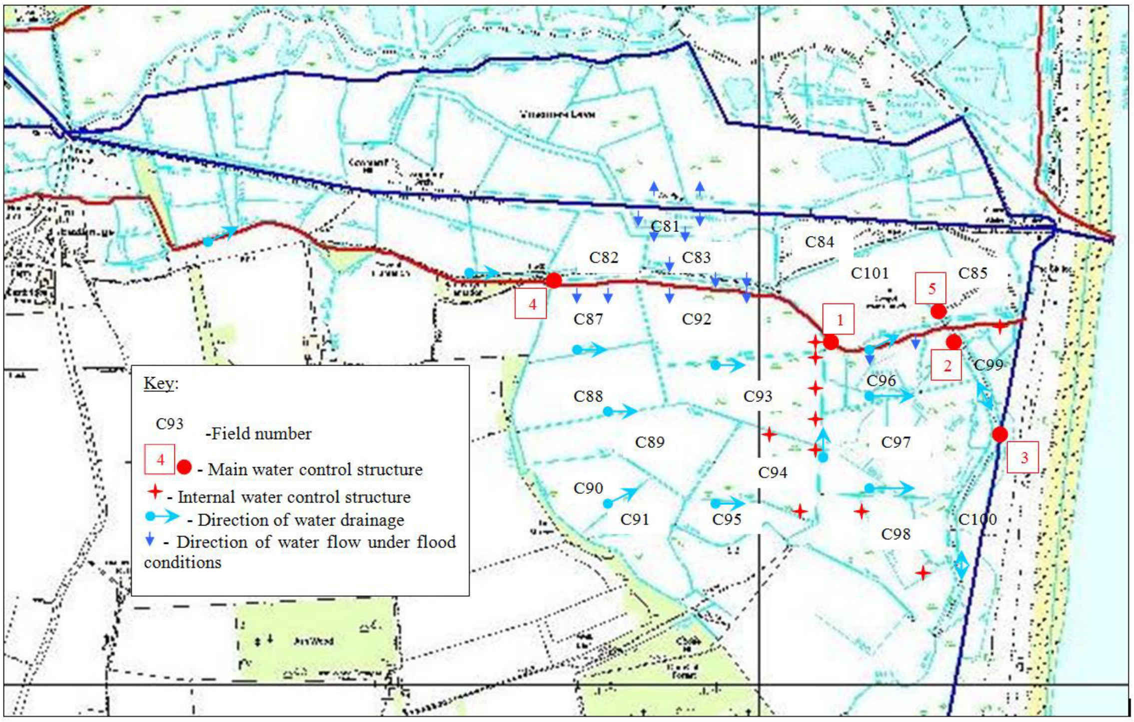

Figure 2 - Figure 4 from 2006 Black & Veatch Report

A summary of the typical flood chronology as described by Robin Harvey (meeting of 22nd October 2014) of the RSPB includes:

- *Flooding upstream of Dam Bridge in Eastbridge Meadow (the triangular area immediately North of Dam Bridge which is a separate hydrological unit from Meadowmarsh – the Africa-shaped block of reedbed and fen to the north-west) often occurs first as flows increase (however recent works by the EA to the sluice upstream of Dam Bridge have occurred which may change this – although it is thought it will not change the mechanism). Recent experience indicates that Eastbridge Meadow is still flooding due to overtopping of the New Cut bank despite repairs to the EA water control structure into the New Cut.*
- *Water levels in the North Levels will rise (Old Minsmere River) due to backing up from the east (Minsmere New Cut levels at the sluice will impede drainage) and water will begin to flow into the lowered reed beds over the banks into these areas (water does not flow through the pipes unless the upstands are out) (this is opposite the washland)*
- *Water levels in the South Levels increase due to water spilling from the Leiston Drain and over time, Drain No 7. It should be noted this is due to backing up from the Minsmere Sluice Complex and not from a direct result of water from the Minsmere New Cut (water in the New Cut is the main driver though as this has the largest capacity of all of the drains exiting at the sluice)(this is after water will spill from IDB Drain No 7)*
- *As water levels rise water will flow across the visitor trail between the Minsmere River and the Scrape and into the Scrape.*
- *Flooding over part/all of ~450m of the left hand bank (northern) of Minsmere New Cut immediately downstream of Dam Bridge due to lowest bank levels along the watercourse (the bank levels have been increased as asbestos has been removed/capped in specific locations between Dam Bridge and Minsmere Sluice, however it is expected that the flow mechanisms*

into North Levels will not change due to these changes – see locations on drawing 109417-0010AB General_Arrangement.pdf). It is possible that higher bank levels along the northern side of the New Cut may lead to increased overtopping along the southern side.

- *As water levels in the system back up from Minsmere sluice, water will spill into the North and South Levels over low points of both banks of the Minsmere New Cut before water levels fill the system and equate*

The location of the various meadows and features can be observed in Figure 3.

These two commentaries provide similar storylines of observed flood mechanism with which to calibrate the model results against.

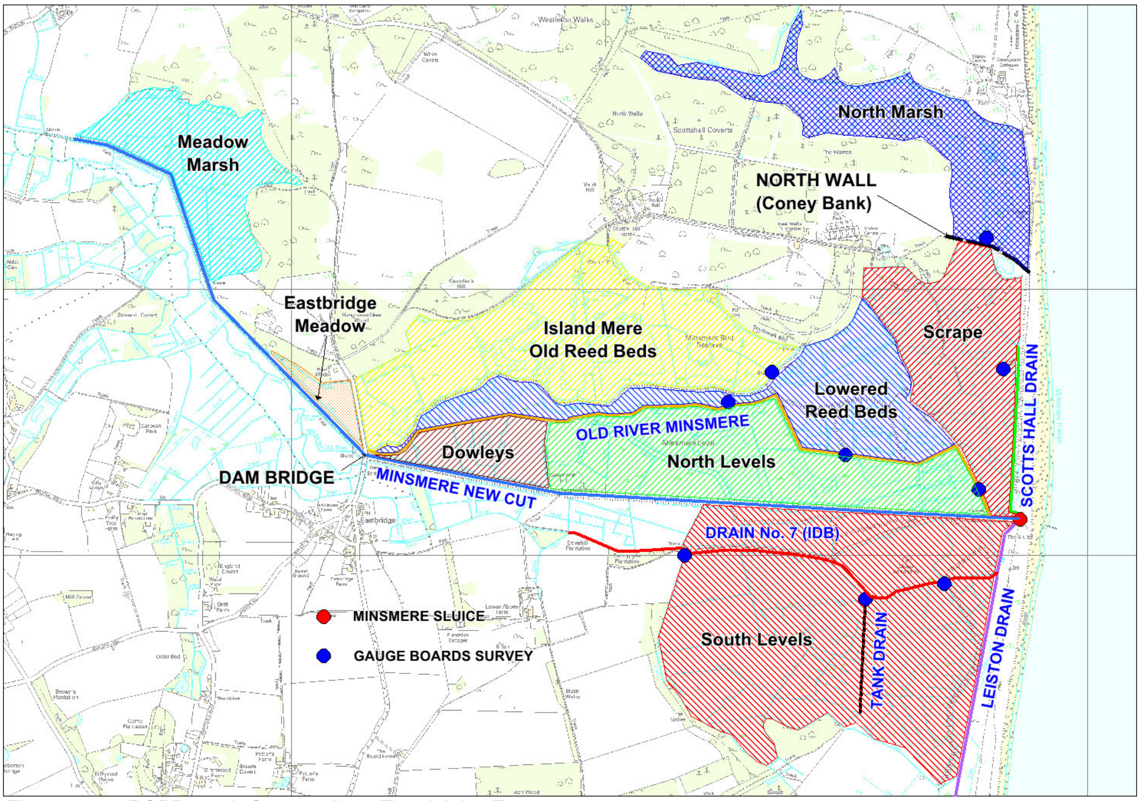

Figure 3 - RSPB and Surrounding Floodplain Feature Location

During the calibration process an anomaly in the model results was observed at Minsmere sluice, to overcome this occurrence several changes to the model schematisation were tested to isolate the problem. Eventually the (numerical) problem was resolved by adding a dummy reach (reach with no physical meaning). The design of the sluice was further checked with as-built drawings from recent renovation works. A small change in the size of the flap valve and addition of a penstock to be able to control the opening of the southern New Cut sluice were included in the model schematisation.

For the January 2003 and March 2010 calibration events that have been undertaken, a number of scenarios have been tested to help identify improvements or sensitivity to the relationship between modelled results and recorded data. The matrix of model run parameters includes:

- Roughness coefficients based on field observations for January and March (in-bank channel roughness 0.040-0.045)
- Conservative estimates of roughness coefficients for January and March (in-bank channel roughness 0.060-0.065)
- Changes to PR of the upland catchments (50% and 40%)
- Different initial water levels for the 1D and 2D domains (0.00, 0.20 and 0.50m AOD)
- Differing tidal boundary conditions based on observed data

The January 2016 event has been run for four different scenarios to help identify improvements and sensitivity. These runs are focussed on the impact of opening the southern culvert of the Minsmere New Cut at Minsmere sluice. The matrix of model run parameters includes:

- Roughness coefficients based on field observations for January and March (in-bank channel roughness 0.040-0.045)
- Conservative estimates of roughness coefficients for January and March (in-bank channel roughness 0.060-0.065)
- Different initial water levels for the 1D and 2D domains (0.00 and 0.50m AOD)
- Opening the southern culvert of the Minsmere New Cut at Minsmere sluice

After initial calibration for the January 2016 event additional calibration and sensitivity runs were undertaken to improve the models results. The results from these runs are added in an additional chapter. They provide a better insight in the system and help understand the system's response to the various parameters and inputs.

The flood mechanisms and results from these various models and scenarios are discussed in more detail herewith.

3. Hydrological Inputs

Analysis of rainfall (various gauges) and flow data (Middleton) indicated that for the January 2003, March 2010 and January 2016 events the percentage runoff (PR) to the gauge was ~50%. Therefore for the calibration and validation, upland inflows within the model have been run with a 50% PR. In order to test calibration against this figure, further models have been run with a 40% PR to test results sensitivity. The model inflow for the Minsmere New Cut has been generated using data recorded at Middleton and the stage discharge relationship of the ISIS-TUFLOW model at this location.

Inflows for the Middleton Gauge for the three events analysed are shown in Figure 4 to Figure 6.

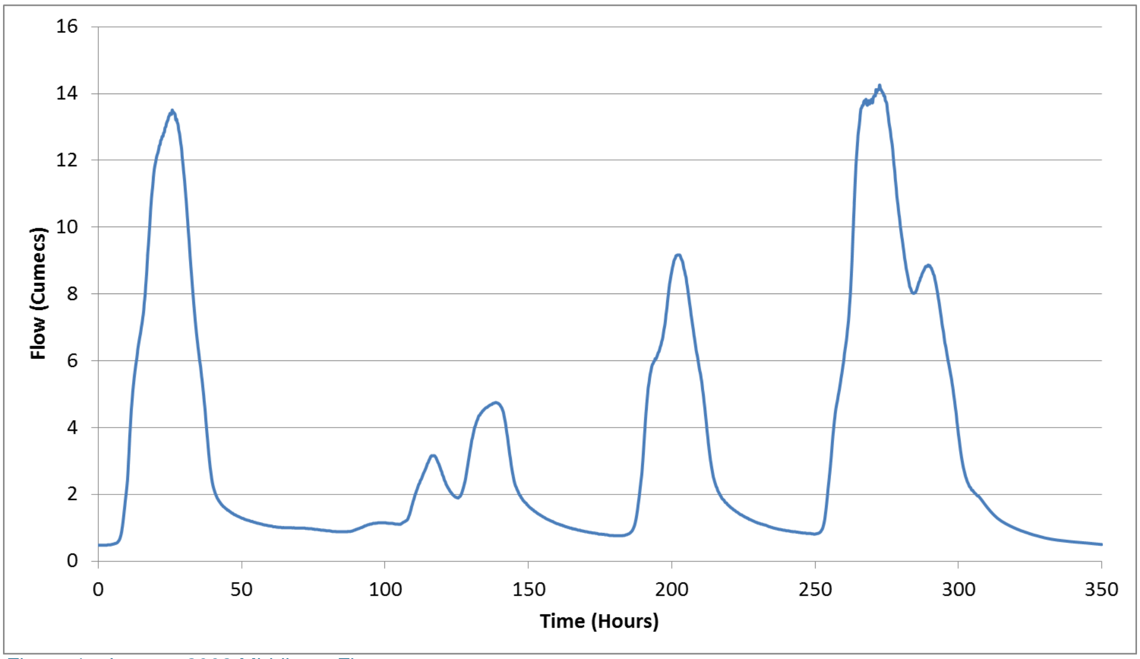

Figure 4 - January 2003 Middleton Flow

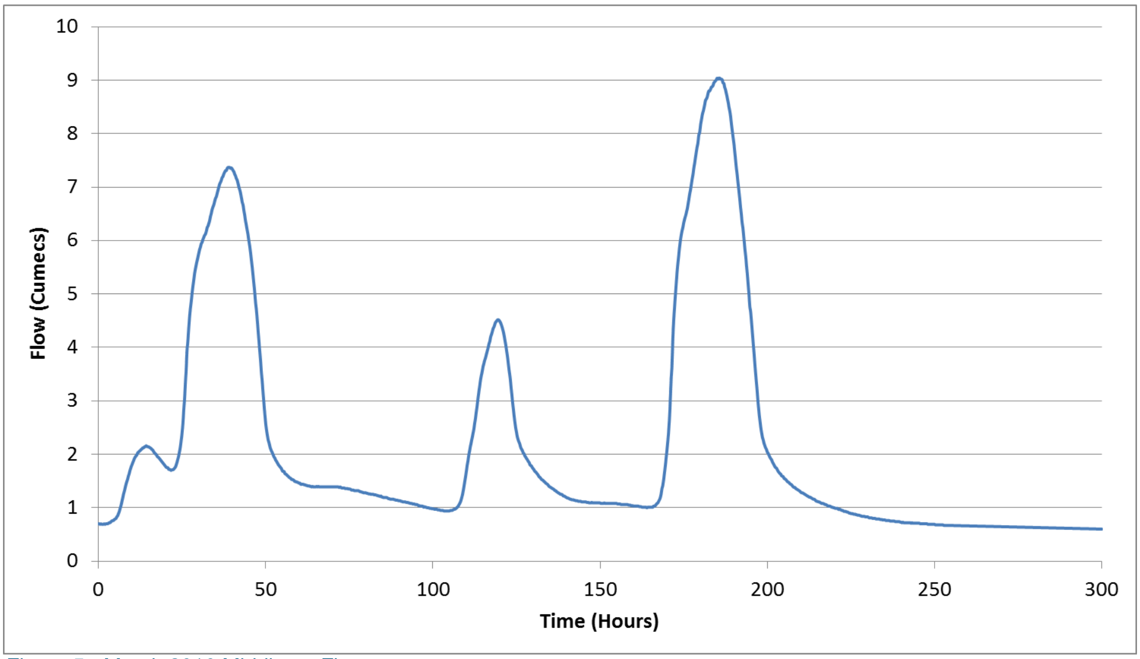

Figure 5 - March 2010 Middleton Flow

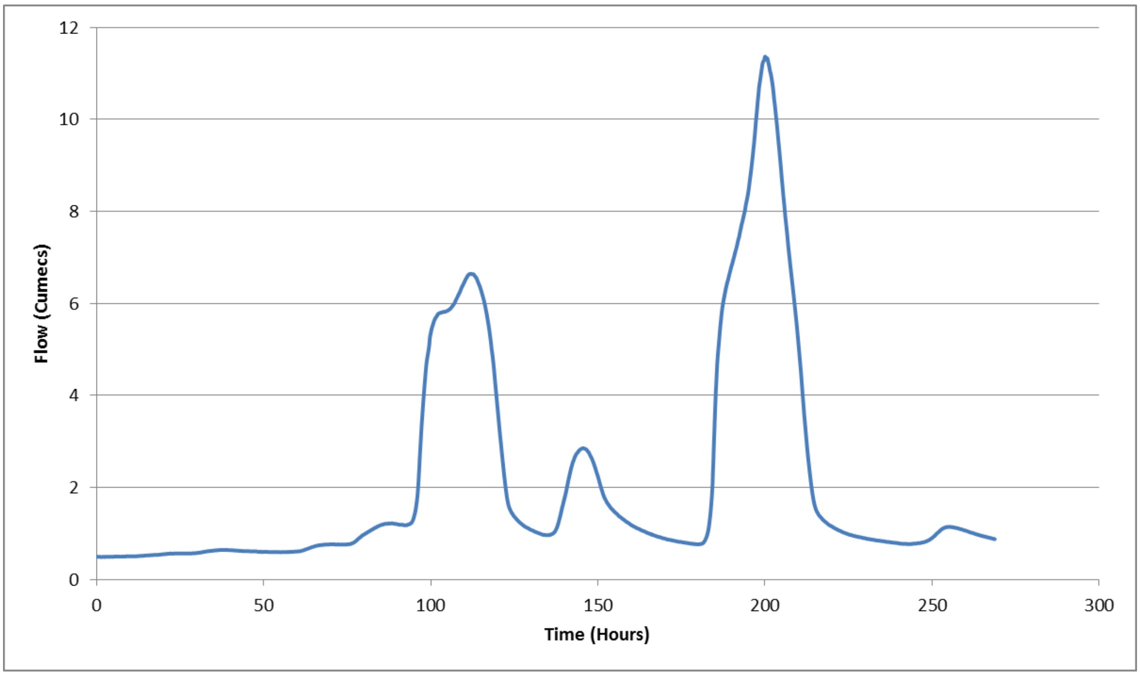

Figure 6 - January 2016 Middleton Flow

Rainfall inputs to the 'Lowland' system (from gauge data) have been prescribed as a 50% PR for the floodplain upstream of Eastbridge and 90% PR downstream of Eastbridge. These figures were determined from analysis of the floodplain elevation and slope within the lowland area. Downstream of Eastbridge it is expected that the floodplain would be waterlogged and therefore would exhibit a high percentage runoff whereas upstream of this location, due to the higher (but still shallow) gradient it was expected that there would be a lower runoff. Observed rainfall data from Benhall (2010 and 2016) and Earl Soham (2003) have been used in the calibration. Rainfall depths were checked against the daily rainfall gauge at Westleton to ensure that there were no anomalies in the volume of rainfall. None were found.

Tidal boundary data for the calibration has been derived utilising recorded tide data at Lowestoft. The recorded data at Lowestoft has been translated to be representative of the tidal harmonics at Minsmere Sluice. Admiralty harmonics have not been used for this translation as analysis of the outputs from this transformation, and following discussions with CEFAS, it was determined that the Admiralty harmonics at Minsmere sluice are not correct. High and low tide data for Lowestoft and Sizewell B stations (February 2009 to December 2012 supplied by CEFAS) have been utilised to identify a suitable relationship/transformation of tide levels from Lowestoft to Minsmere Sluice. Figure 7 shows the three limb linear relationship that has been adopted to transform recorded stage data at Lowestoft to Minsmere Sluice, which has been used to create calibration boundary conditions.

In order to test the sensitivity of the model results to tidal boundary conditions, the raw Lowestoft gauge data (converted to mOD for mCD) has also been run for a selection of events/scenarios for the 2003, 2010 and 2016 events.

Figure 8 to Figure 10 show the two different tidal boundary conditions applied.

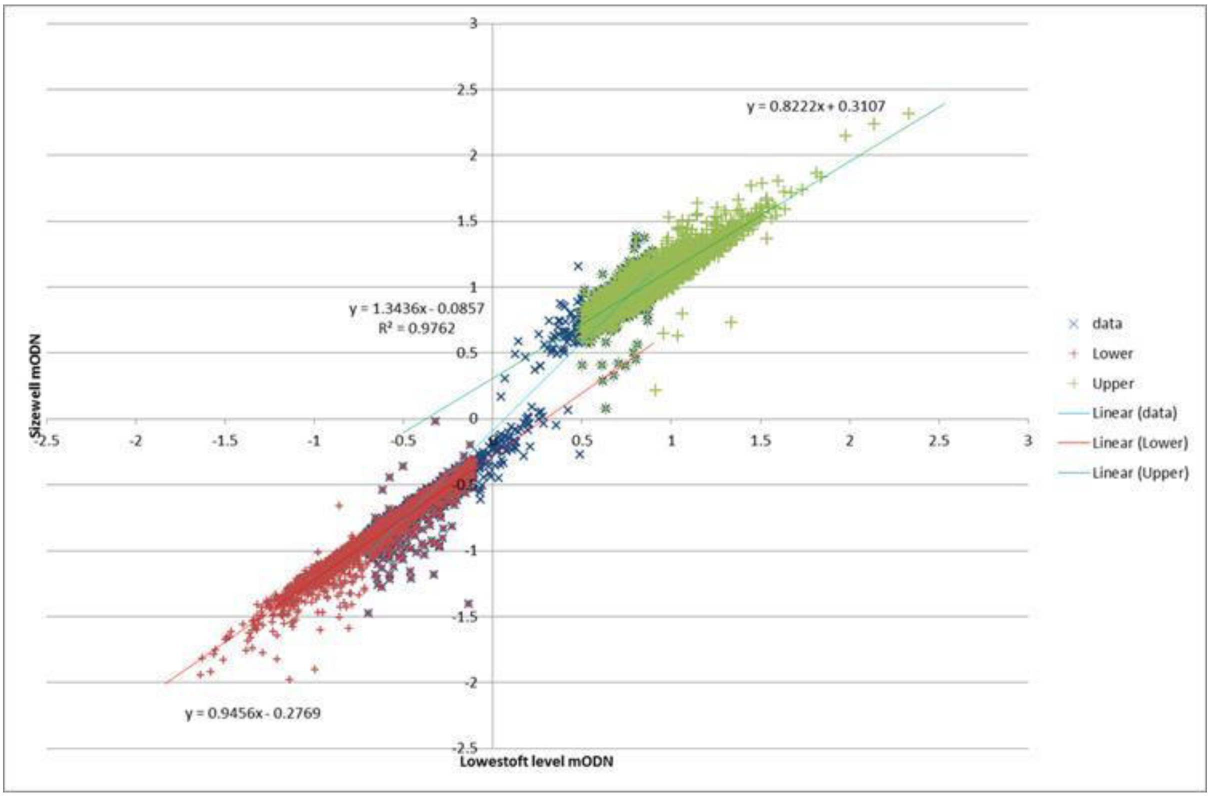

Figure 7 - Lowestoft and Sizewell B Tide Relationship

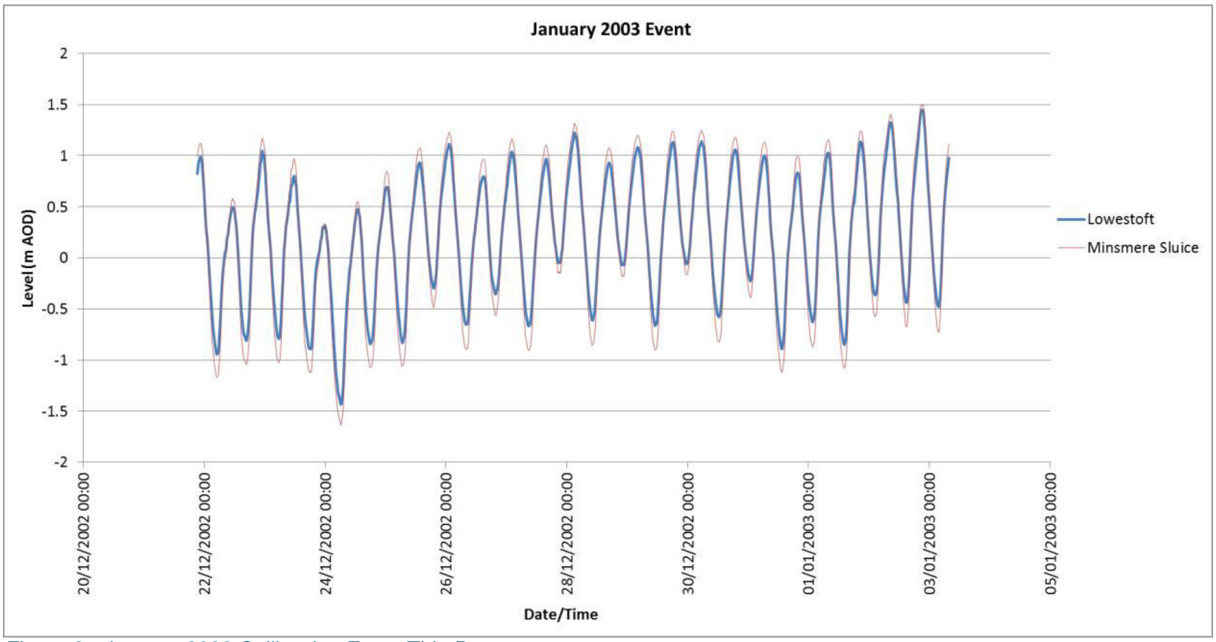

Figure 8 - January 2003 Calibration Event Tide Data

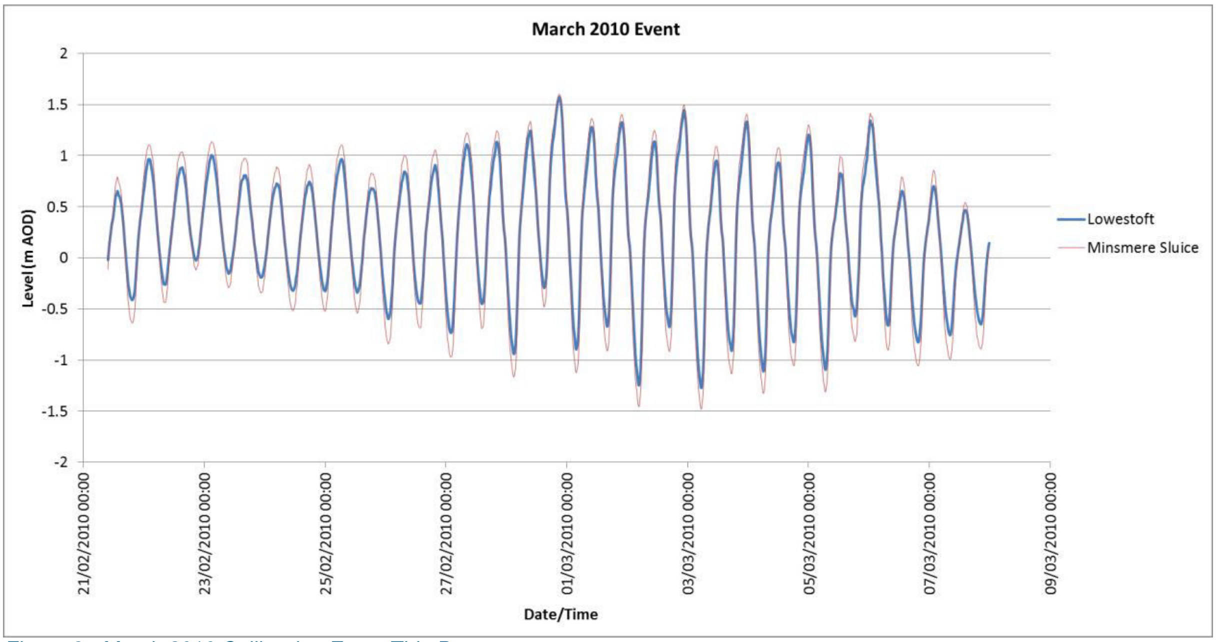

Figure 9 - March 2010 Calibration Event Tide Data

Royal HaskoningDHV
Enhancing Society Together

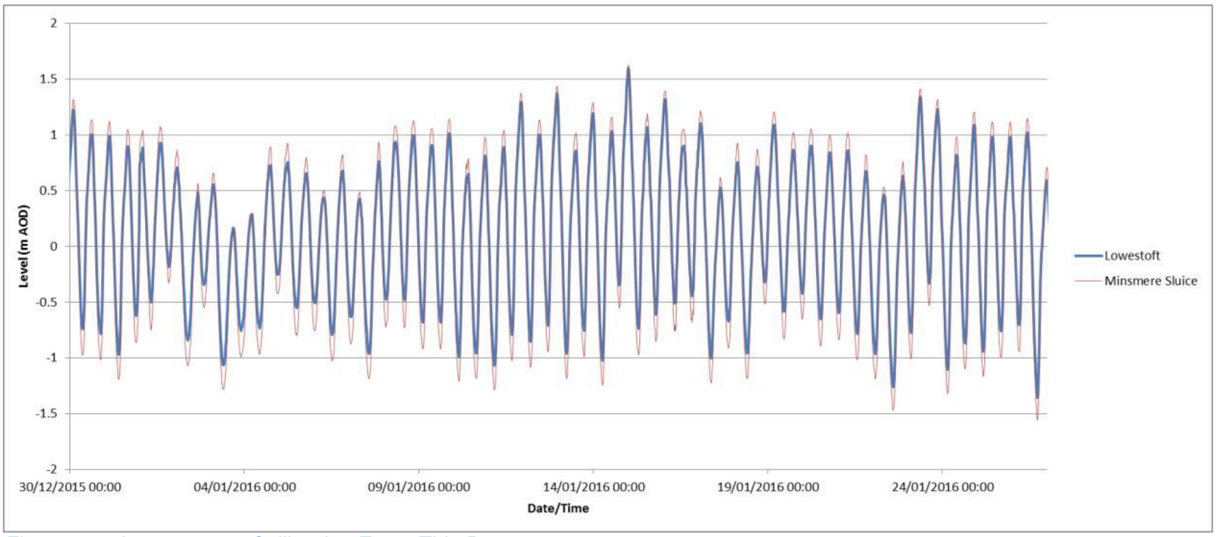

Figure 10 - January 2016 Calibration Event Tide Data

4. Improved Rating Model Flood Mechanism Observations

The inflow at Minsmere New Cut for the calibration events is derived using an improved rating for the Middleton Gauge. This improved rating is used to derive inflows from stage readings. A number of key findings and observations from the work undertaken to derive an improved rating at Middleton Gauge are salient to the calibration as they give greater confidence in the ability of the model to replicate flood behaviour in the study reach. The work undertaken to derive this improved rating and the findings and observations are therefore discussed here in summary.

The improved Middleton rating curve

The Environmental Agency initially provided a rating for the Middleton gauge. However this rating functions only for low flows and is seen as unreliable for out of bank flows. This rating could therefore not be used for the calibration of the fluvial model. Since there is no flow data available for Middleton (which represents 60% of the total catchment to the outfall) it is considered sensible to try to generate flow data. The best way to do this is by using the fluvial 1D-2D linked model to generate a new rating across the floodplain and through the channel at Middleton. Using the model for this rating is based on the assumption that the relationship between flow and water level is correct, which is an assumption that forms the base of hydraulic modelling. The model was run with a static tide boundary (0.00mAOD) and baseflow inputs in all other 1D elements of the model other than in the Minsmere New Cut. In the Minsmere New Cut an increasing flow was used (a 'stepped' hydrograph was utilised as shown in Figure 11).

The model was tested for a range of different roughness coefficients (0.045, 0.055, 0.065 and 0.085 between the model upstream limit and Dam Bridge) to determine sensitivity to this parameter. This sensitivity test shows that channel roughness has a significant impact on the flood mechanisms. A channel roughness of 0.065 has been chosen for the rating as this resulted in flood mechanisms most closely described by RSPB and others.

The new modelled rating provides a means of deriving a flow series from the water level series supplied by the Environment Agency. This allows analysation of the data (mindful of its limited accuracy) to calculate improved hydrological parameters such as TP, PR and Unit Hydrograph shape.

These parameters where then used to modify and improve the other sub-catchment inflows. The calibration runs undertaken used a Qt boundary for each event, i.e. used the rating to provide the flows to enter as the upstream boundary.

Using the derived rating curve is supported as it is considerably better than not using the data at all. Sensitivity testing will be carried out to assess the model's sensitivity to a range of hydraulic and hydrological parameters. The model is used for an FRA in which the 'before' and 'after' situation will be compared. Whilst we aim to calculate flows as accurately as possible, the definitive flow values used are therefore less important than any variation in water levels resulting from the development.

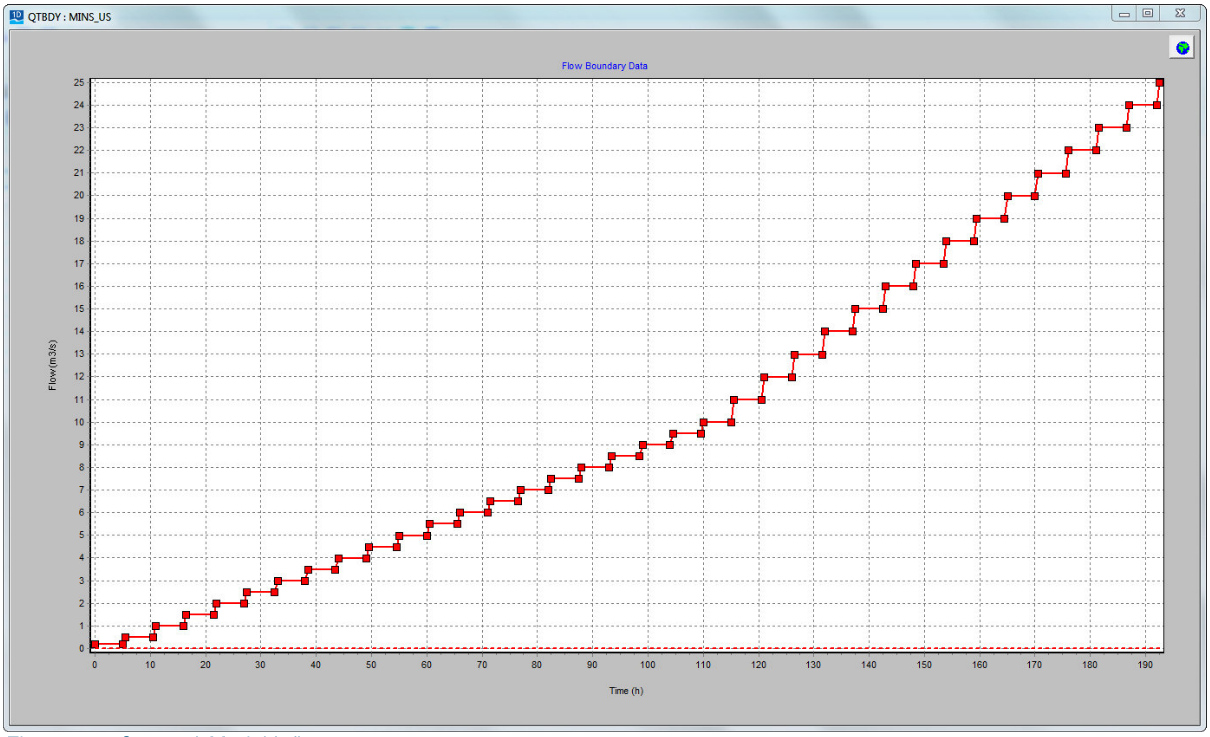

Figure 11 - 'Stepped' Model Inflow

Observations of the model animations indicated that for differing roughness coefficients, different flood mechanism occurred. Appendix A of this note provides a number of screen shots from the four model runs and a commentary of the differences between the models.

The key observations for the stepped hydrograph were:

1. The first flooding mechanism was observed for all roughness coefficients at the washland ~600-1,000m upstream of Minsmere Sluice on the right hand bank (as expressed in both commentaries by the RSPB)
2. The second flood mechanism in the lower roughness coefficient models runs (0.045 and 0.055) was water spilling from the Minsmere New Cut upstream of Dam Bridge into both Eastbridge Meadow (left hand bank) and Eastbridge Levels (right hand bank) upstream of Dam Bridge. Water spills into the Eastbridge Levels only and at a greater rate in the higher roughness coefficient scenarios (0.065 and 0.085). This second mechanism is also described in both RSPB commentaries.

3. The third mechanism in the lower roughness coefficient models runs (0.045 and 0.055) shows overtopping of the left hand bank opposite the washland was observed at or just after flooding occurring upstream of Dam Bridge, as described in the Black and Veatch RSPB commentary. For higher roughness coefficients this was observed at a later stage (however the model is run with a constant tidal boundary condition therefore this may be seen to occur earlier in all events if a tidal boundary was applied).
4. The fourth mechanism was observed to show water spilling into Meadow Marsh over the left hand bank of the Minsmere New Cut.
5. The fifth flooding mechanism observed from the models was water spilling from the right hand bank of IDB Drain #7 into the Southern Levels. Flooding into the southern levels was not observed from the Leiston drain (as described in the RSPB commentaries) probably due to the fact that a baseflow had been run in all other watercourses and a static tidal boundary was implemented which otherwise may have raised levels along its length.

A second set of model runs were undertaken with a hydrograph peaking at 30 cumecs (an event which would be significant in the Minsmere catchment: ~>1:100 years (to be determined)) with other similar boundary conditions to the former runs. In these models roughness coefficients downstream of Dam Bridge were also modified. The results of these four model runs are shown in Appendix B along with a commentary of the observations. A similar set of observations/flood mechanisms were seen from these models however the first and second observations from the previous model runs were reversed.

These findings indicate that roughness coefficient (density of vegetation/seasonality) can influence where water will spill from the Minsmere New Cut. Different flood mechanisms may therefore occur at different times of the year for similar magnitude/duration/intensity events. The findings also indicate that the model is able to simulate the observed flood mechanisms for the Minsmere New Cut.

5. Validation 2003 and 2010 events

A matrix of model runs was undertaken for the January 2003 and March 2010 validation events. The validation results for the January 2003 event are shown in Table 1 while those for March 2010 are shown in Table 2.

Table 1 - January 2003 Run Matrix

	High Roughness (0.060-0.065)		**Low Roughness (0.035-0.045)**
	50% PR	**40% PR**	**50% PR**
0.00m AOD Initial Water Level	X	X	X
0.20m AOD Initial Water Level			X
0.50m AOD Initial Water Level	X	X	X

Table 2 - March 2010 Run Matrix

	High Roughness (0.060-0.065)				**Low Roughness (0.035-0.045)**			
	50% PR	**40% PR**	**50% PR Lowestoft Tide Data**	**40% PR Lowestoft Tide Data**	**50% PR**	**40% PR**	**50% PR Lowestoft Tide Data**	**40% PR Lowestoft Tide Data**
0.00m AOD Initial Water Level								
0.20m AOD Initial Water Level	X	X	X	X	X	X	X	X
0.50m AOD Initial Water Level								

In order to put the magnitude of the events into context, Figure 12 shows the Middleton gauge stage record. Assuming all gauging shown is reliable; the January 2003 event is one of the largest events on record. The peak stage recorded for January 2003 was 1.347m, relating to a level of 2.877mAOD (this is 0.199m higher than that observed in the March 2010 event).

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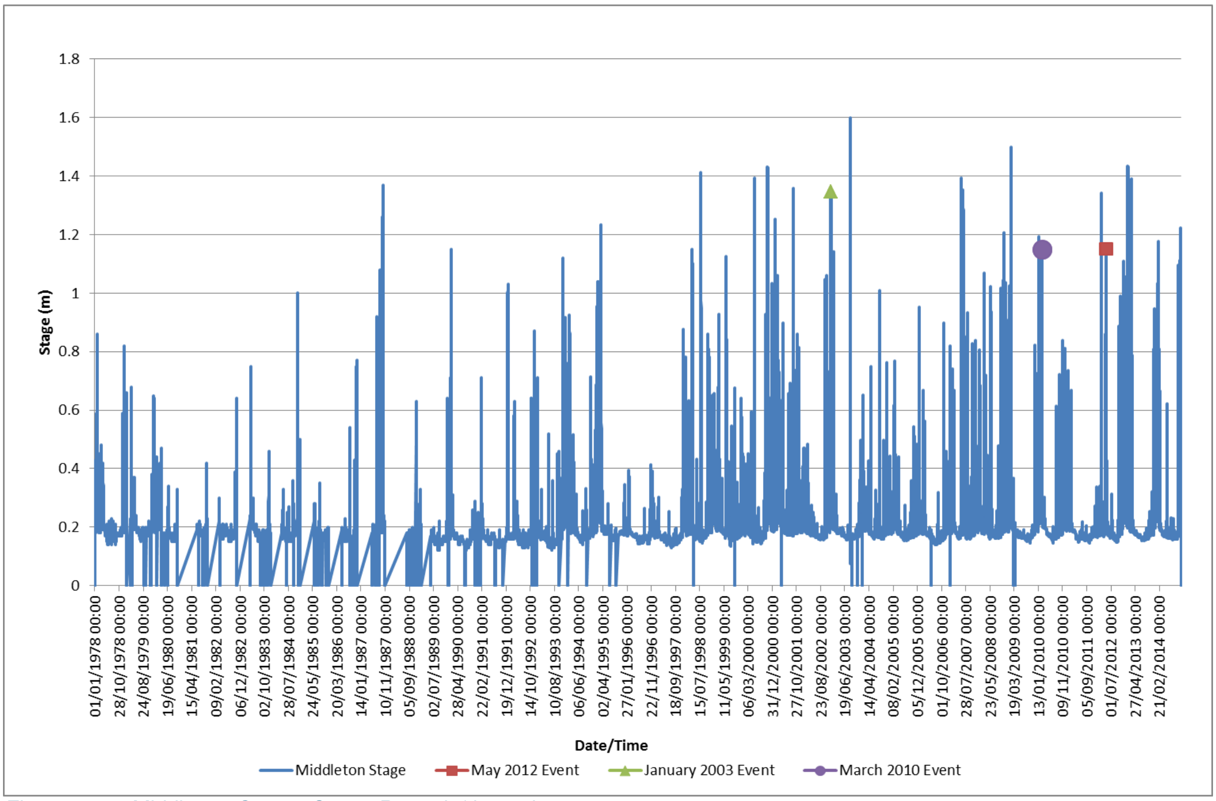

Figure 12 - Middleton Gauge Stage Record (the points represent the selected model run events)

The May 2012 event is a similar magnitude event (based on stage at Middleton) as that of March 2010. Figure 13 (source RSPB website) shows flood conditions on the Minsmere New Cut during the May 2012 event (timing of photograph relative to the peak not known) opposite the washland. It would therefore be reasonable to expect given the magnitude of the validation events, that similar flood mechanisms, as has been observed in May 2012 (as photo below) and can be expected in the simulation results of the January 2003 and March 2010 events.


Figure 13 - May 2012 Observed Flooding into the RSPB Reserve (Source http://www.rspb.org.uk/community/placestovisit/minsmere/b/minsmere-

A number of sites in the RSPB reserve and South Levels have been selected to undertake the analysis of modelled and observed changes in water depth in the floodplain. Figure 14 shows the five locations selected. These points have been selected based on the availability of data and the appropriateness of each point to represent larger areas/discreet compartments of the floodplain. The results (changes in water depth) of the January 2003 validation runs can be seen in Figure 15 to Figure 19. The naming of the results indicate the roughness coefficients (high or low 'n'), Percentage Runoff used (40 or 50%) and the initial condition (water level) in the hydraulic model (IC0, IC2 and IC5, 0.00, 0.20 and 0.50m AOD). The change in water depth has been generated by utilising RSPB gauge board level information. The change in depth has been calculated relative to the recording prior to the start of the rainfall event for each validation. It has been assumed that recording have been taken at 12:00 midday on the day recorded in the data. Table 3 shows the RSPB recorded data for January 2003.

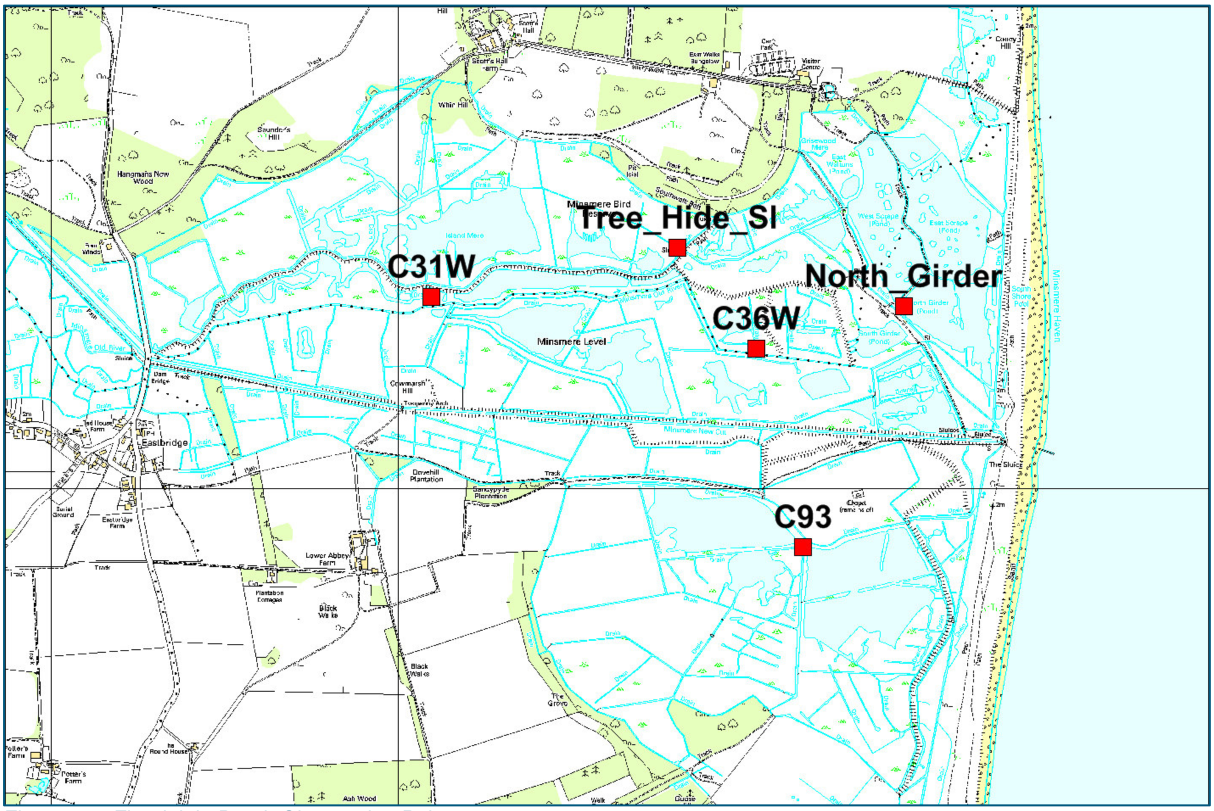

Figure 14 - Floodplain Depth Observation Points

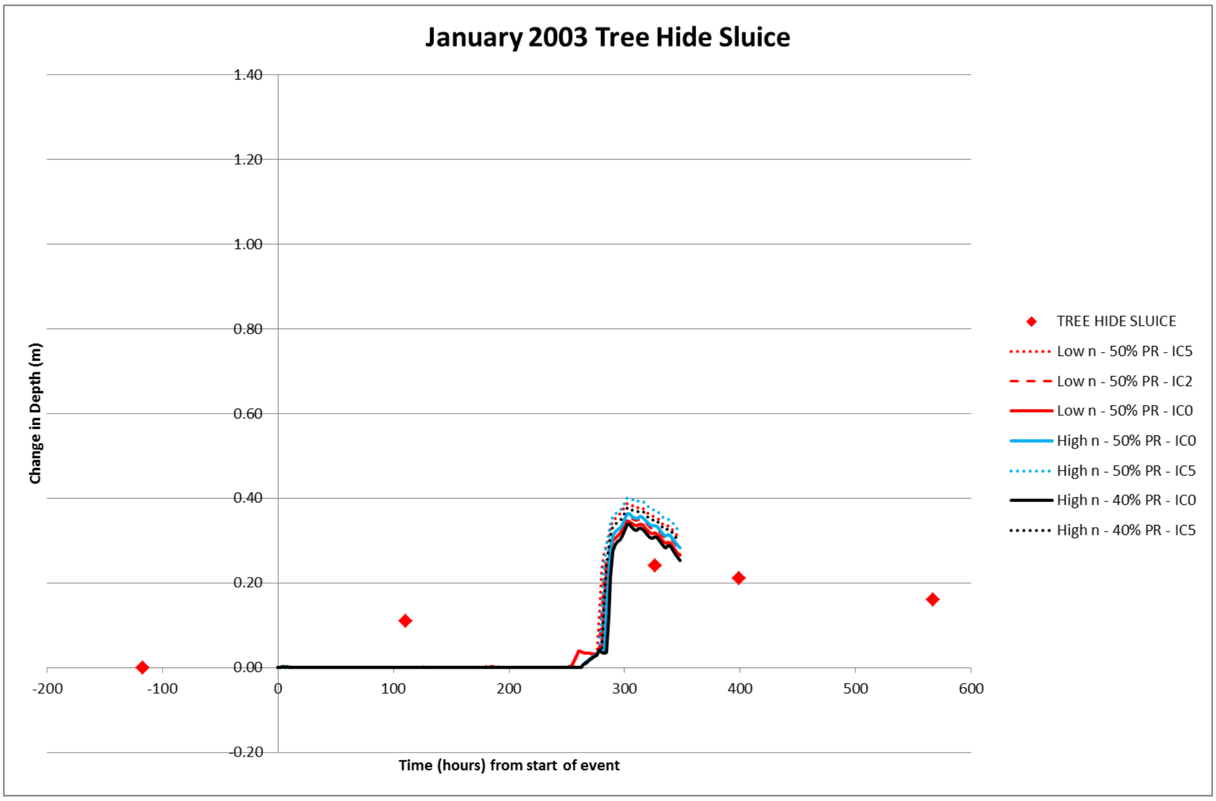

Figure 15 - January 2003 Tree Hide Sluice Change in Modelled and Observed Water Depth

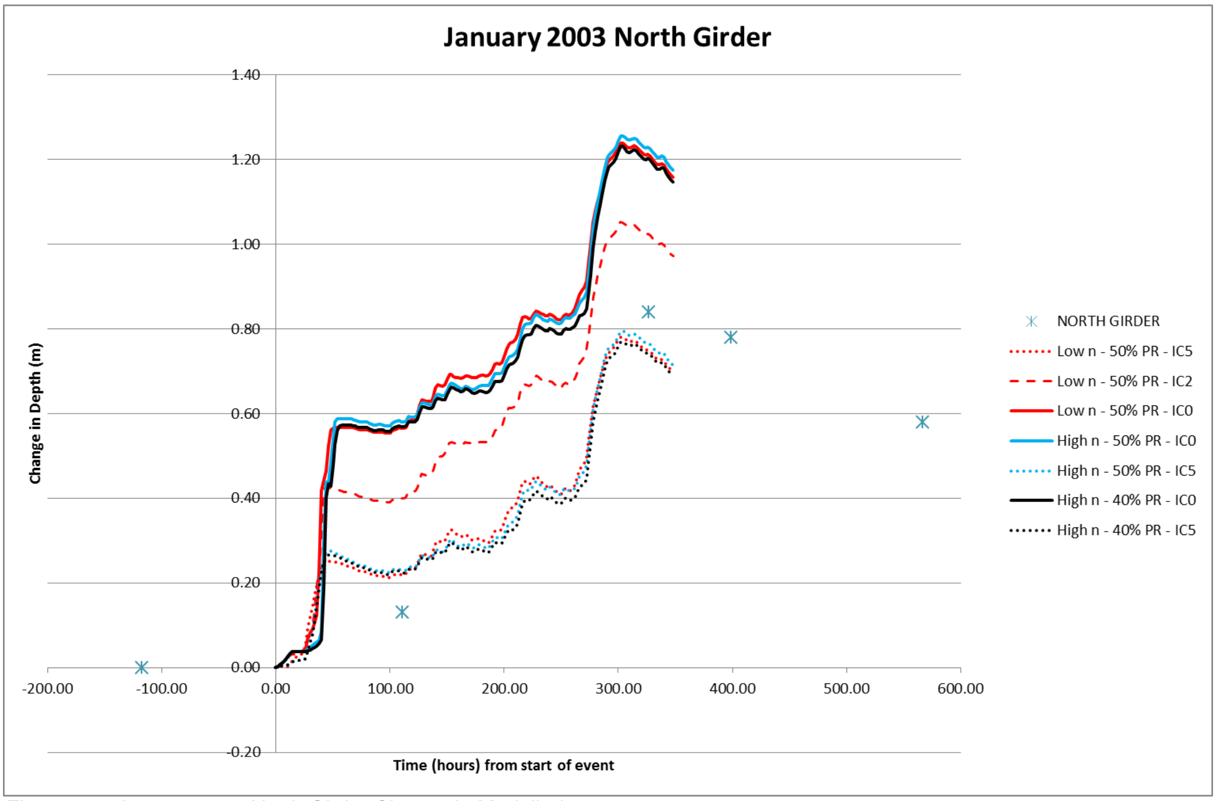

Figure 16 - January 2003 North Girder Change in Modelled and Observed Water Depth

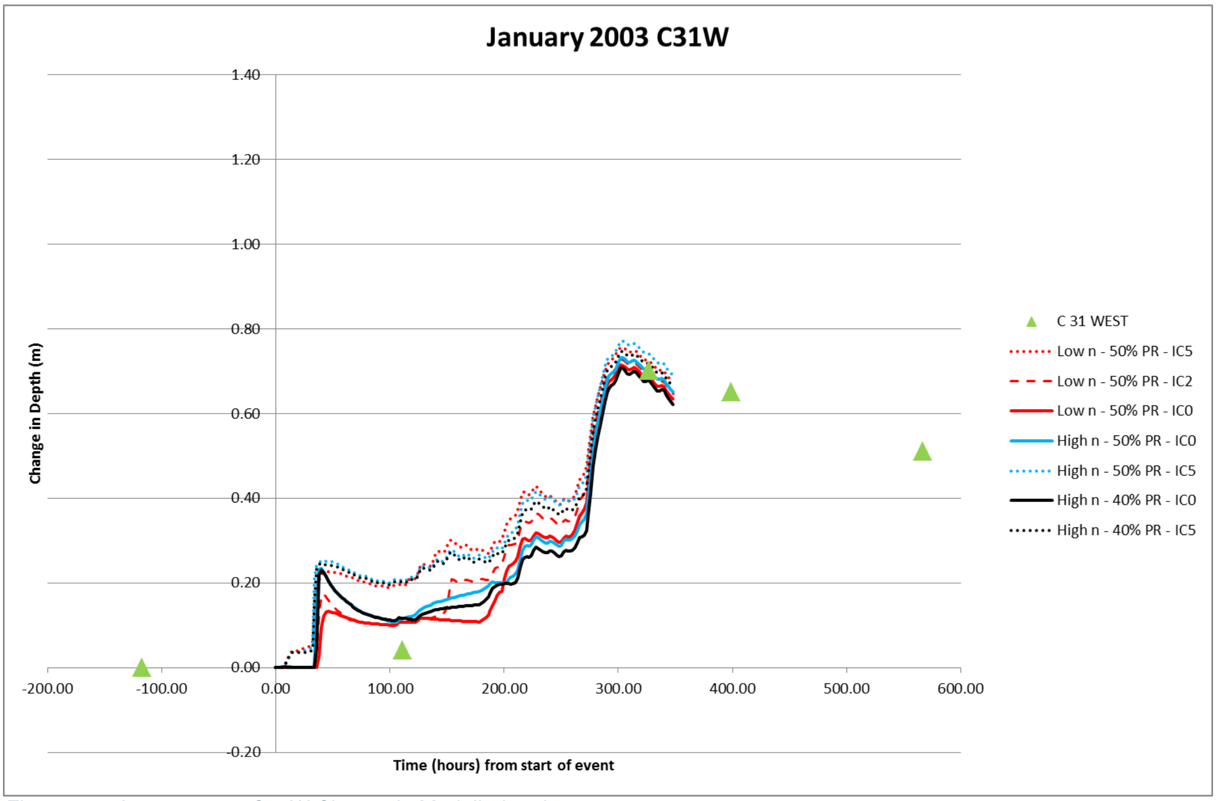

Figure 17 - January 2003 C31W Change in Modelled and Observed Water Depth

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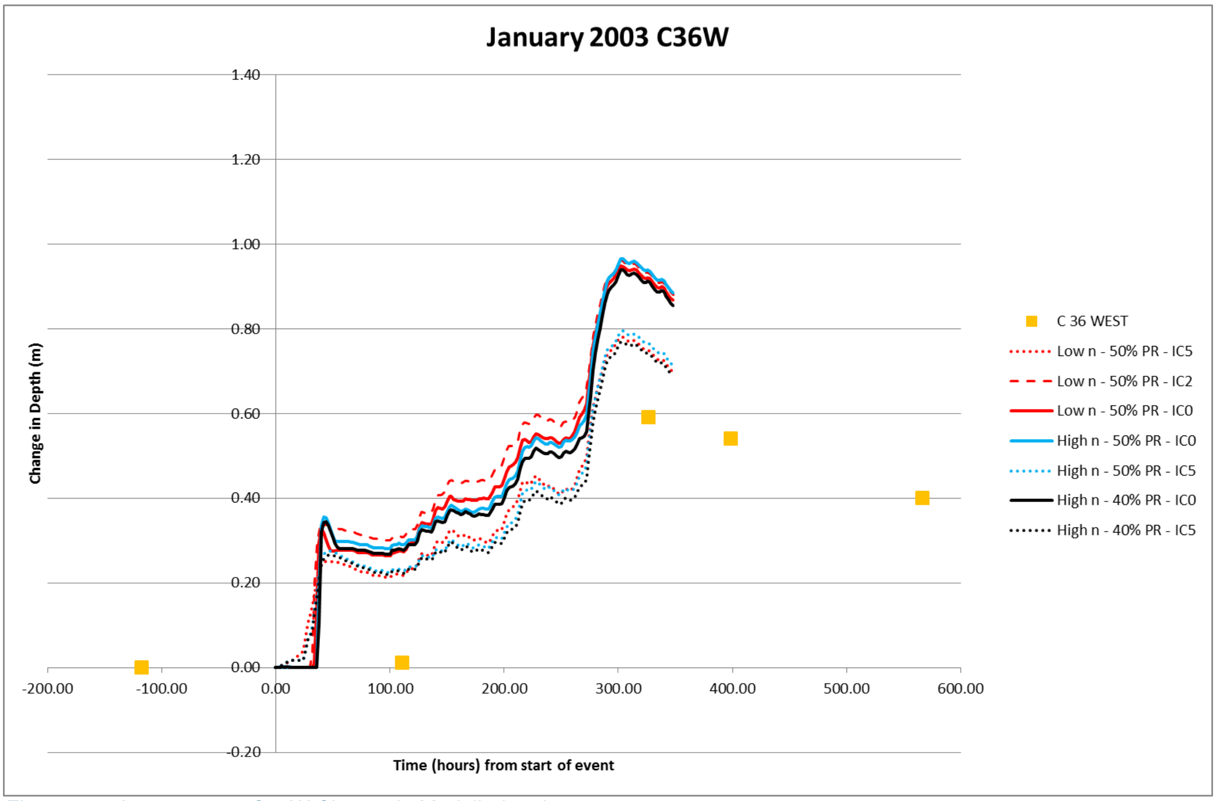

Figure 18 - January 2003 C36W Change in Modelled and Observed Water Depth

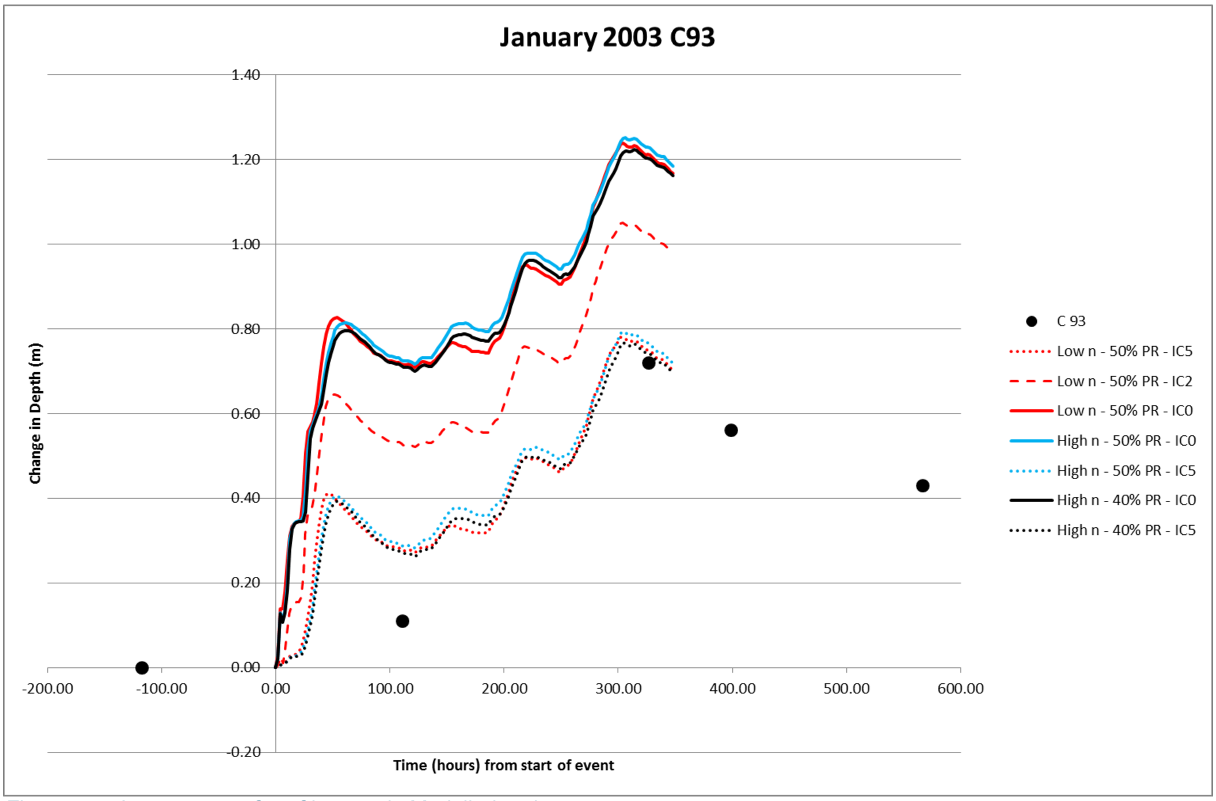

Figure 19 - January 2003 C93 Changes in Modelled and Observed Water Depth

Table 3 - January 2003 RSPB Recorded Gauge Levels

		Level Relative to Gauge Board				
DATE	WEEK NO.	TREE HIDE SLUICE	NORTH GIRDER	C 31 WEST	C 36 WEST	C 93
03-Dec-02	49	1.36	0.53	0.88	0.69	0.72
10-Dec-02	50					
17-Dec-02*	51	1.46	0.38	0.55	0.58	0.73
27-Dec-02	52	1.57	0.51	0.59	0.59	0.84
04-Jan-03	1	1.7	1.22	1.25	1.17	1.45
07-Jan-03	2	1.67	1.16	1.2	1.12	1.29
14-Jan-03	3	1.62	0.96	1.06	0.98	1.16
21-Jan-03	4	1.61	0.84	0.87	0.84	1.06

* Date from which all changes of depth are relative to (recording immediately before the start of calibration rainfall)

There are a number of observations that can be concluded from the results of the January 2003 validation. The modelled changes in flood depth exhibit a positive correlation with the observed changes in depth. This is particularly apparent for the results from the models utilising an initial water level of 0.5mAOD. For some of the results (C93 and North Girder), there is a marked difference in the magnitude of the change in depth between the 0.00 and 0.50mAOD initial water level models. The maximum water level in the RSPB reserve varies by only 45mm across all seven model scenarios; therefore this explains why the magnitude is greatest where the initial water levels are lower. Despite models being run with differing roughness coefficients, the observations show that there is little difference in flood depths/levels within the RSPB reserve in this large magnitude (in respect to the gauge record length) event and therefore the peak modelled water levels are not sensitive to this parameter.

Observed flood mechanisms from the various January 2003 models are similar to those observed in the rating improvement model runs. Channel roughness coefficient influences the location and timing of water spilling from the Minsmere New Cut; the model runs with a lower roughness coefficient show a better correlation between the previously observed mechanism and the modelled outputs (water spilling opposite the washland). Selected outputs from the model runs can be observed in Appendix C.

Figure 20 to Figure 24 show a comparison between the observed and modelled changes in flood depth within the RSPB reserve during the March 2010 event.

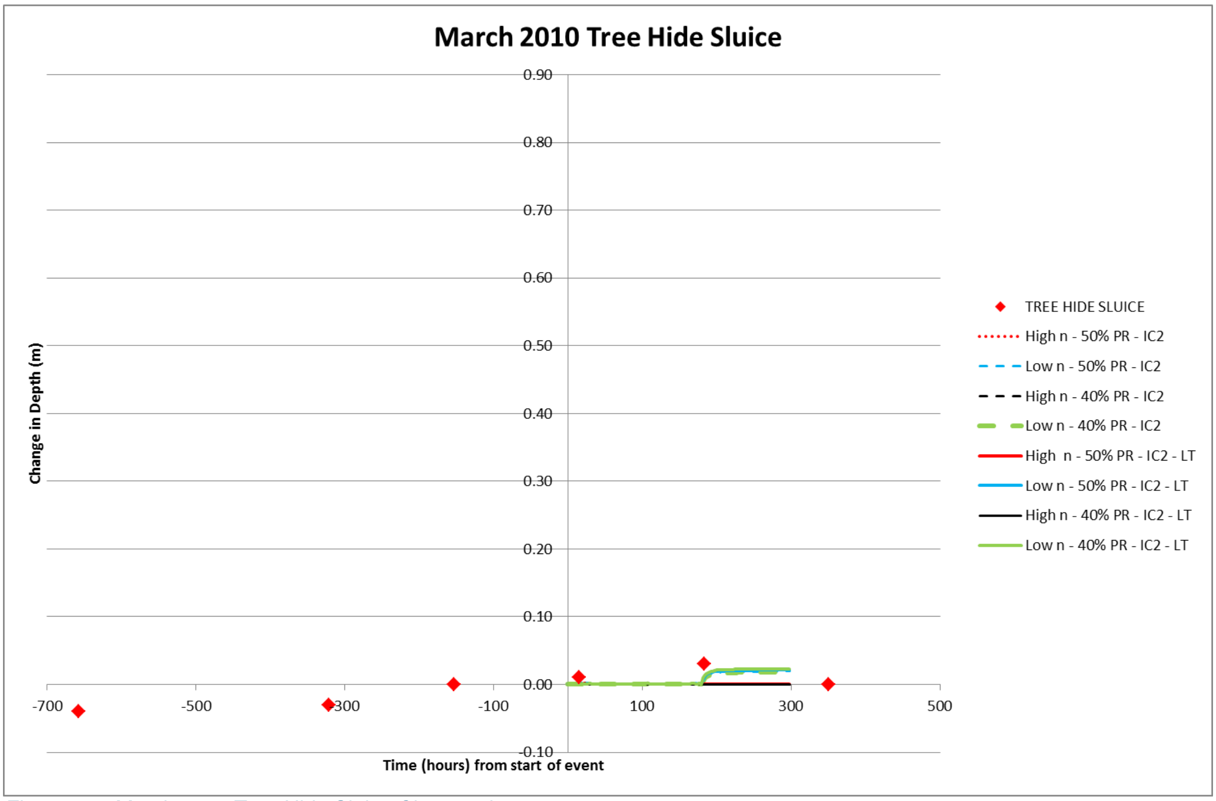

Figure 20 - March 2010 Tree Hide Sluice Changes in Modelled and Observed Water Depth

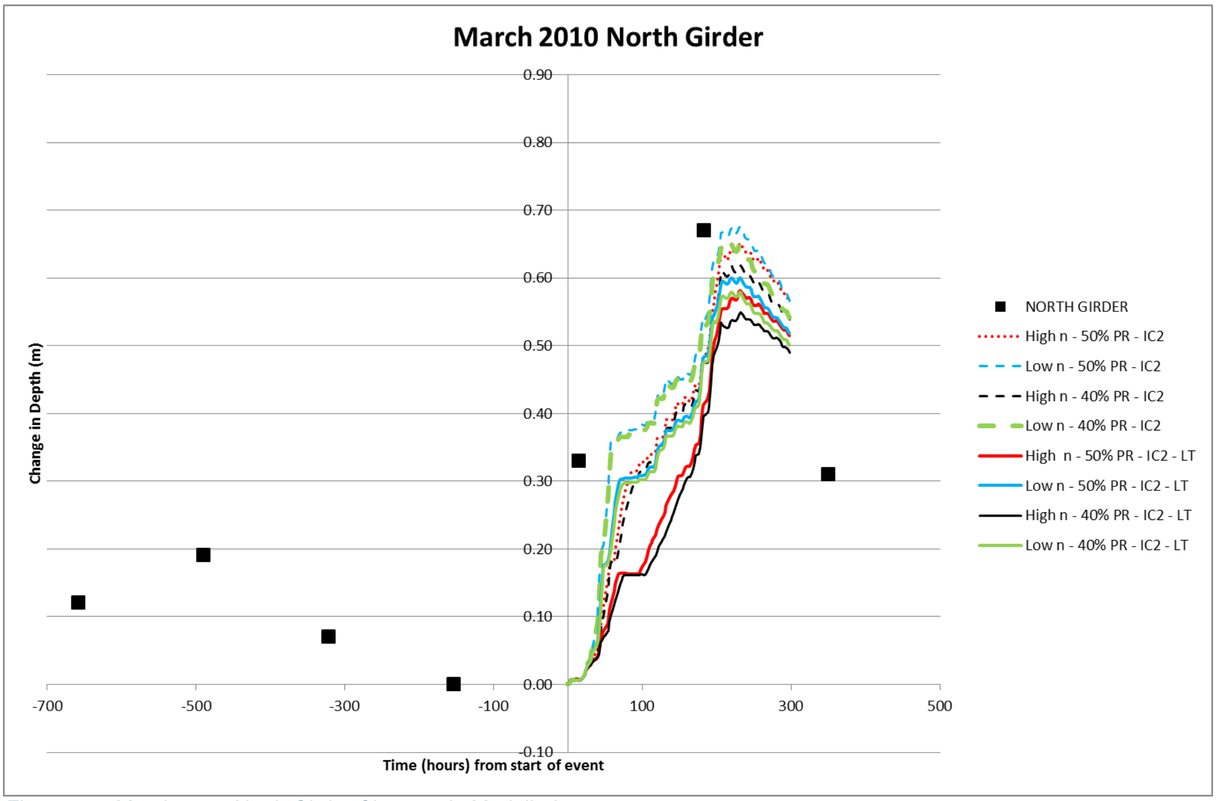

Figure 21 - March 2010 North Girder Changes in Modelled and Observed Water Depth

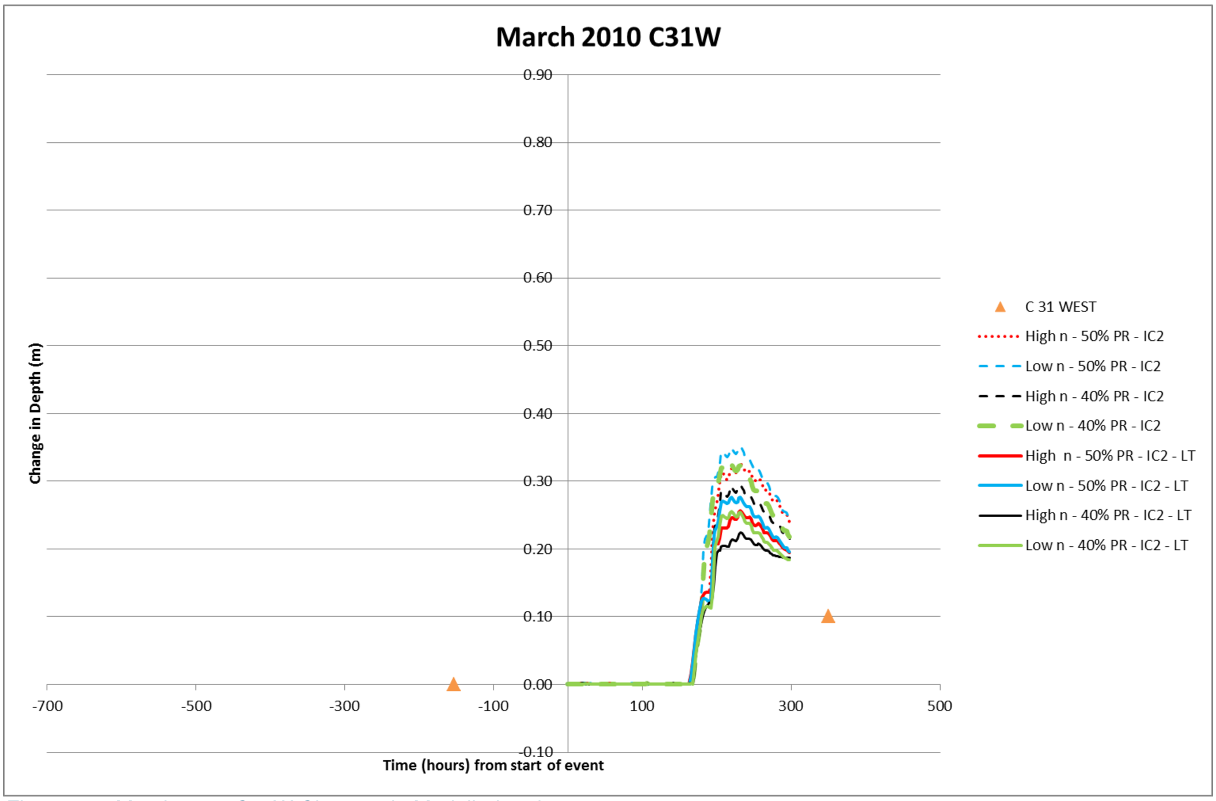

Figure 22 - March 2010 C31W Changes in Modelled and Observed Water Depth

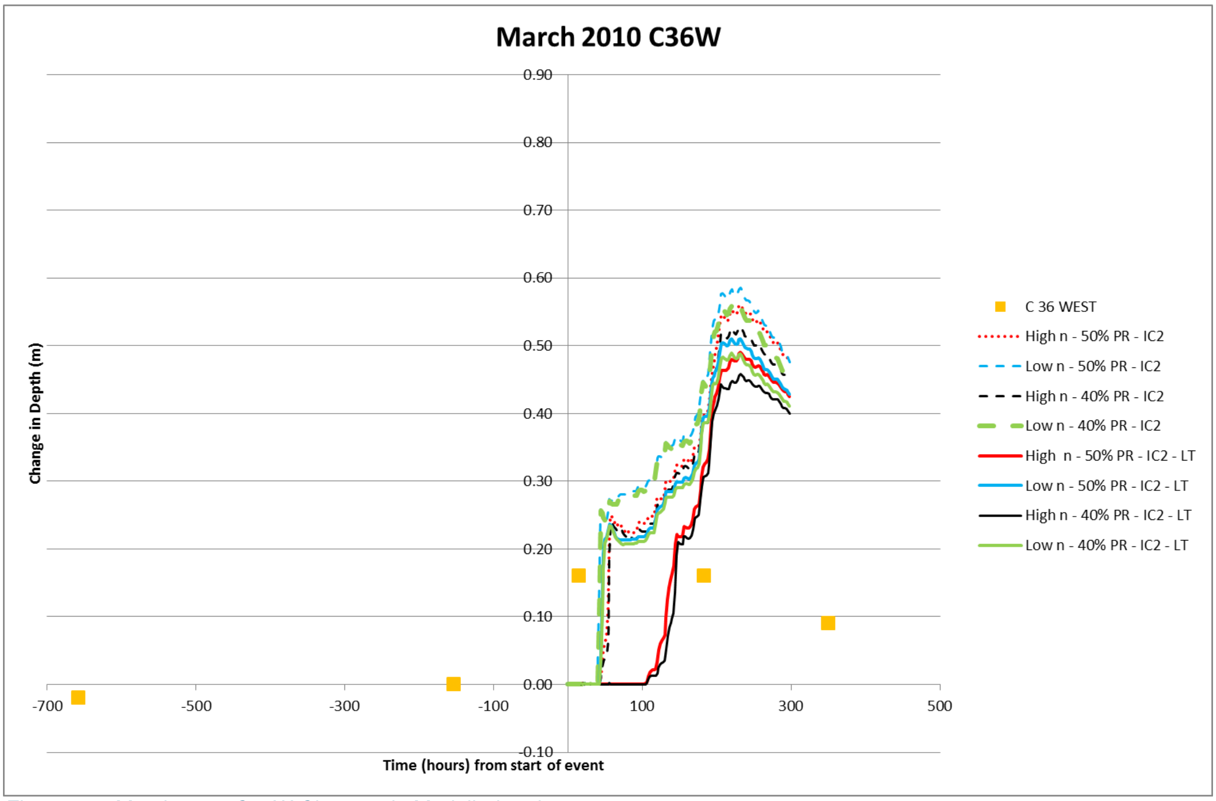

Figure 23 - March 2010 C36W Changes in Modelled and Observed Water Depth

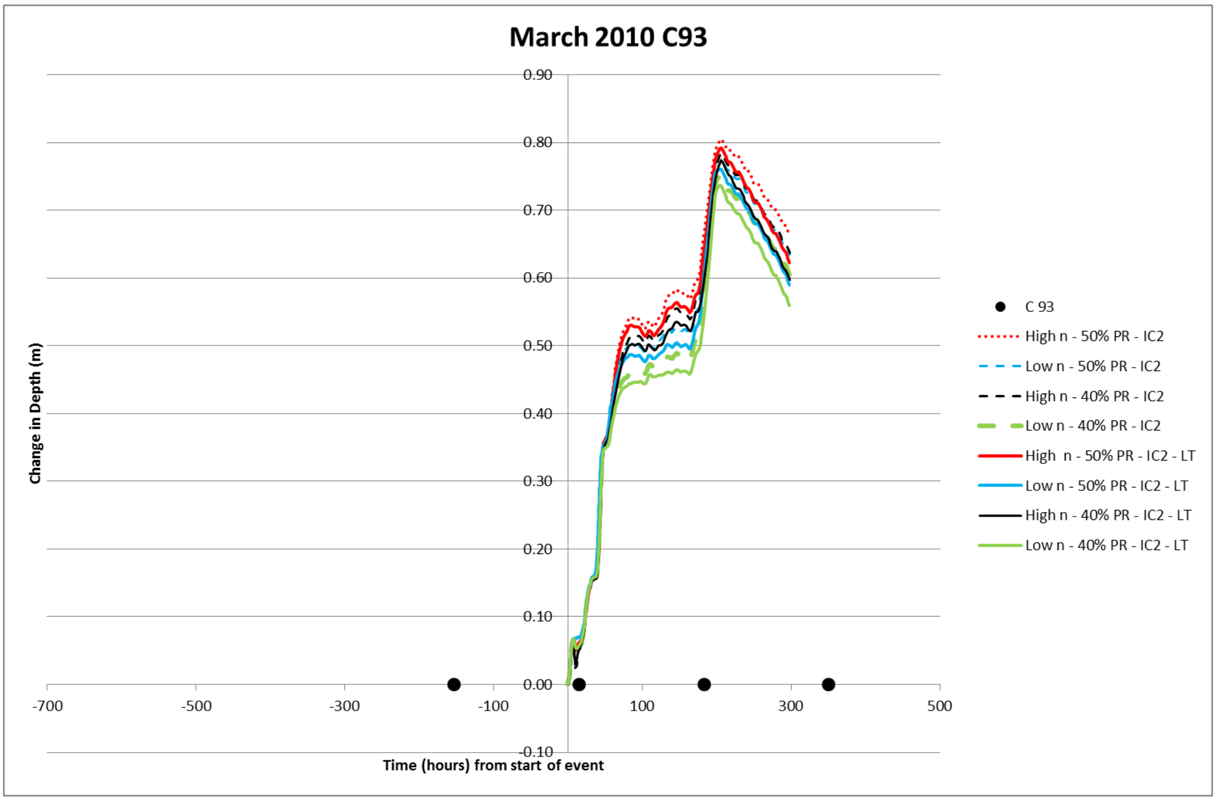

Figure 24 - March 2010 C93 Changes in Modelled and Observed Water Depth.

The observed data at all points other than North Girder indicate that there was little observed variance in water levels during the event; this has resulted in a variance in the correlation between the observed and modelled results. The results for Tree Hide Sluice and North Girder give a reasonable correlation between the observed and modelled changes in flood depth. It is however highlighted that results for this event is for the 0.20mAOD initial water level and therefore it is expected that the modelled change in flood depth would be less if a 0.50mAOD water level was utilised.

Observations of the RSPB water level recordings (Table 4) indicate that initial water levels for many locations in the reserve were high for a long period of time prior to the beginning of the rainfall event which is in contrast to the findings of the January 2003 event. These high water levels may be a result of a series of previous rainfall activity or water level management within the reserve.

The greater variance between modelled and observed changes in depth may be a result of the practicalities of undertaking analysis of changes in depth in the absence of absolute water level data. Maximum modelled water levels in the North Levels utilising the transformed tide from Lowestoft to Minsmere Sluice vary between 0.82 and 0.88mAOD depending on which roughness coefficient and percentage runoff is used. The results from models utilising the Lowestoft tide data indicate that water levels decrease in the North Levels by up to 80mm (maximum level 0.75 to 0.80mAOD). The model results are therefore not sensitive to relatively minor changes to tidal boundary condition, roughness coefficient and percentage runoff.

Table 4 - March 2010 RSPB Recorded Gauge Levels

		Level Relative to Gauge Board (m)				
DATE	**WEEK NO.**	**TREE HIDE SLUICE**	**NORTH GIRDER**	**C 31 WEST**	**C 36 WEST**	**C 93**
26-Oct-09	43	1.36	0.38	0.31	0.56	0.67
02-Nov-09	44	1.42	0.40			
09-Nov-09	45	1.38	0.43		0.60	
16-Nov-09	46	1.40	0.46	0.44	0.63	0.79
23-Nov-09	47	1.39	0.47	0.50	0.63	0.80
30-Nov-09	48	1.46	0.54		0.72	0.90
07-Dec-09	49	1.50	0.55	0.64	0.73	
14-Dec-09	50	1.44				
21-Dec-09	51	1.44	0.61			
28-Dec-09	52		0.65			
04-Jan-10	1	1.44		0.73		
11-Jan-10	2	1.48	0.62	0.64	1.00	0.94
18-Jan-10	3	1.55	0.93	0.85	1.00	1.00
25-Jan-10	4	1.60	0.65		0.82	
01-Feb-10	5		0.72			
08-Feb-10	6	1.61	0.60			
15-Feb-10*	7	1.64	0.53	0.56***	0.84	1.00****
22-Feb-10	8	1.65	0.86		1.00	1.00
01-Mar-10	9	1.67	1.20**		1.00**	1.00**
08-Mar-10	11	1.64	0.84	0.66	0.93	1.00

* Date from which all changes of depth are relative to (recording immediately before the start of calibration rainfall)

** RSPB indicate that water level was above the gauge board therefore these levels are estimates

*** RHDHV assumed level. Level taken as same as C31E as both readings are similar throughout the record

**** RHDHV assumed level. Level estimated from other gauges in south levels.

Flood mechanisms in the March 2010 event are similar to those observed in previous events however the first two mechanisms are those of the filling of the washland and then spilling on the left hand bank opposite the washland as described in the 2006 RSPB commentary.

The results indicate a good validation between recorded and modelled changes in water levels; moreover the model is shown to replicate the timing/ordering of observed mechanisms indicated by the RSPB (although there are no recorded observations for these two specific events). Although there have been limitations in the data to undertake the calibration for large events, the analyses undertaken have shown that the model is able to represent the system's behaviour well which makes the model suitable for use in determining flood risk in the catchment.

These results have been discussed with the RSPB; the minutes from a meeting held on 25th April 2016 are included in Appendix D.

Minsmere sluice

Minsmere sluice is the most important structure in the system and controls the amount of water leaving (and entering) the system. Minsmere sluice underwent renovation works in 2012. The renovation works took place due to the poor condition of the sluice structure and corroded and inefficient sluice gate function (Figure 25). Although it was estimated that the condition of the sluice did not affect the operation of the sluice, pre-renovation data might be affected by the holes and corrosion in the valves and sluice gates or siltation of the culverts.

Figure 25 - condition of the Minsmere sluice pre (up) and post (down) renovation works in 2012 (source: http://www.rspb.org.uk/community/placestovisit/minsmere/b/minsmere-blog/archive/2014/04/10/minsmere-sluice-update-work-now-complete.aspx).

For the period 2005-2006 there is a limited dataset available for Minsmere sluice comprising water levels near the sluice (Minsmere New Cut, Scotts Hall drain). As the structure has a crucial role on the water levels in the system it is worth trying to use this dataset. This chapter describes the applicability of the data.

Figure 26 shows the water level at Middleton, which is a good indicator for the amount of water flowing through the Minsmere New Cut arriving at Minsmere Sluice a while later. It also shows the selected calibration events (blue circles). The period of data available for Minsmere sluice (2005-2006) is shown by the yellow box. The figure shows that the selected events for calibration are in the high peak level range compared to the peak level within the period of data availability for Minsmere Sluice. It is expected that the peak level in the period 2005-2006 is even below bank full. Apart from the data being also pre-renovation, it was concluded that this data is not suitable for calibration of the FRA model because of no overlap with other stations and representing only in-bank flow conditions.

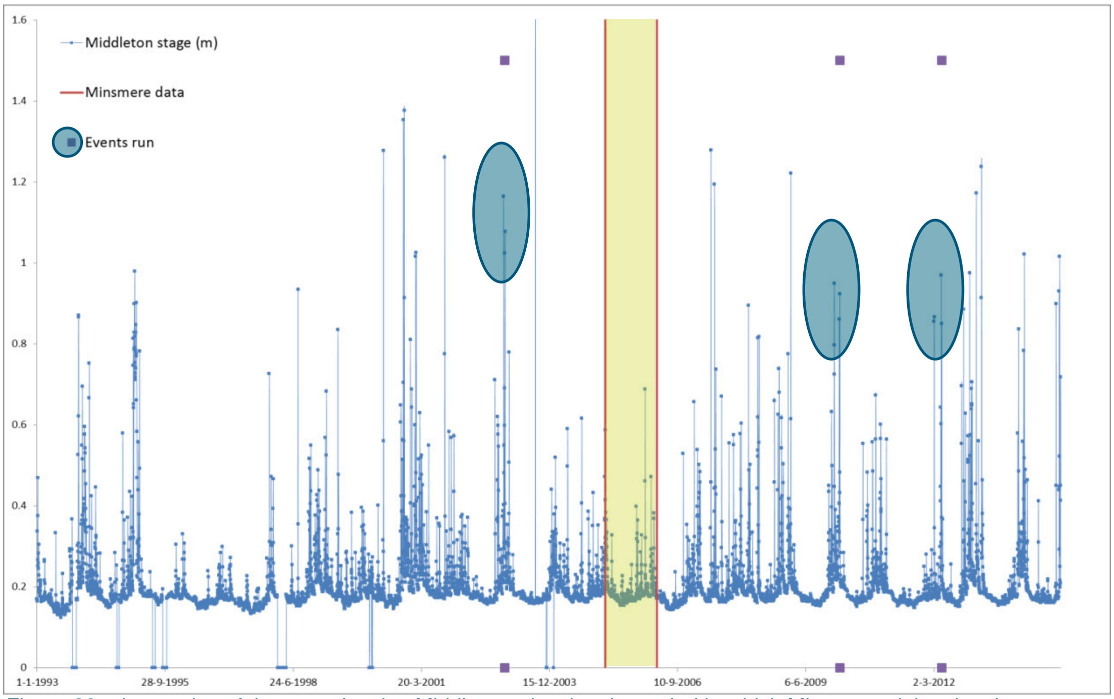

Figure 26 - time series of the water level at Middleton, showing the period in which Minsmere sluice data is available.

The data around Minsmere sluice can however be used to provide better system understanding of the water system around the sluice, and in particular the interaction of water levels in each of the chambers (north, south and Minsmere New Cut). An analysis has been carried out on the water levels measured in Minsmere New Cut and Scotts Hall drain (north) and the water levels measured upstream at Middleton (Figure 27). The figure shows that the water levels downstream at Minsmere sluice follow the upstream water levels at Middleton with a slight delay. The water levels at Scotts Hall Drain show no response to the water levels at Middleton. Since Scotts Hall Drain is only connected with Minsmere New Cut above bank full conditions, this indicates that the flow remained in-bank for the period 2005 -2006 as indicated in the figure below.

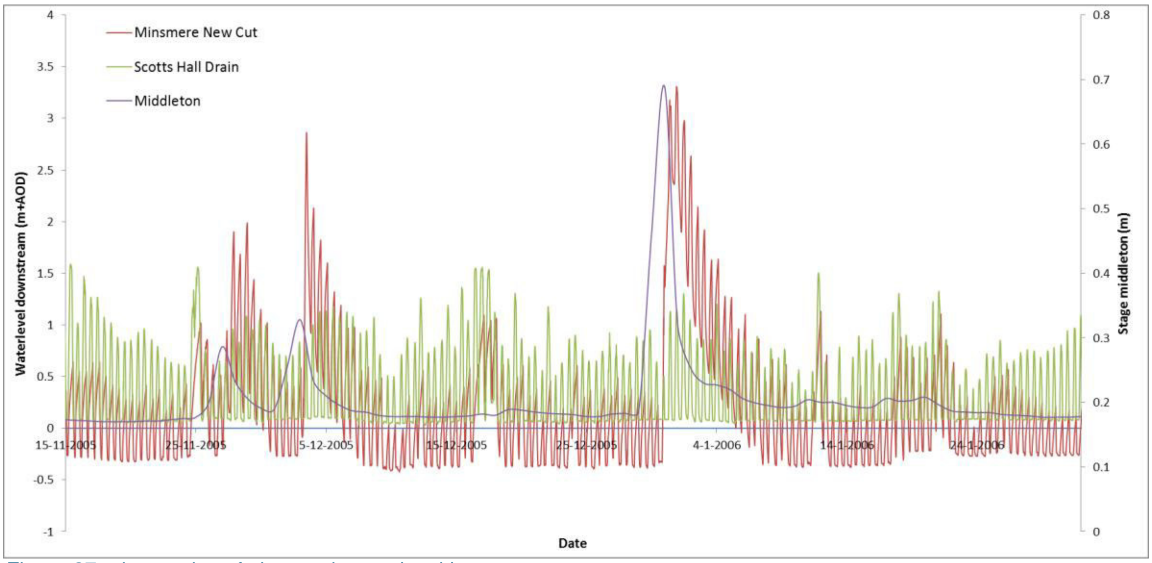

Figure 27 - time series of observed water level in Minsmere New Cut, Scotts Hall drain and Middleton (left axis).

6. Calibration 2016 event

After having run the 2003 and 2010 calibration runs for the Minsmere system, the Leiston system remained uncalibrated due to missing data for these periods. For January 2016, sufficient data is available for most gauges in Leiston system (see Appendix F) in order to execute calibration. The other benefit is the only dataset representing the post-renovation period of Minsmere Sluice, providing better representation of the sluice's capacity. The following adjustments and sensitivity checks have been made to the model schematisation as part of the calibration process:

- Adjustment of the initial conditions (spatial varying)
- Inclusion of a weir to mimic obstruction in the Leiston drain as observed in the data between adjacent gauge stations.
- Reduction of lateral inflow to isolate the impact of backwater from Minsmere.

One important sensitivity check was the performance of Minsmere sluice under varying operational conditions. This check was possible for 2016 since detailed data for Leiston drain was available enabling a validation of the opening of the southern chamber of the sluice. Other sensitivity checks have been executed but these checks did not provide relevant information for improving the calibration.

As the January 2016 event is a recent occurrence, there are additional locations at which modelled and observed data can be compared; these are at the temporary gauges on the Leiston system. Figure 28 shows the location of the temporary gauges. The results of the simulations are shown in Figure 32 - Figure 34 representing stations G4, G1, G6 and G8 (in order from upstream to downstream).

Appendix F shows the data availability for Minsmere system for 2016. Although for most stations data is available, most measurements miss the flood peak and can therefore not be used for calibration.

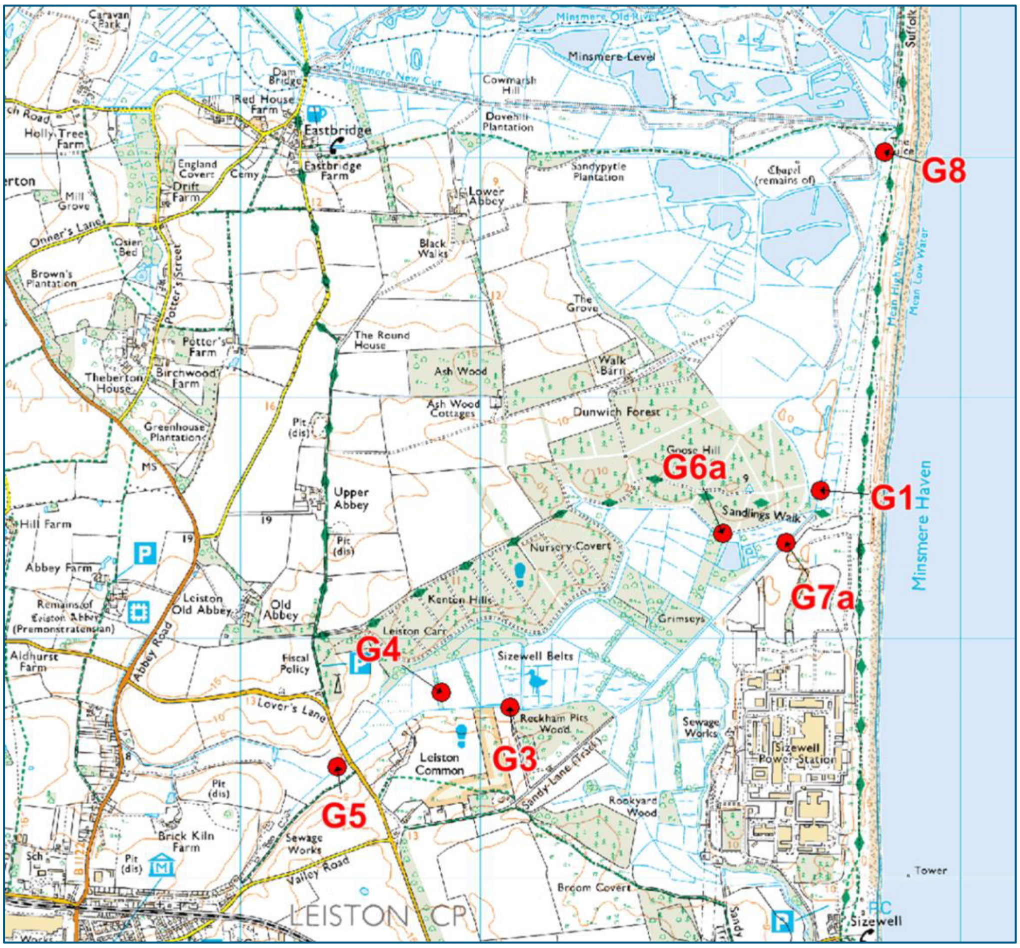

Figure 28 - Temporary Gauge Locations

Sensitivity check opening Minsmere Sluice

Subsequent to the results of the 2003 and 2010 calibration runs being presented to the RSPB and Environment Agency, a number of model runs for the January 2016 event have been undertaken. Evidence has been provided by the Environment Agency that following discussions with the RSPB, the sluice on the southern culvert of the Minsmere New Cut at Minsmere sluice was opened on 4th January 2016. The modelling has assumed that this has opened to allow a $1m^2$ opening into the southern chamber. This opening size has been assumed from as built drawings; evidence of the absolute opening aperture during the event was not provided. It has been assumed that the sluice was opened at 9:00 AM which is 110 hours into the model run. The model has been modified to include a sluice unit to open at this time during the event.

Appendix A shows a comparison of observed and modelled changes in depth within the RSPB reserve and South Levels. Since observations during peak flow are missing, the results cannot be used for further interpretation. The model run matrix for the January 2016 event is shown in Table 5, the initial water level is set at 0.0 m+AOD which is in line with the observed level at station G8. For each run a 50% percentage runoff to the upland inflows was used.

Table 5 - January 2016 Run Matrix

	Low Roughness (0.035-0.045)		High Roughness (0.060-0.065)	
Southern Culvert (SC)	SC Open	SC Closed	SC Open	SC Closed
0.00m AOD Initial Water Level	X			
0.50m AOD Initial Water Level	X	X		X

The results show that at North Girder and C36W the opening of the sluice has reduced the change in depth compared to the model run without opening the sluice. The opening of the sluice has drawn down levels in the Minsmere New Cut which reduces the volume of water that spills north into the reserve. For these two gauges the maximum change in flood depth is similar to that recorded by the RSPB. Modelled water levels at Tree Hide Sluice have not increased by any significant amount as it fills only through direct rainfall and water has not been observed to flow from the river into this area of the floodplain. Similar observations were seen for the other calibration events. This is likely to be due to simplifying the schematisation of the numerous water level control structures within the RSPB reserve which has previously been stated as a simplification of the system. Results for all the model runs are similar for C93 in the Southern Levels.

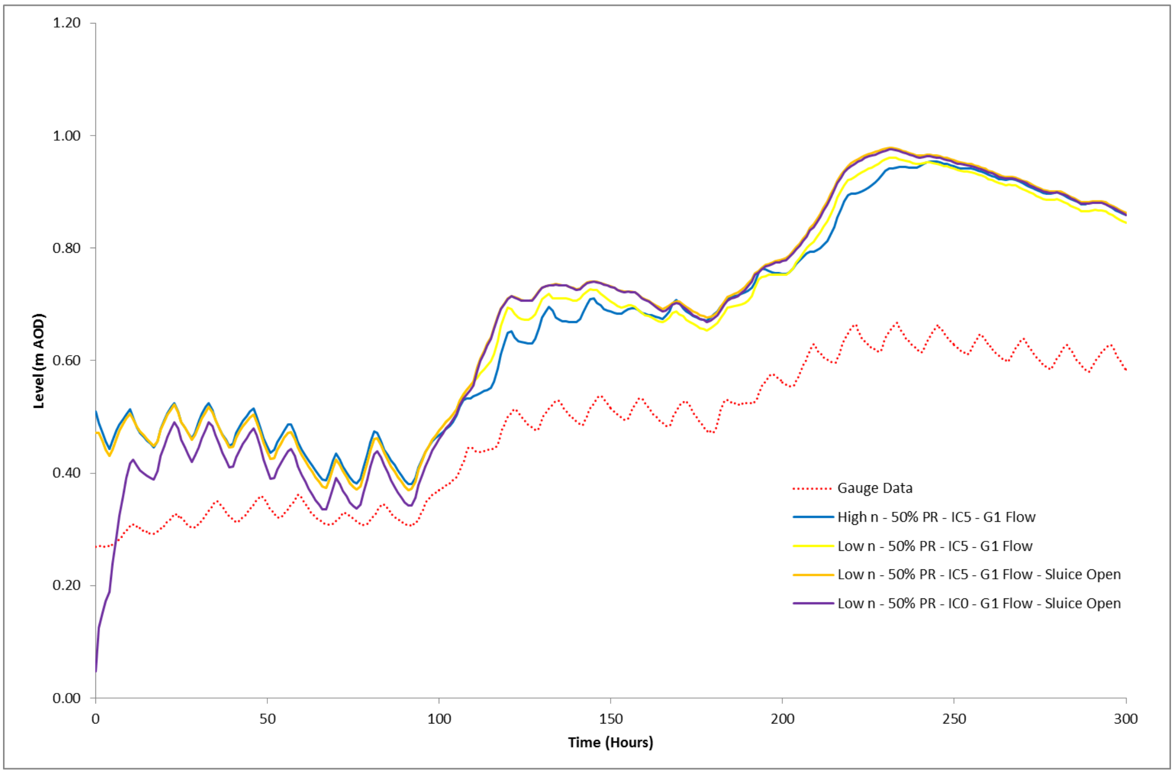

Figure 29 - Gauge G1 January 2016 Calibration Stage

Gauges G6, G1 and G8 are all in the lower reach of Leiston Drain. Figure 29 (G1 gauge) shows that despite different roughness coefficients and initial water levels being utilised, peak levels are similar across the suite of model runs for location G1 (halfway Leiston drain). The model results with the opening of the south chamber (penstock) at Minsmere Sluice show that water levels are higher in those events when it has been opened compared to those when it is not. There is only a small difference in levels between these scenarios due to the floodplain already being inundated from the previous flood peaks. It is believed that for smaller scale flood events, the opening of the sluice would have given a larger difference between the sets of results, however for this large event the effects are relatively drowned out. Although the opening of the sluice does inhibit Leiston Drain from discharging, it does also reduce water levels in the New Cut which subsequently reduces the volume of water spilling into the floodplain which would eventually drain to the Leiston Drain (via IDB drain #7), therefore it is questionable if in large events the opening of the sluice causes significant changes to the drainage of Leiston drain during the event. It is expected that if the models are run for a longer duration so that drawdown of the system can be observed (as will happen for subsequent design runs, construction and commissioning scenarios), the consequences of the opening of the sluice would be more clearly marked (levels are higher for longer in the Leiston system if it is left open).

Calibration and sensitivity runs for Leiston System

Figure 32 shows the recorded stage trace of G8 which clearly shows the timing and effect of the opening of the penstock at Minsmere Sluice (at approximately 110 hours). Opening the sluice impedes drainage of the Leiston system as the flow from the Minsmere New Cut also fills the southern chamber of Minsmere Sluice which the Leiston system drains into.

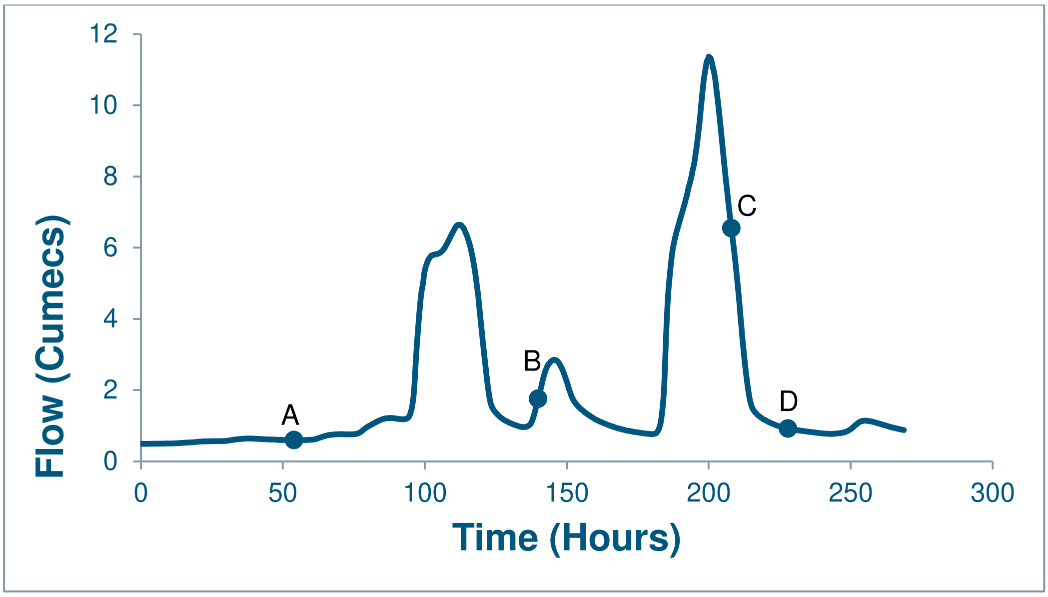

Figure 30: Inflow at Middleton, black dots indicate the time steps of the flood maps (A-D).

Figure 31 shows 4 stages of flooding in Sizewell Belts for moments indicated in Figure 30 (discharge at Middleton on y-axis). The first map (A) shows the inundation before the event, the base flow situation. Map B shows the maximal flood extent in Sizewell belts caused by the first inflow peak. Maps C and D show the flood extent caused by the second inflow peak with D showing the maximum flood extent for the 2016 event. The maps in combination with the graph of Figure 30 show that flooding of the Sizewell belts respond to the inflow at Middleton with a delay of 28 hours.

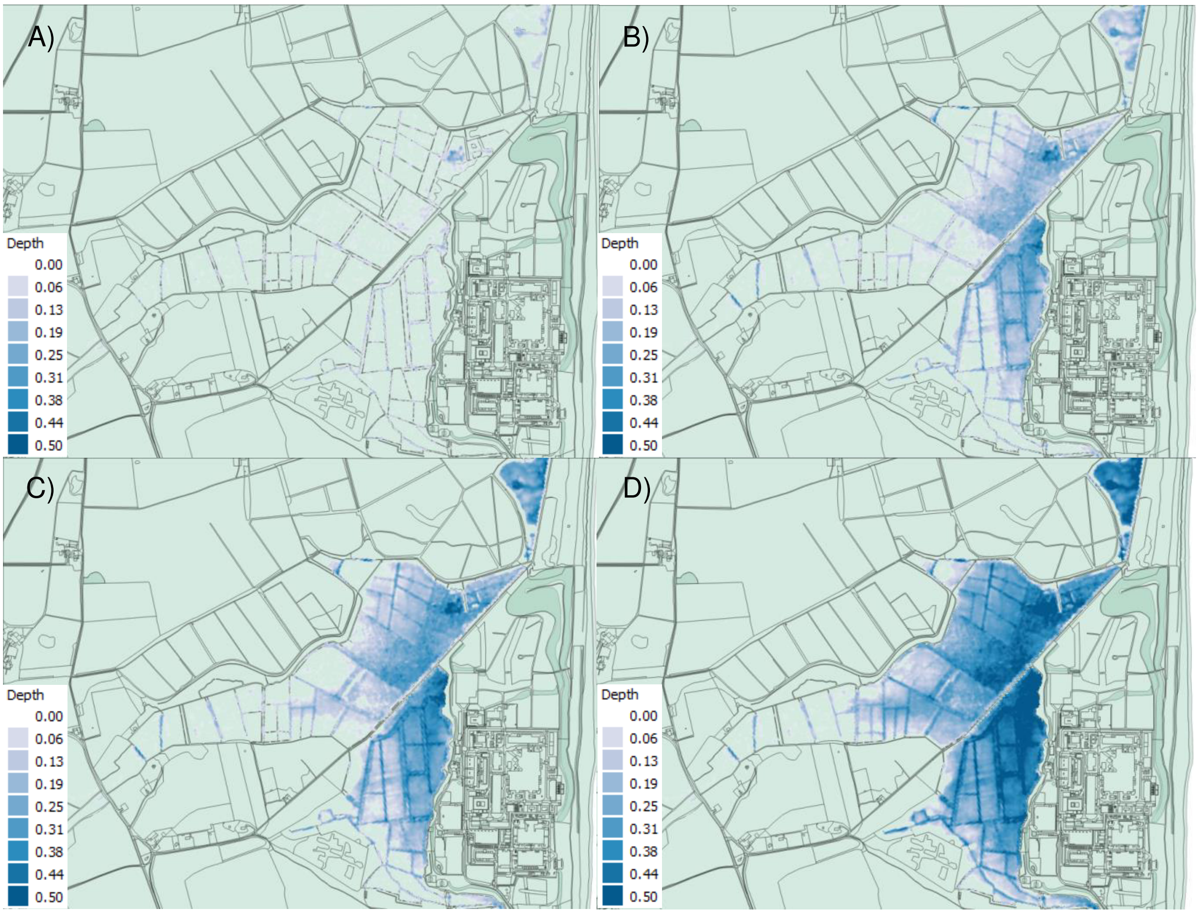

Figure 31: four stages (A-D) of flooding in the Sizewell Belts, showing the flood extent and the water depth in relation to the discharge at Middleton (fig. 26).

During the first year of monitoring, the weir crest geometry was modified at gauge G4. Therefore the gauge datum that was supplied at the time the gauge was installed has changed. For this analysis therefore, the lower values of the recorded data have been aligned with those of the modelled data. The peaks of the modelled results at gauge G4 are shown to be lower than those of the recorded data for the early peaks in the record. It is believed that this is due to the channel conditions at gauge G3. Field observations have shown that the weir at G3 (which is lower than the crest at G4) is often covered in vegetation or has a build-up of debris against it. The model does not include the effects of such occurrences; it assumes the weir is clear of debris. Therefore it is expected that in this event the model assumes more water to be flowing over the G3 weir than that that may have occurred; therefore the modelled levels would be lower at G4 than those that were observed. Despite this occurrence there is an approximate 100mm difference in peak water levels.

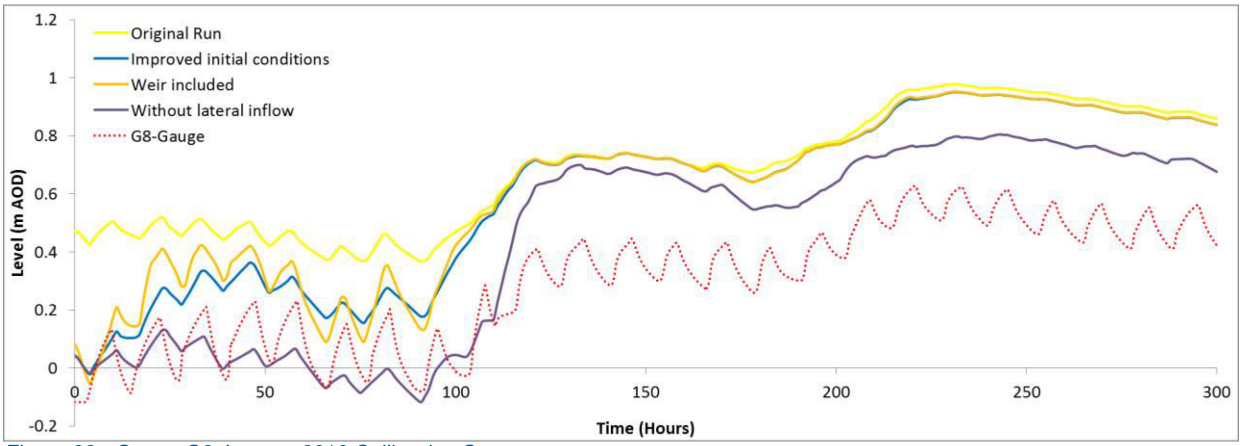

Figure 32 - Gauge G8 January 2016 Calibration Stage

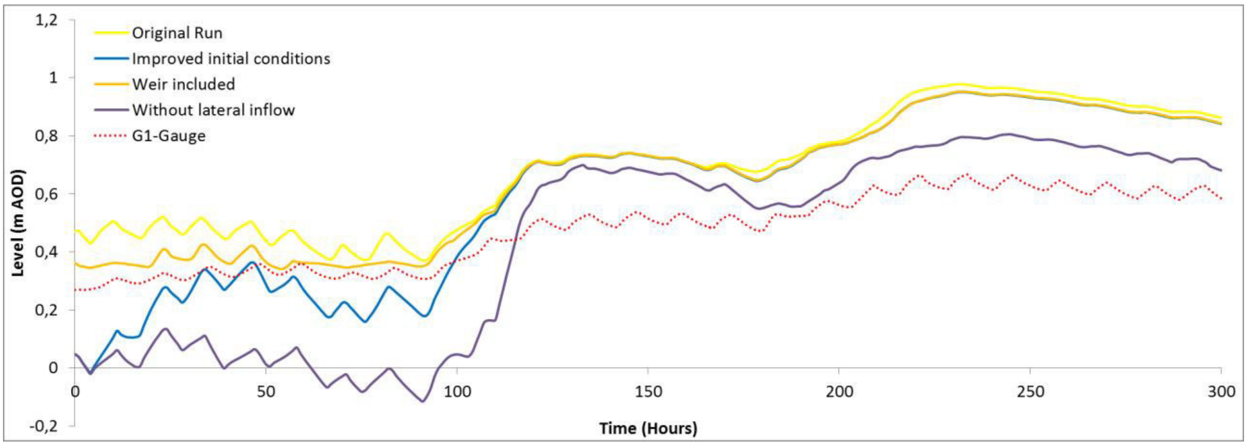

Figure 33 - Gauge G1 January 2016 Calibration Stage

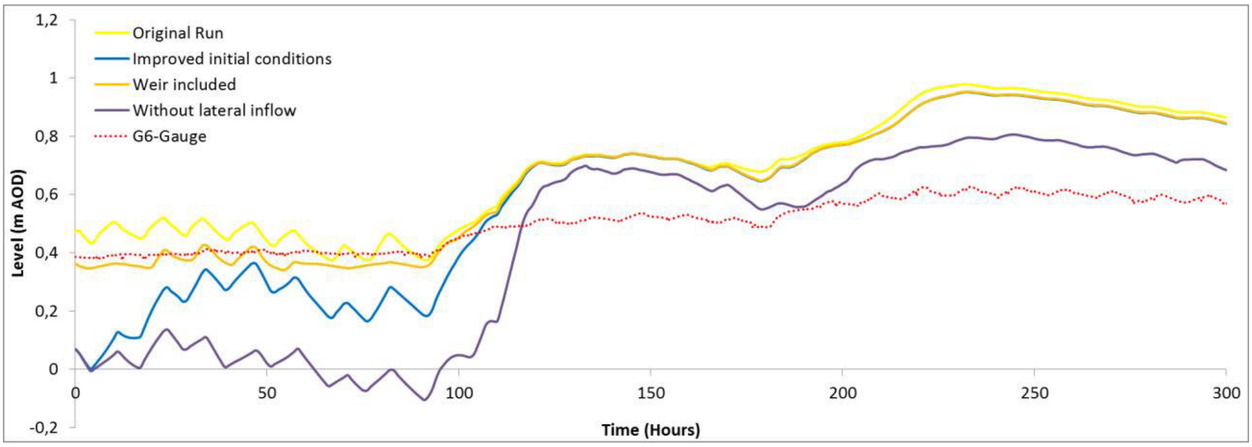

Figure 34 - Gauge G6 January 2016 Calibration Stage

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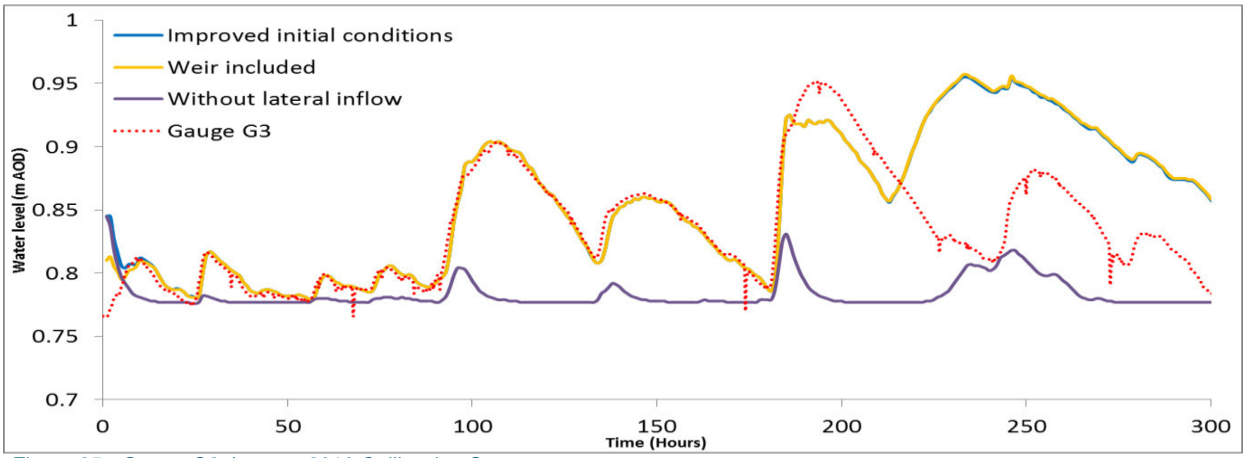


Figure 35 - Gauge G3 January 2016 Calibration Stage

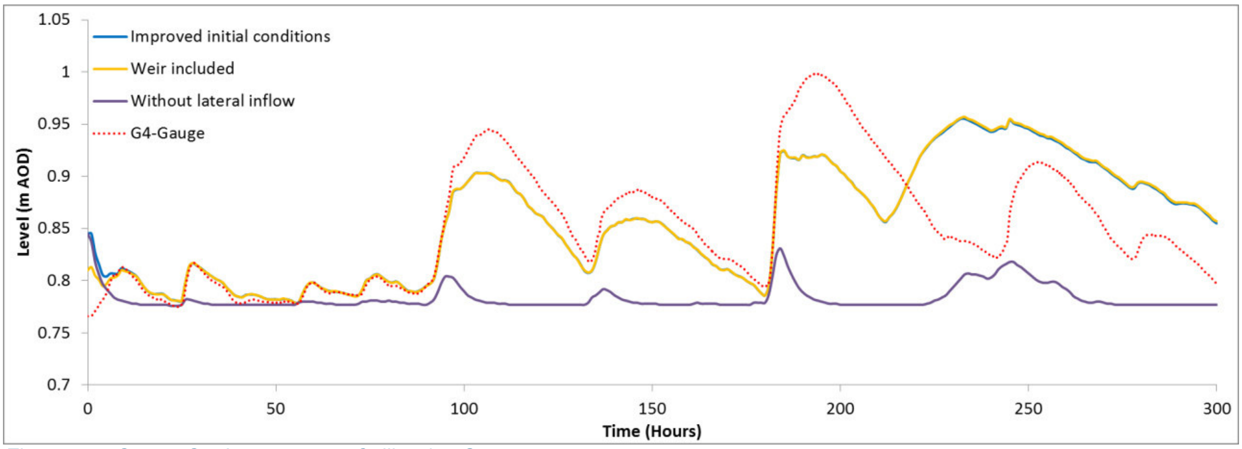


Figure 36 - Gauge G4 January 2016 Calibration Stage

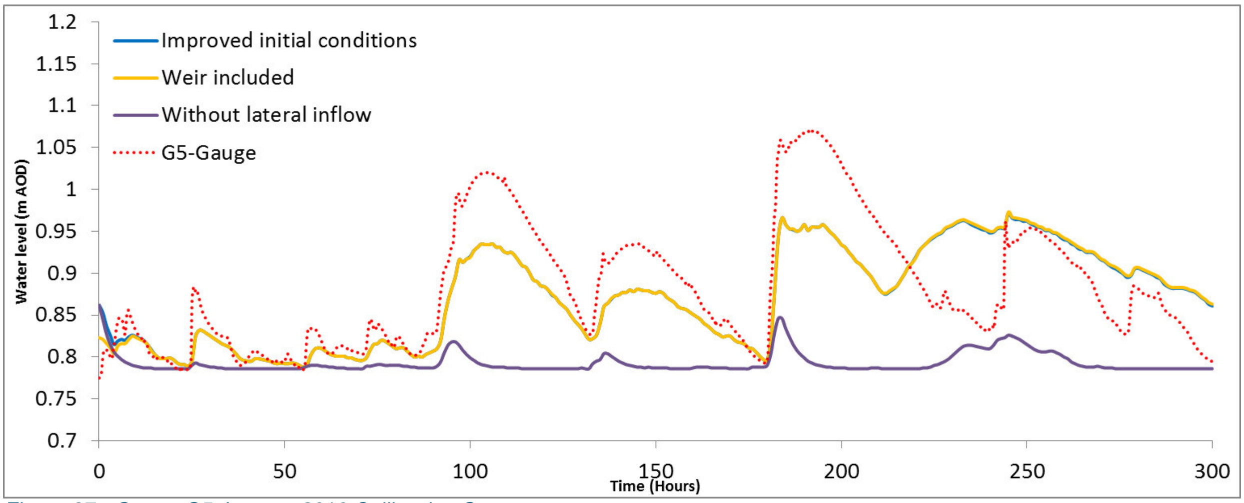


Figure 37 - Gauge G5 January 2016 Calibration Stage

Based on the model runs, the following conclusions can be drawn:

1. G3 / G4 / G5 (as shown in Figure 35,36, and 37 above): water level variation for this station very much relies on inflow of local flow. Model simulations show a good fit with the local inflow

as derived from the improved FEH formulas. The model deviates from the observations at the end of the flood event as a result of an overestimation of backwater from Minsmere (see other stations). G5 and G3 show a slightly underestimation of the flood levels by approx.10 cm.

2. G1 / G6: the model simulations show a good match for the initial conditions applied in the model. Especially at the low flow period prior to the arrival of the flood, the lateral inflow seems to be included correctly. Stations G1/G6 show a 30cm higher water level compared to G8 (near Minsmere Sluice) which was not correctly simulated in the original runs. This jump (also visible in Figure 38) is quite sudden as the distance between the two stations is only 1km. This indicates that there is some sort of bottleneck between G8 and G1. The bottleneck is probably caused by a (partial) blocking of the channel by vegetation or debris. To model this effect a dummy "weir" has been included in the model to mimic the effect of the obstruction. The result shows an almost perfect fit for the first 100 hours, up to the moment drain #7 starts overflowing in the model.
3. G8: this stations show most effect of tidal variation in the water level, which is well represented by the model for low flow events. For high flow events the simulated water level in Leiston drain is overestimated as a result of back flow from Minsmere. This effect is present in the actual situation but to a minor extent. The impact of the backwater effect is most visible for the downstream station (G8) and less for the upstream stations (G4). During high flow events the tidal signal is flattened out as it does not propagate properly through the gates of Minsmere sluice. The complexity of the multidimensional behaviour of Minsmere Sluice in combination with the energy losses in the culverts make it highly complicated to adjust the sluice schematisation to allow for better tidal propagation. Considering the impact of adjusting the sluice schematisation on other elements of the system it was decided not to further optimise the tidal inflow in Leiston drain.

It is concluded that the Leiston system shows good representation of the actual system. The upstream station G4 shows the best fit for most of the time. The other stations especially show a good fit for low flow conditions up to the moment when bank overtopping along Minsmere New Cut (and drain #7) occurs. The conclusion can be drawn that flooding of Leiston system is very much determined by the backwater effect from Minsmere. Overestimation of Minsmere flooding therefore also leads to overestimation of Leiston flooding (for G8, G1 and G4). G5 and G3 show underestimation of flood levels up to the moment the backflow from Minsmere arrives in the upperpart of the Leiston System. Further calibration of Leiston system is not possible as long as the Minsmere system cannot be calibrated more accurately. The quality of the Minsmere calibration is limited to the quality of the base model as well as lack of sufficient calibration data for most gauging stations in Minsmere system.

The text box, below, provides anecdotal evidence and describes the sensitivity of water levels in the Leiston system to blockages and the natural functioning of the system.

*3.5.72 - After completion of the remedial works to the sluice, Suffolk Wildlife Trust noted that winter water levels (2014-15) in Sizewell Marshes were higher than they have previously been. This had been attributed by the Environment Agency to the presence of several blockages on Leiston Beck. Some blockages were removed by the Environment Agency at the start of week commencing 30 March 2015. Afterwards, a gradual reduction in water levels was observed in monitoring data from the site. It should be noted that woody debris was observed in parts of the channel during the geomorphological walkover survey (**Section 3.6**), and it is therefore possible that similar blockages could occur again in the future as part of the natural functioning of the system.*

From: Sizewell C Main development site surface water conceptualisation, RHDHV, Sept 2015.

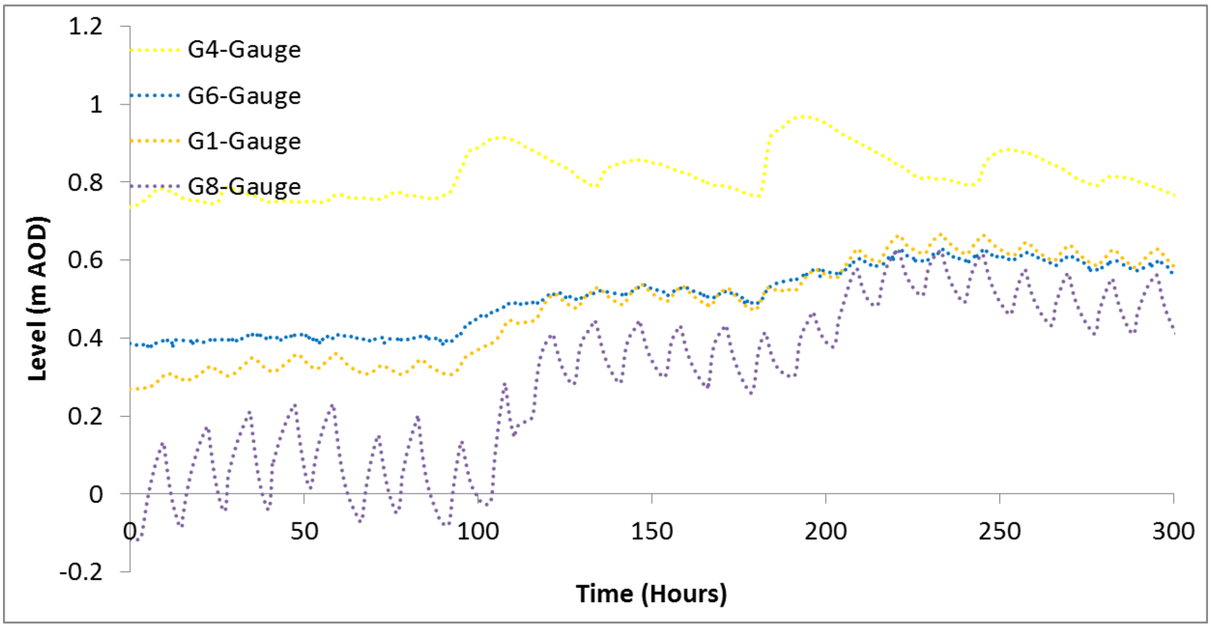


Figure 38 - Gauged water levels for the January 2016 event

Minsmere flooding

Knowing the relevancy of a correct simulation of flooding along the Minsmere New Cut it was looked at the distribution between flooding of South Levels and North Levels. The Minsmere system can be divided into two major flood areas, north of the Minsmere new cut (North Levels) and South of the Minsmere new cut (South Levels). The modelled water levels in the north (North Girder) and south (C93) are quite different. Water levels in the north vary between 0.46 and 0.56m +AD, showing very little reaction to the hydrograph. Water levels in the south vary between 0.4 and 1.0 m +AD, showing direct linkage to the passage of flood wave. This example shows that the model predicts most flooding to the south whilst almost no water floods to the north. When compared to the field observations this indicates that the amount of water flowing south is overestimated for the 2016 event.

It was also observed in the model results that flooding is initiated upstream from Middleton bridge, allowing flow to enter drain #7 from where it starts inundation of the South Levels. Inundation of Middleton upstream of Dam Bridge is most likely the result of inaccuracies in the cross-sections in 1D ISIS model. Solving this matter requires to go back to the basis of the 1D model, which would only make sense if better calibration data is available allowing full model calibration.

7. Model Performance Conclusions

For the purpose of determining the flood risk of the proposed developments of Sizewell-C a hydraulic 1D-2D model (ISIS-TuFlow) has been developed. The existing 1D ISIS model has been integrated with a 2D Tuflow model and (semi) calibrated for three hydraulic events using flood trash marks, field observations of flooding, absolute gauge data for 2016 flood event and supporting relative gauge data (water level differences). Considering the limited data availability and quality, it was not possible to execute a full calibration for this model. Looking at the different data sets used for calibration the following conclusions can be drawn:

Positives

- The model performance based on a comparison between simulated levels and gauge data is more complex as gauge data is only available on a weekly basis for most stations in Minsmere system, while the flood hydrograph has a typical daily behaviour this means that the observed hydrograph often misses the peak moment of the hydrograph. Some stations however have data points taken near the peak. The 2003 and 2006 data for C93 gauge shows a good match with the model simulation results for the flood peak. The same happens for the 2006 data point at North Girder that nearly hits the flood peak. Other stations have missing flood data for these events and thus cannot be used for validation/calibration.
- The January 2016 event, which contains detailed gauge data for the Leiston drain, shows a direct relation between observed and modelled data which provides confidence that the model is a good representation of the actual situation from daily stage up to flood stage.
- Observations from mainly the RSPB were followed as much as possible. The model simulations more or less match with flood observations, which also provide confidence that flood mechanisms are represented well by the model.

Negatives

- Due to the limited data points application of a statistical method to quantify model performance (e.g. RMSE or r^2) is not possible for this model. The model performance for the 2010, and 2003 events is therefore based on a visual match completed by system understanding.
- Flood observations have been used as much as possible. Although the model behaviour can be linked to observations, a strong conclusion cannot be drawn since the observations miss exact information about time and location.
- For 2016 absolute gauge data is available for the Leiston system. Since the Leiston system is largely hydraulically influenced by backflow from the Minsmere system, calibration of the Leiston system can be executed to only a limited extent (effect of calibration is damped by the stagnant water from Minsmere).

The overall conclusion is that the model has been calibrated to a reduced extent because of partial data availability (Minsmere) and interrelation between sub-basins (backwater effect from Minsmere in Leiston drain). The model however represents the behaviour of the system well (matched with available data and observations) and is therefore considered representative for the Minsmere and Leiston water system.

For the Leiston system it can be concluded that the model shows good representation of the actual system. The upstream station G4 shows the best fit for a majority of the time period. G5 near Leiston town shows a slight underestimation of flood levels. The other stations show a good fit for low flow conditions up to the moment when bank overtopping along drain #7 occurs. The conclusion can be drawn that flooding of the Lower Leiston system is determined by the backwater effect of Minsmere flooding, while flooding of the Upper Leiston system is more affected by local runoff. Further calibration of the Leiston system is not possible as long as the Minsmere system cannot be calibrated more accurately. The quality of the Minsmere calibration is limited to the quality of the base model as well as lack of sufficient calibration data. Any gauge installed within the Minsmere sluice structure would need extensive work carried out over a period of time to determine if the gauge is recording correctly given the complexity of the sluice structure and would not necessarily add anything to support further calibration of the model.

The model accuracy is spatially varying. For Leiston system the model accuracy is in the order of 5 - 10 cm for low flow conditions and 10-20 cm for high flow conditions. In the South Levels the model accuracy is in the order 10-40 cm due to overestimation of water arriving through drain #7.

8. Model application for Sizewell-C FRA

From the above conclusion we understand the extent to which the model is applicable for the assessment of flood risk for the Sizewell-C development. Apart from calibration, various sensitivity checks have been executed as part of the calibration process, which provide a better understanding of the system. This knowledge is critical when interpreting the model results correctly. During the process the models performance was checked and the optimal model settings during which the model performs best were derived (see Table 6).

With the knowledge gained from the model calibration and sensitivity calculations it is now possible to estimate the application of the model for the assessment of the Sizewell-C development. Inundation of the Leiston system (incl. Sizewell Belts) is mainly due to backwater flow from Minsmere system. This inundation can be considered as *storage* rather than *flow,* which is highly relevant for the estimated impact of the Sizewell C development. In case of stagnant flood water in Leiston system, reduction of the storage capacity of Leiston system as a result of the development has a limited effect on the hydraulic system compared to a situation in which flow dominates the hydraulic system under flood conditions.

The impact of Sizewell-C developments is expected to be in the order of 1–2 cm's while the model accuracy is in the order of 5-10 cm for low flow conditions and 10-20 cm for high flow conditions in the area of impact by SZC. It can therefore be concluded that the impact of the development (site and change in direct runoff) may be within the accuracy level of the model. Since the model does represent the behaviour of the current hydraulic system well, the model can be applied to execute a worst-case approach. The approach assumes that if downstream backwater influences are large the effect of the SCZ development differs from a system under free gravity flow where no backwater effect is present. By running the 'extremes' of the system, it results in a bandwidth of possible outcomes for the effect of the SCZ development on the hydraulic system. Our professional estimate is that the bandwidth of outcomes is rather small.

Worst-case approach

The conclusions from the model calibration and system analysis is that Leiston system can be considered as a free flowing water system up to the moment when flooding of Minsmere arrives and affects the flood level in Leiston system. By applying the final calibrated model, the effects of SCZ development on a free flowing water system can be assessed because of the influence from downstream. We will therefore apply a worst-case approach to consider the bandwidth of possible outcomes for a situation with maximum effect of the SCZ development under a flow dominating system as well as the maximum impact under a stagnant flood water dominated system:

- **Flow dominated system**: Leiston system functions fully independently from Minsmere by eliminating the backwater effect from the model. This simulation will show the maximum possible impact of the SZC development in a flowing system (maximum effect of any obstruction such as SSSI crossing).
- **Stagnant flood water dominated system**: The model, as it is now, overestimates the backwater effect from downstream, which leads to a system that is fully dominated by stagnant water. When looking at water level change, the SCZ development shows a different response to a stagnant water system compared to a flowing water system. A simulation with the current fluvial model shows the maximum effect of SCZ development under stagnant flood water conditions best. After construction of the SSSI crossing (causeway), the backwater effect may be reduced by the smaller flow capacity under the causeway.

The worst-case approach shows the bandwidth of outcomes for the impact of SCZ development. Based on the first outcomes an approach will be developed for further detailing of the FRA. Considering the natural hydraulic impact of extra obstruction by vegetation and dead wood in the channel, the estimated impact of SCZ is very small.

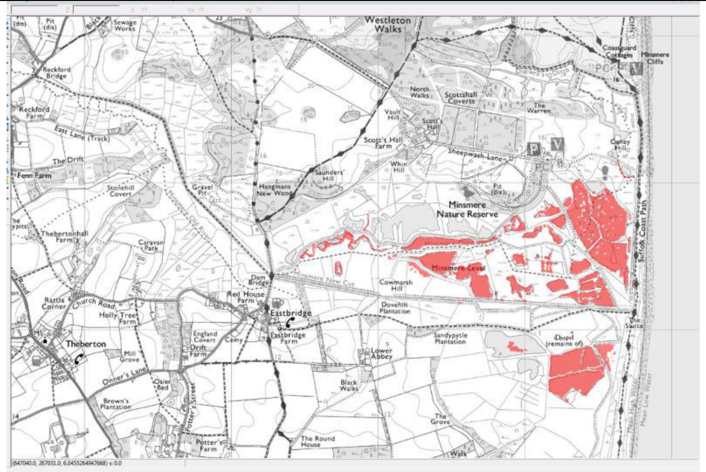
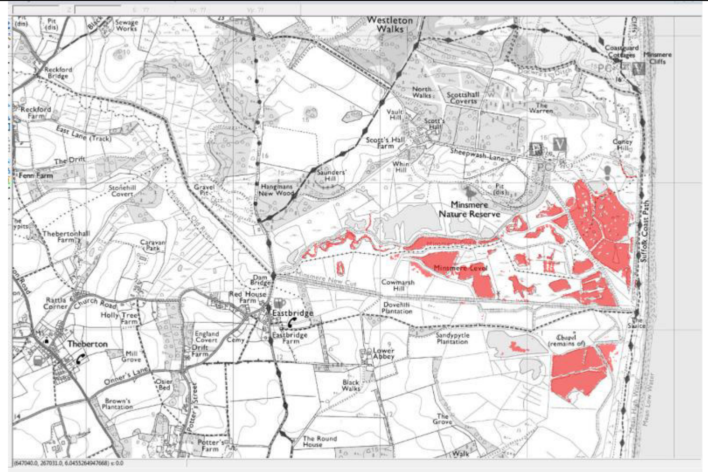
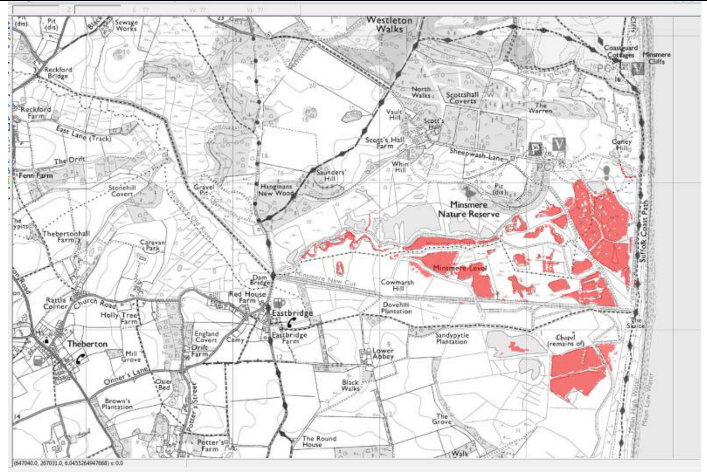
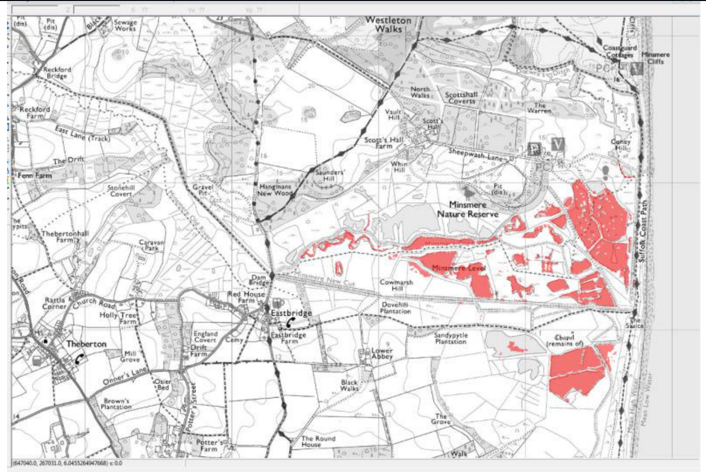
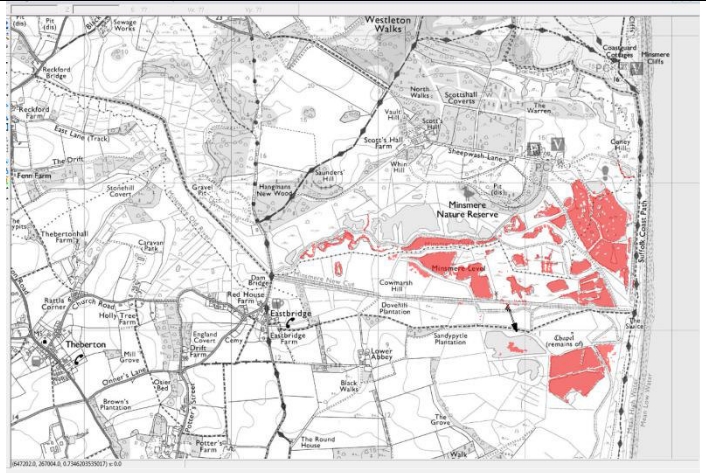
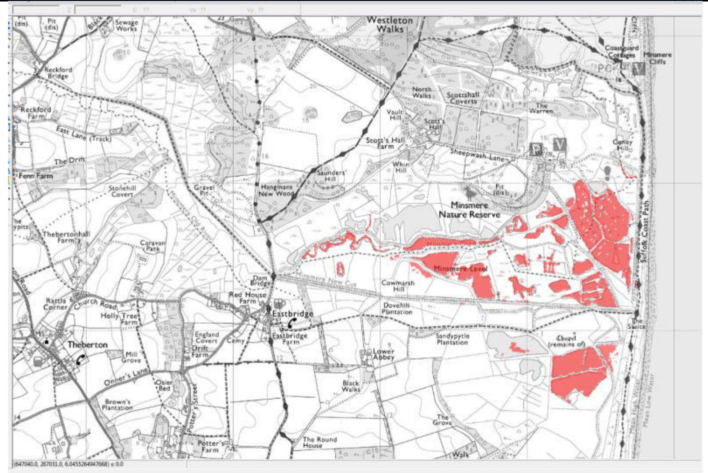
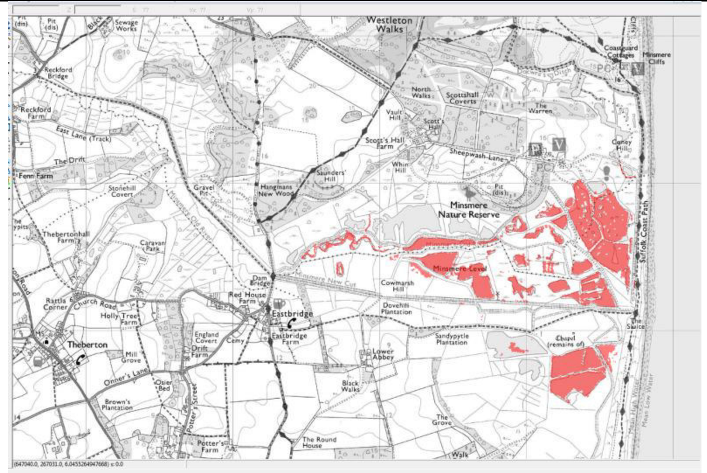
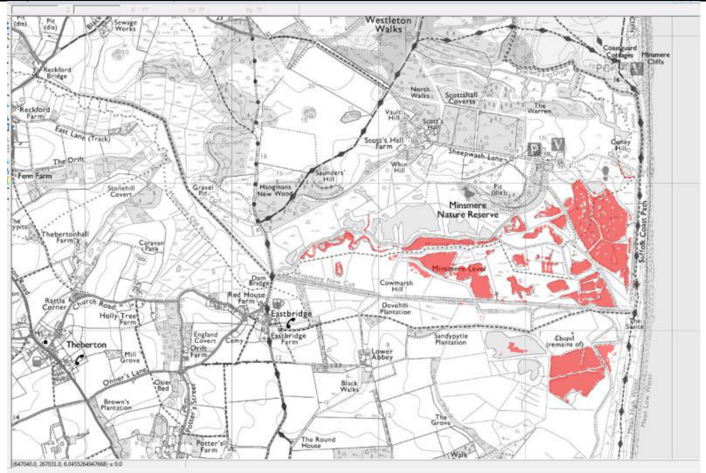
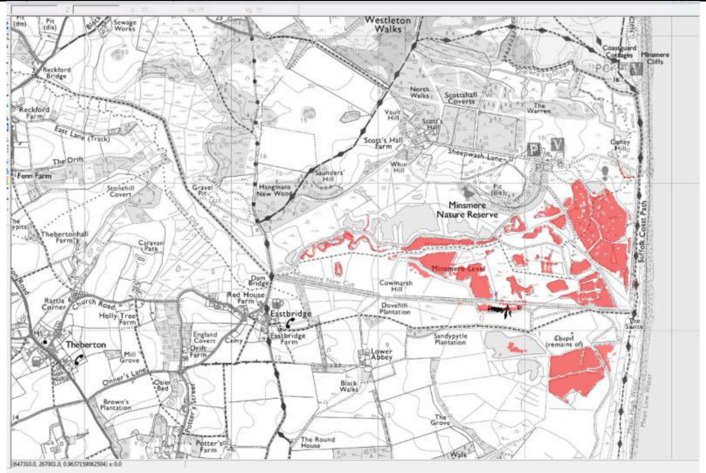
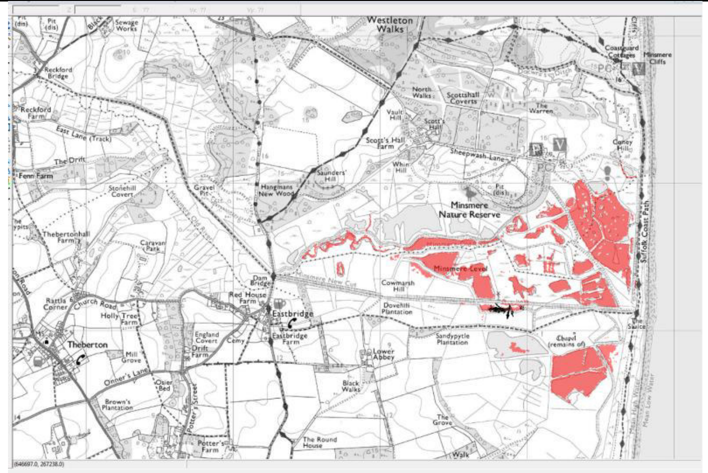
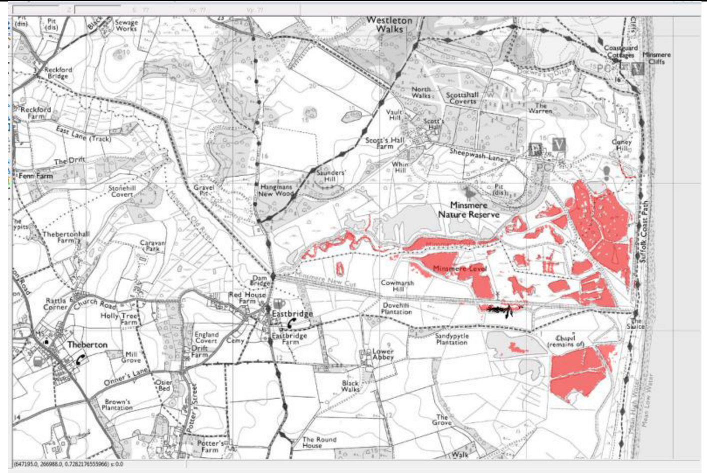
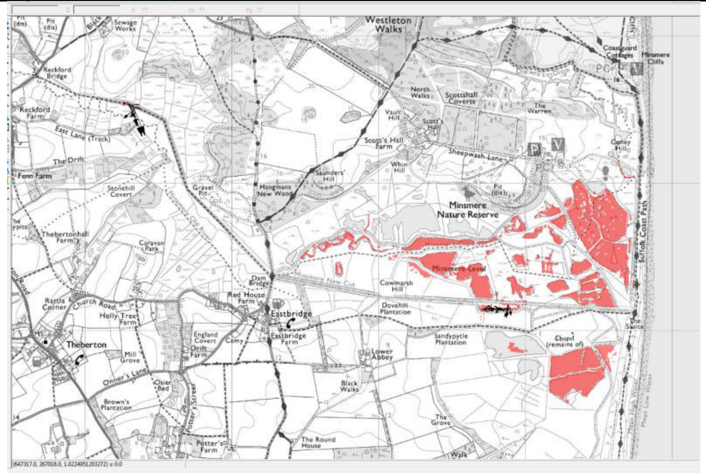
The model is built and calibrated for assessing flood risk impact on significant flood events. It should be noted that the model is not applicable to run 'smaller' below bank full events than those considered. It is recognised that in smaller fluvial events the parameters tested may have a greater influence on the results. Low magnitude flood events may be influenced by the management regimes of local hydraulic works, which are not necessarily included in the model. It is therefore proposed these should not be run due to the difficulties encountered in the past (as discussed in the Memo dated 7th January 2016).

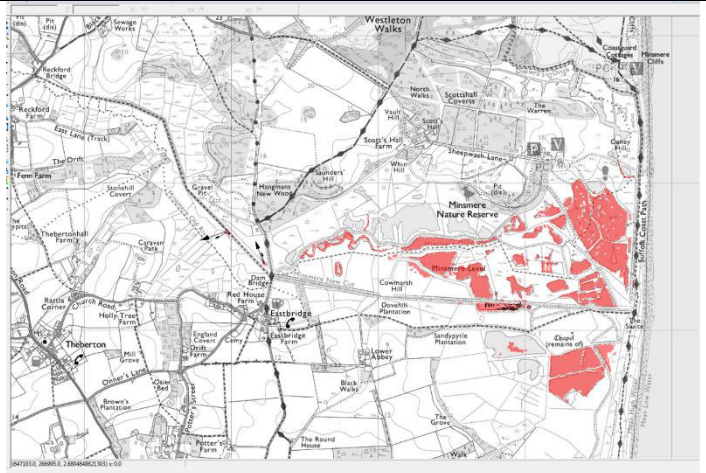
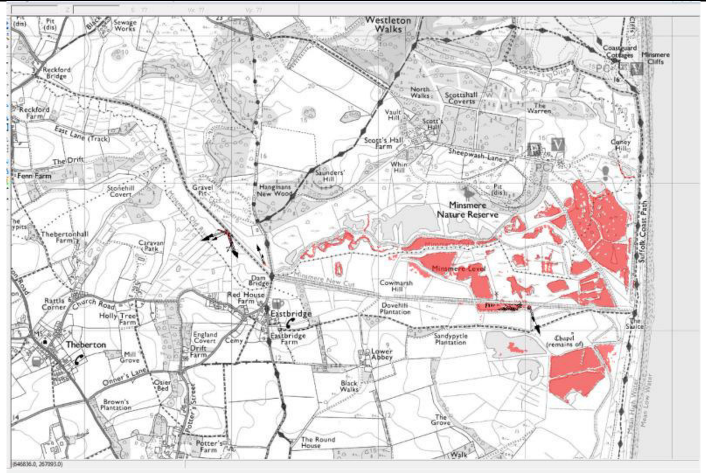
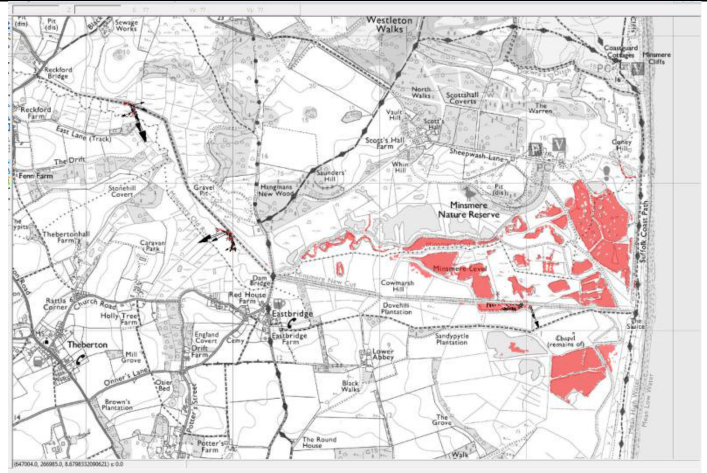
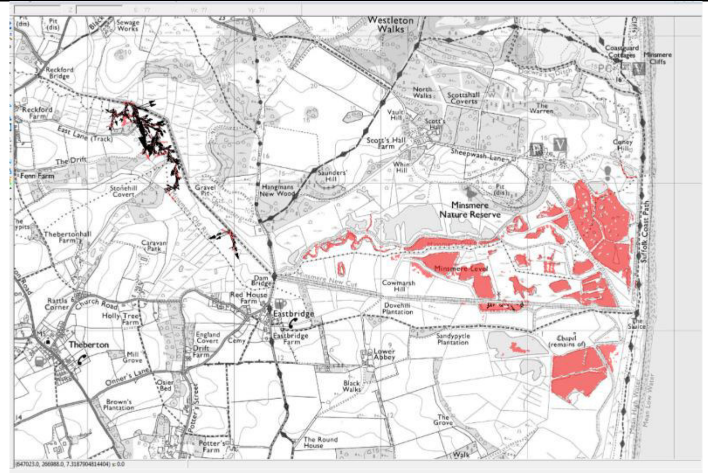
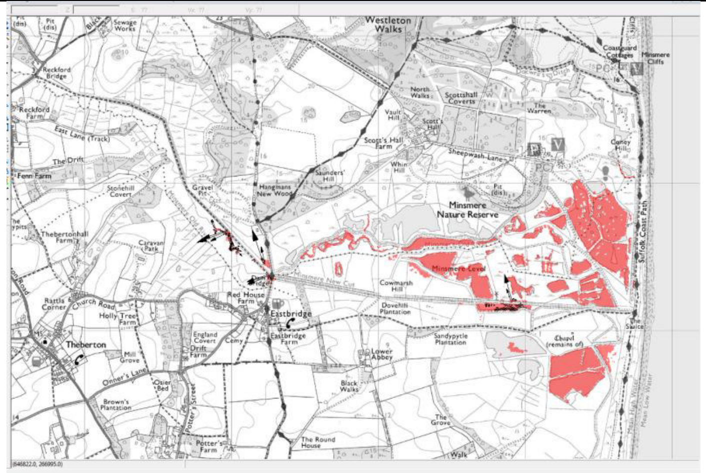
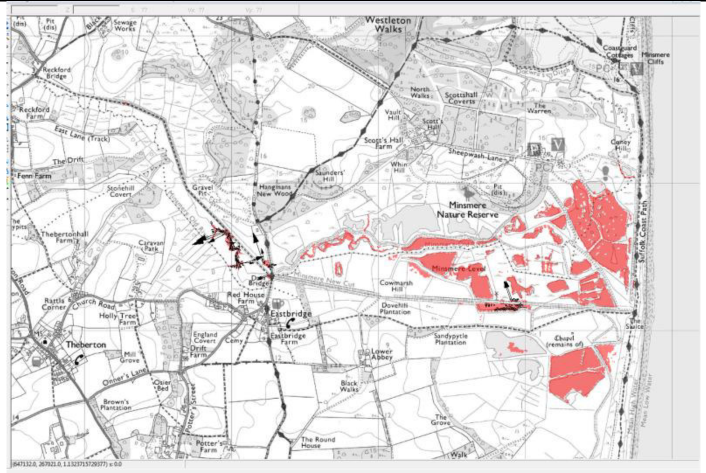
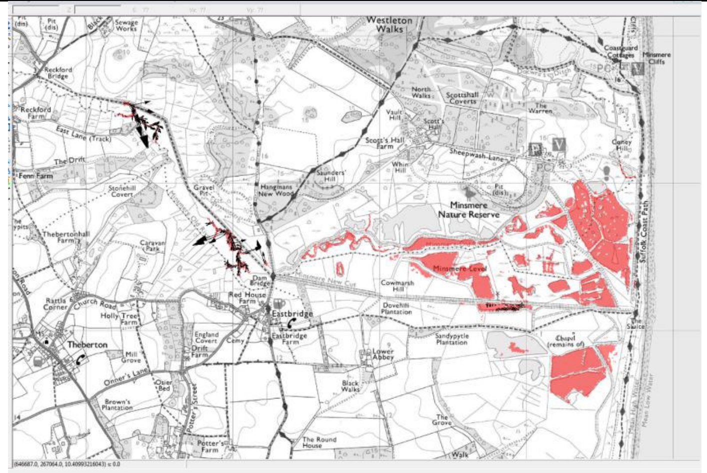
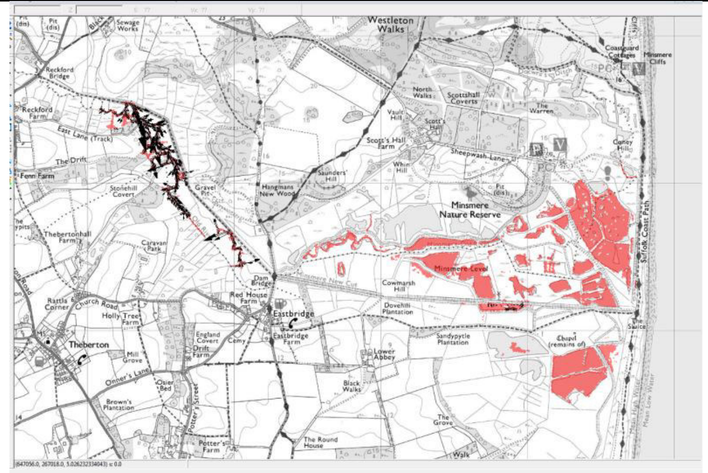
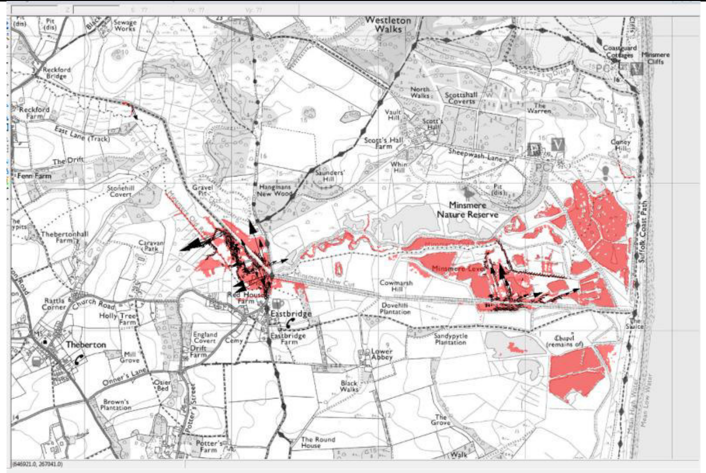
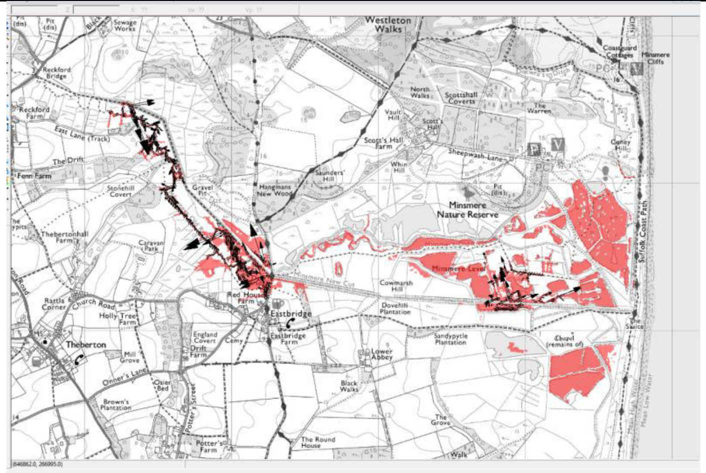
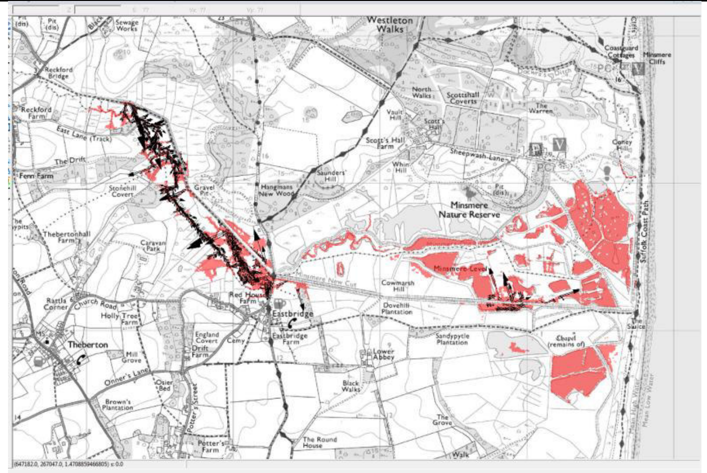
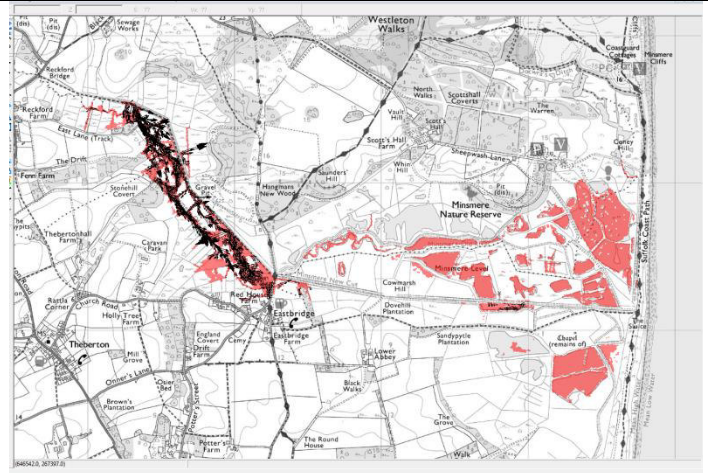
Table 6 – Parameter settings based on calibration runs.

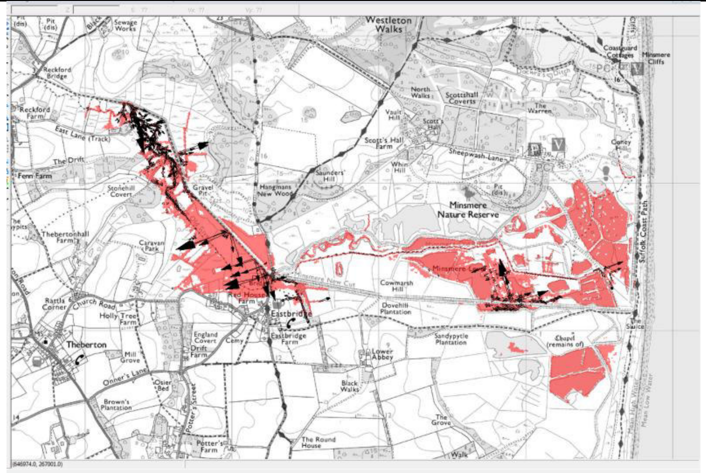
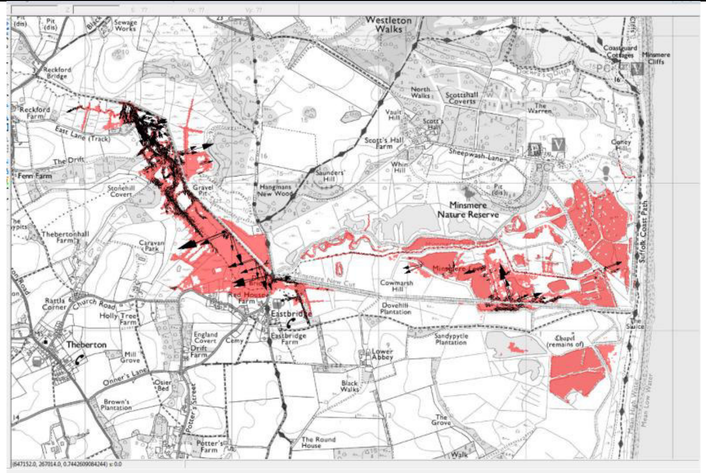
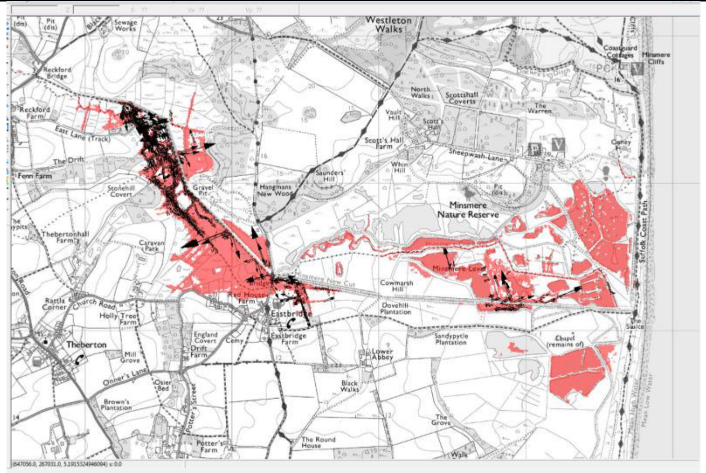
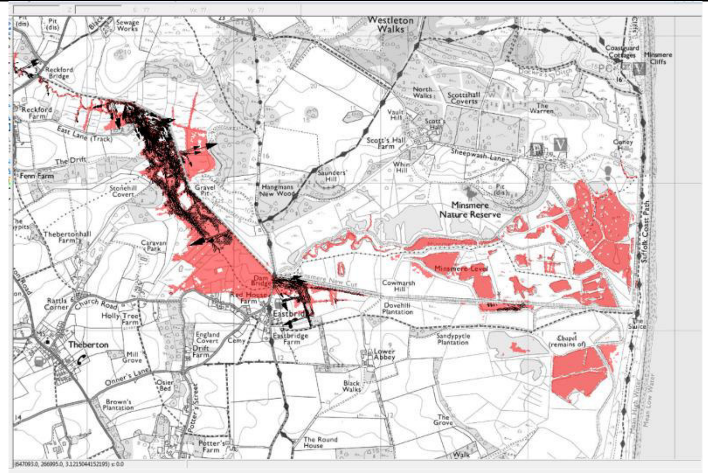
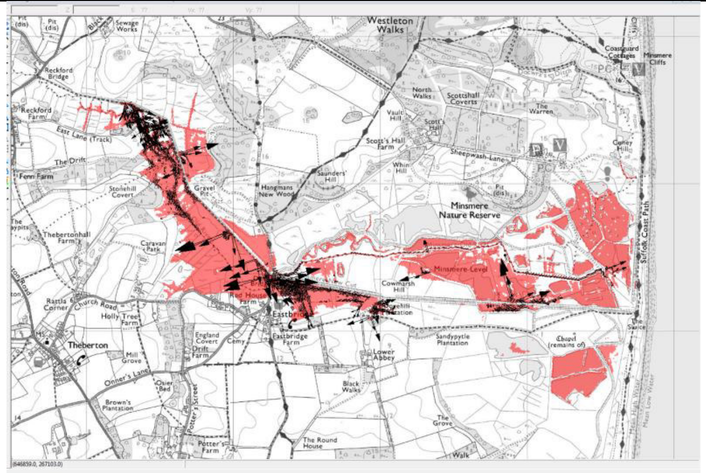
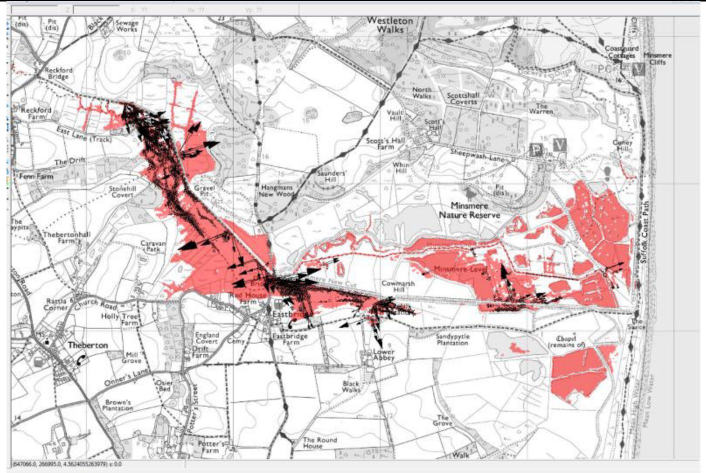
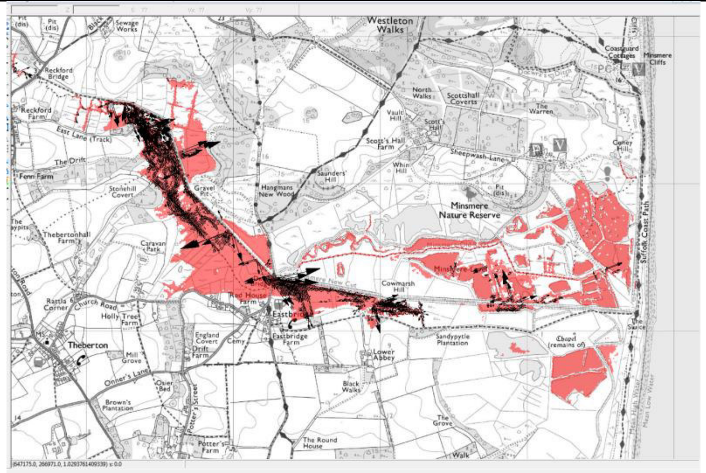
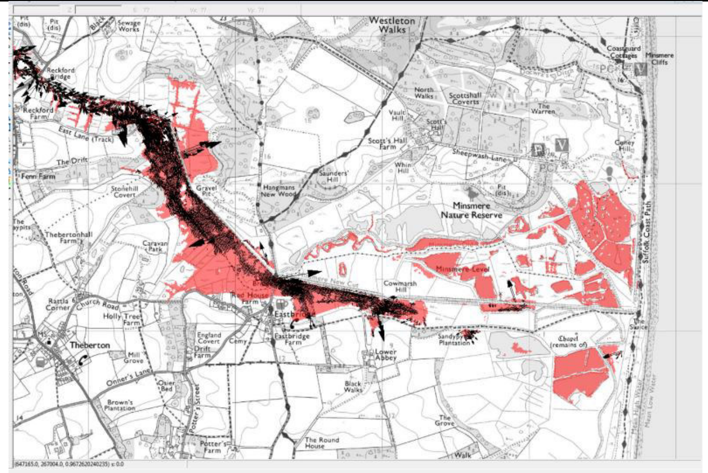
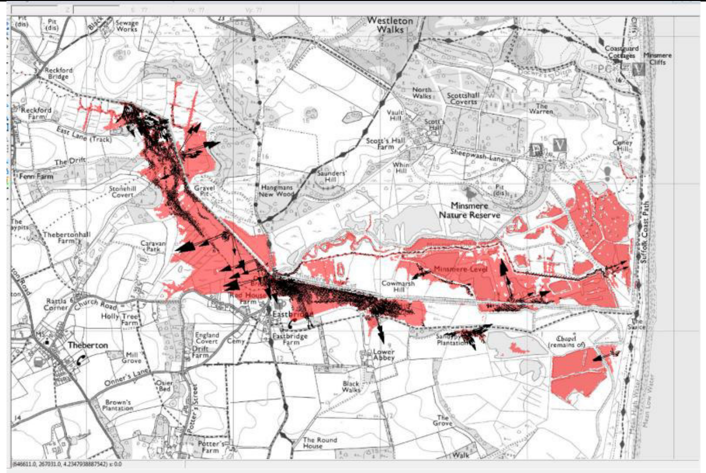
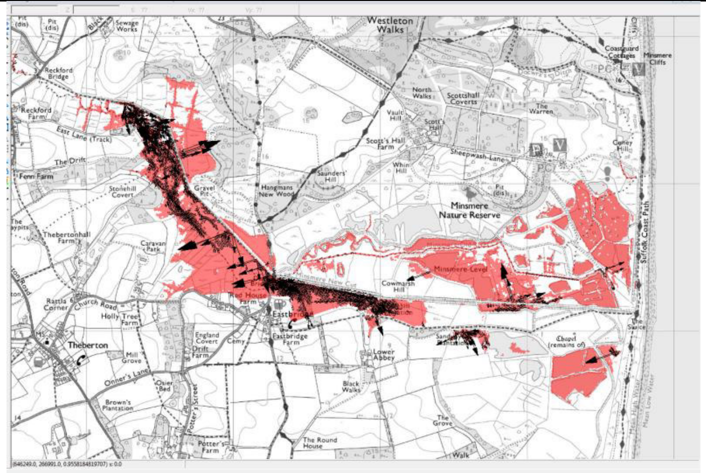
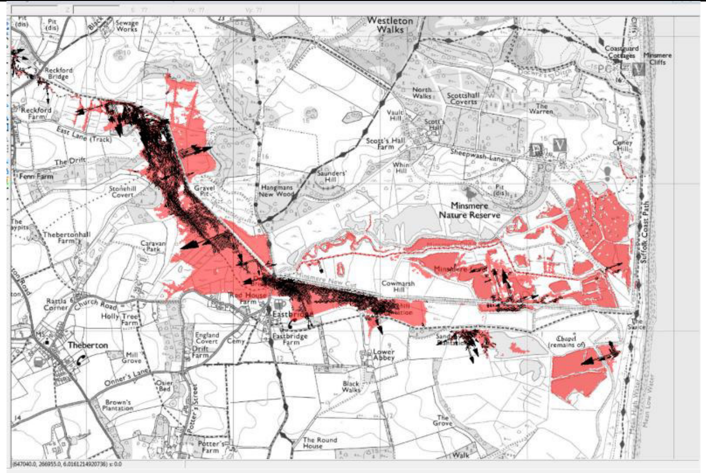
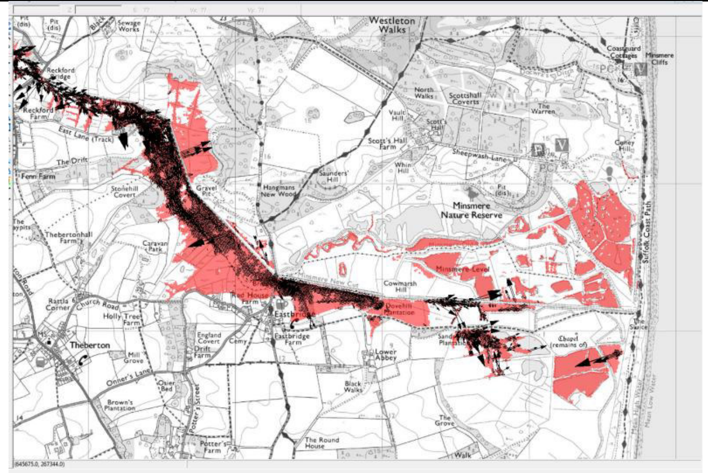
Model parameter setting	Explanation
In channel roughness: 0.08n	A higher roughness provided a better match between the model and observations, 0.08 is chosen after consultation with the EA.
Initial conditions: 0.5m AOD	Initial conditions will be set to 0.5m AOD as this has shown the best results for larger events.
Opening sluice southern culvert Minsmere New Cut	As the sluice will be opened during any large events in the area, design runs will be run with an opened sluice as well.

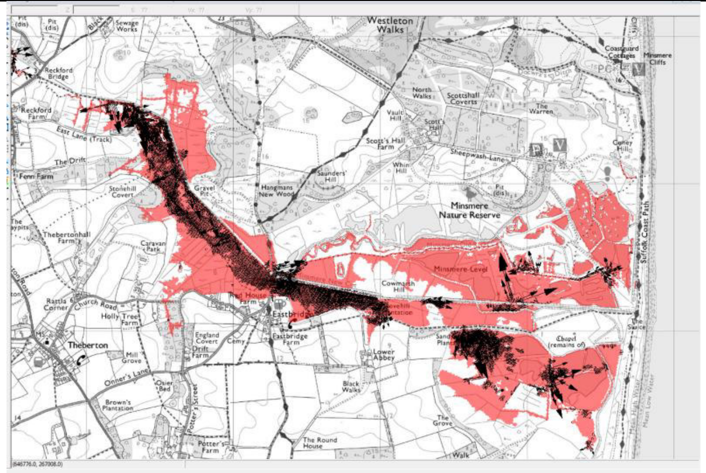
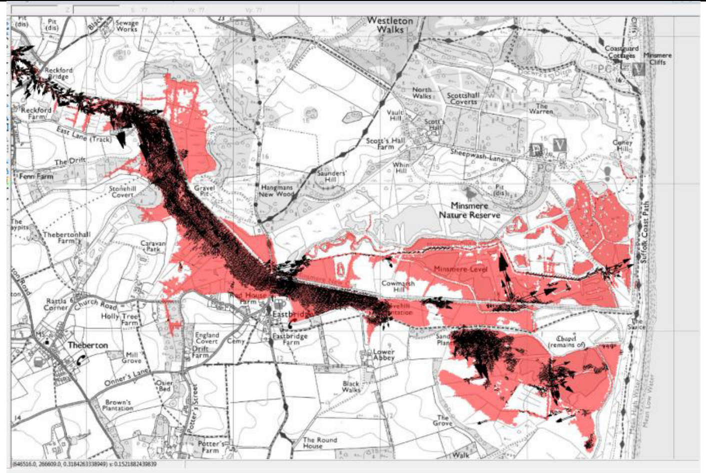
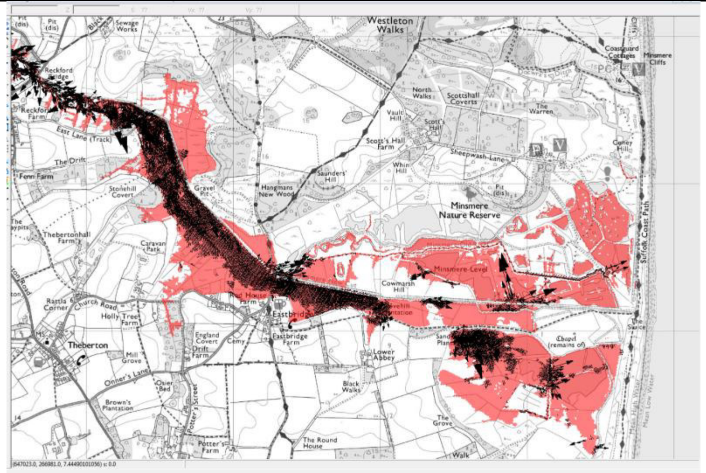
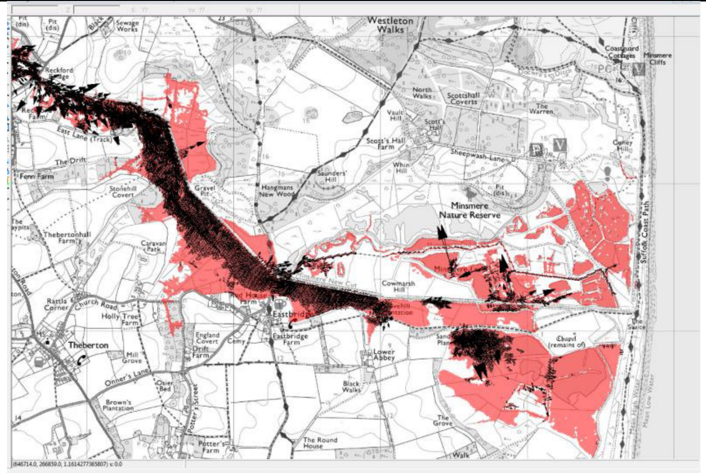
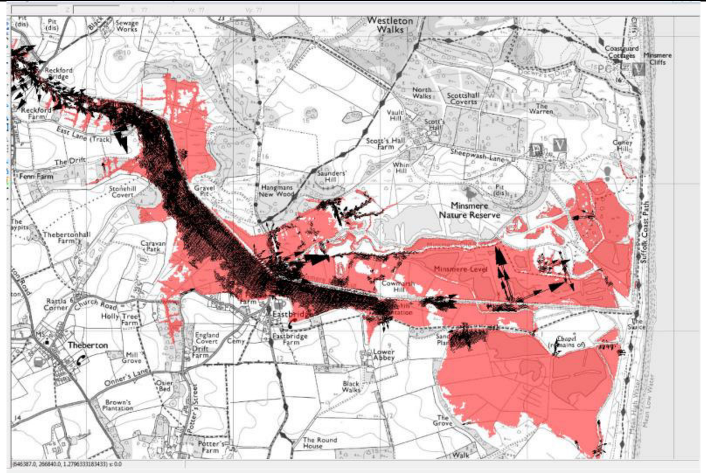
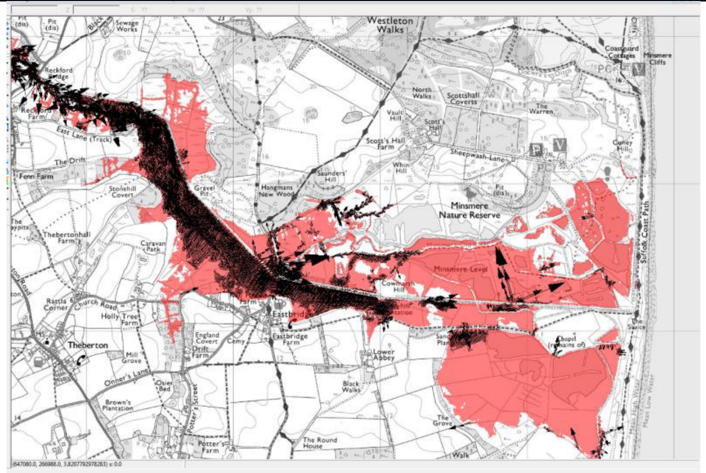
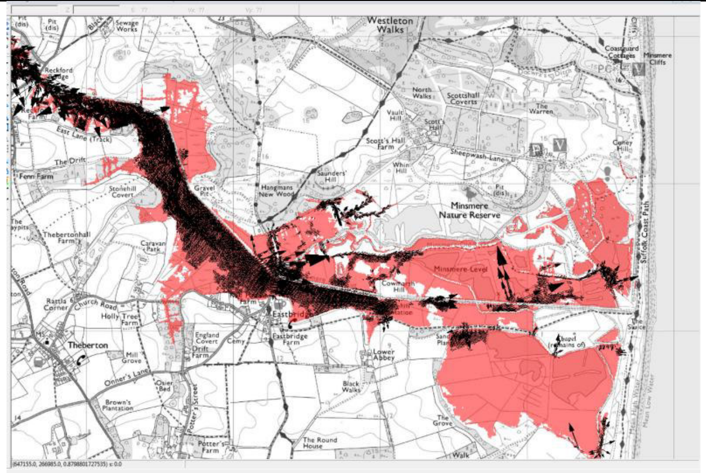
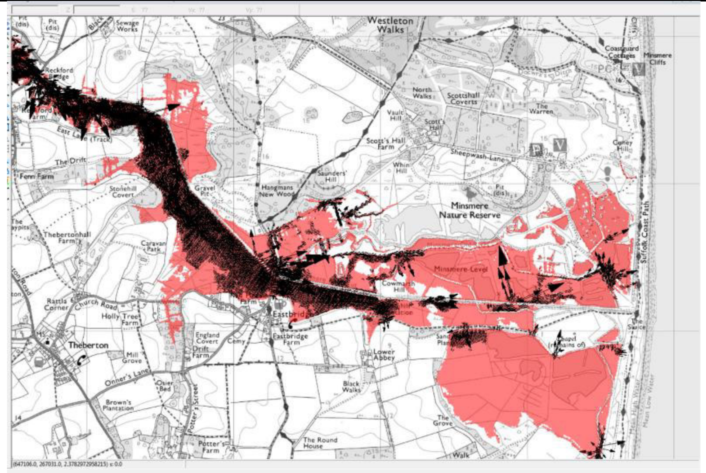
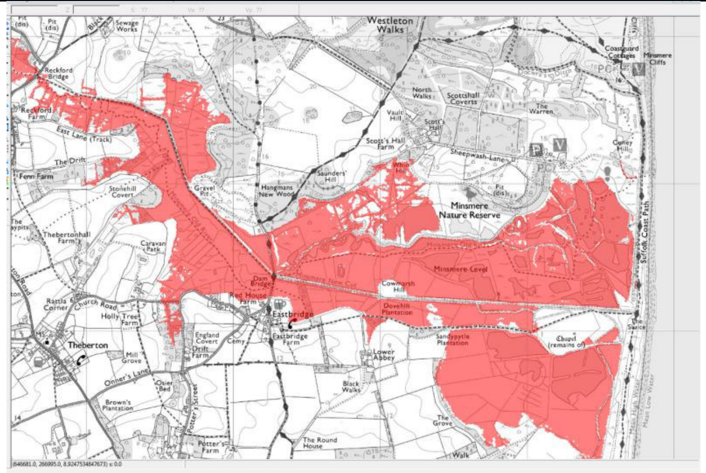
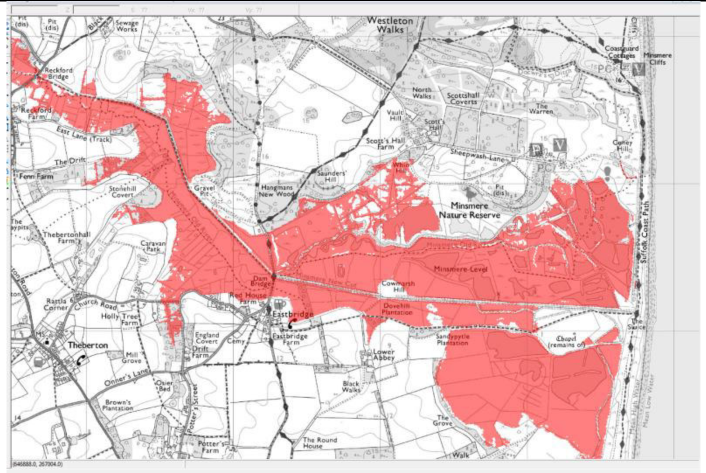
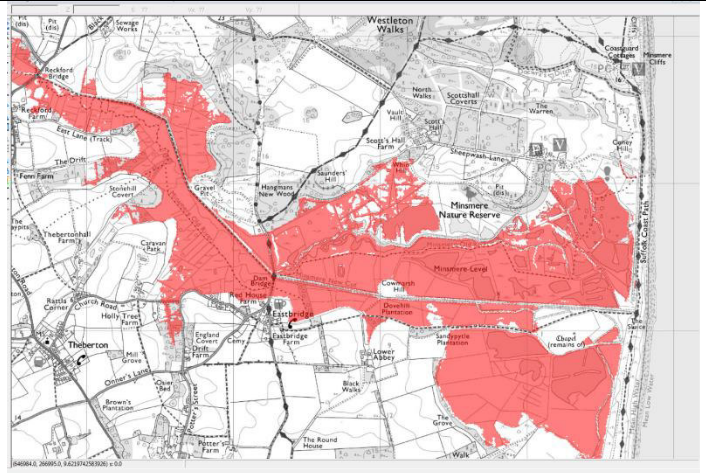
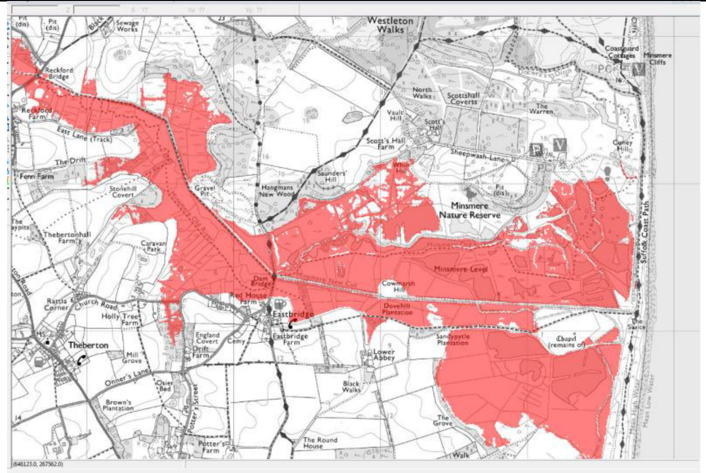
APPENDIX A – Observations of flood mechanisms

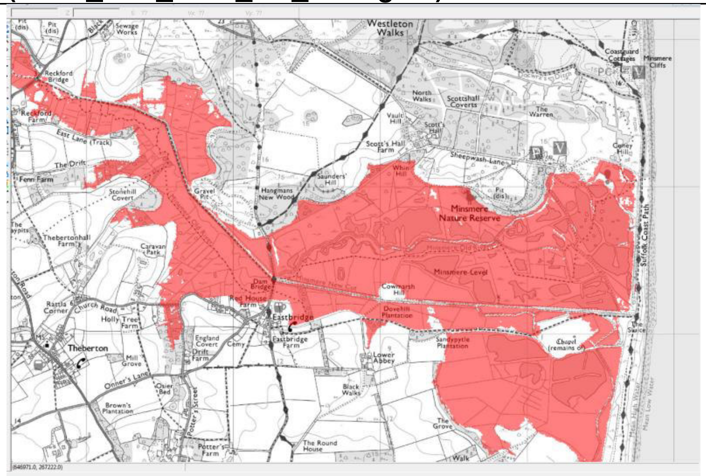
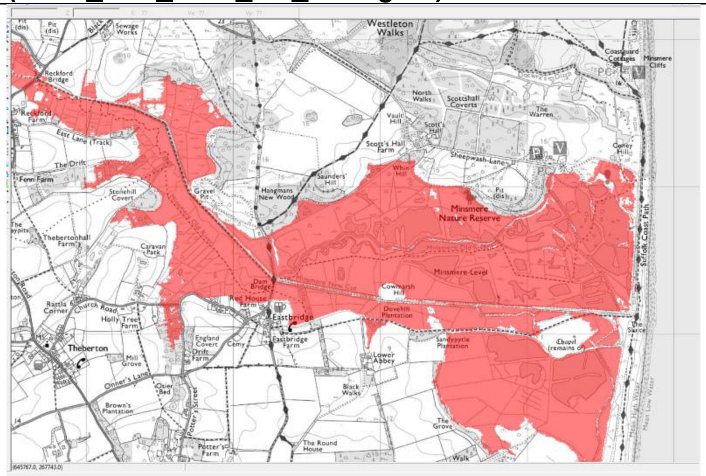
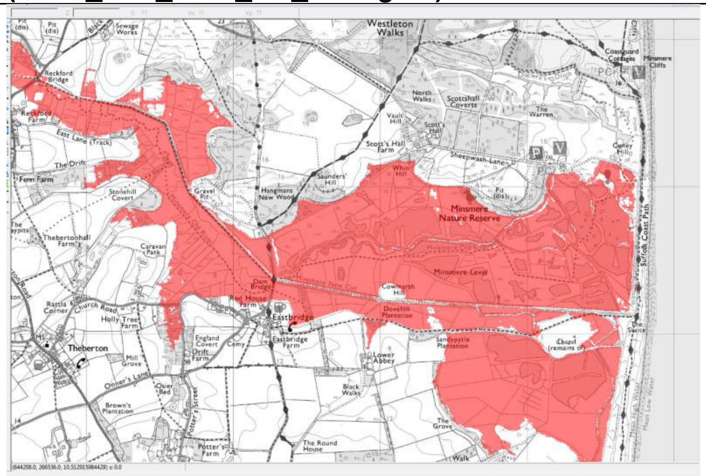
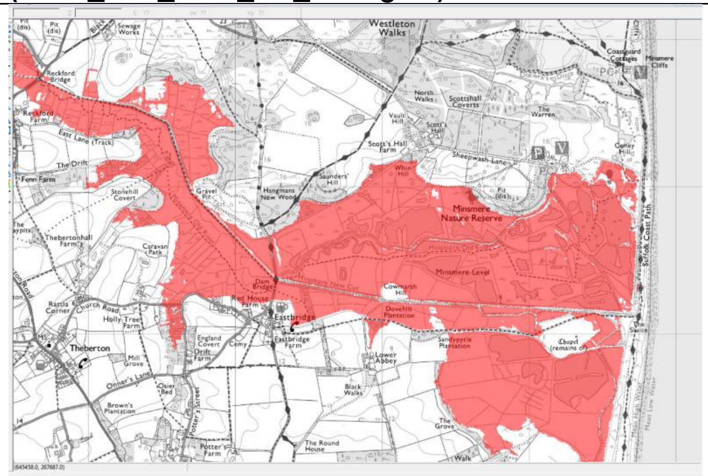
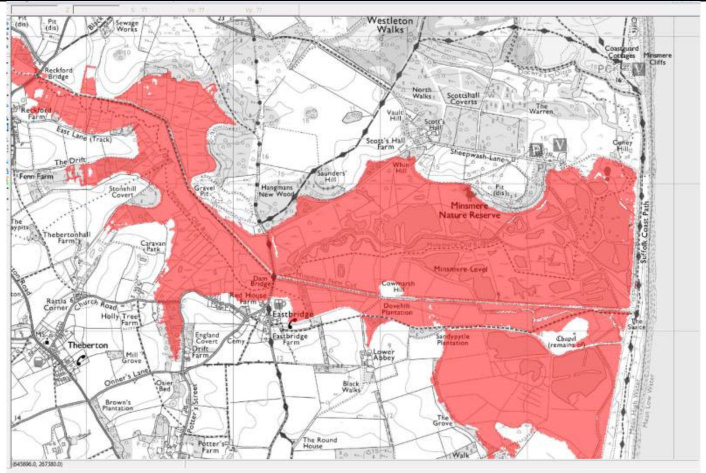
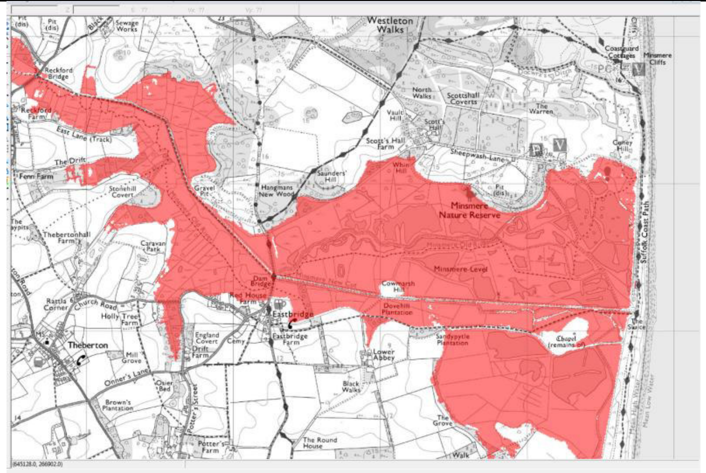
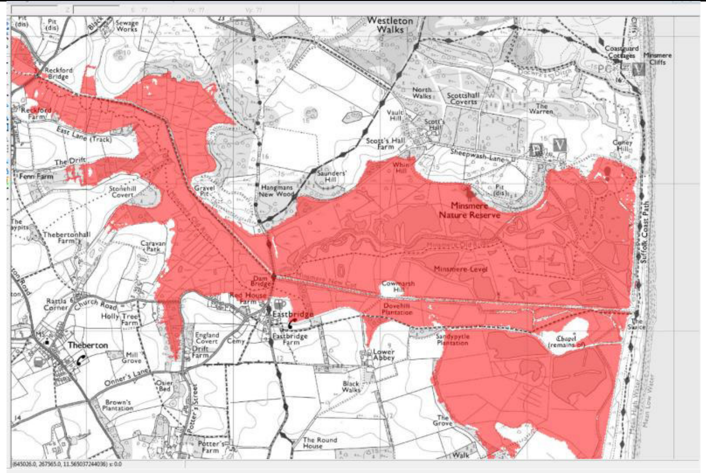
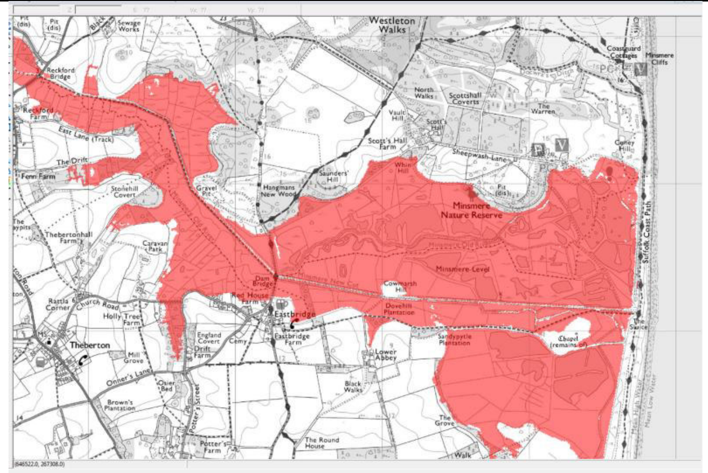
Appendix A – Observations of Flood Mechanisms from the Improved Rating Models

Time (hours)	Minsmere New Cut channel roughness 0.045 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I062_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.055 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I063_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.065 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I064_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.085 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I065_T027_NR_Rating.ief)
0 - 24				
29				
31				

Time (hours)	Minsmere New Cut channel roughness 0.045 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I062_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.055 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I063_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.065 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I064_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.085 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I065_T027_NR_Rating.ief)
35				
36				
44				

Time (hours)	Minsmere New Cut channel roughness 0.045 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I062_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.055 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I063_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.065 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I064_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.085 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I065_T027_NR_Rating.ief)
60				
70				
74				

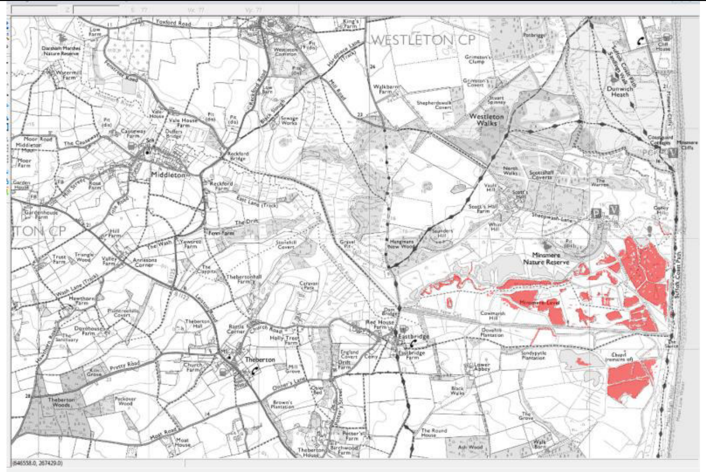
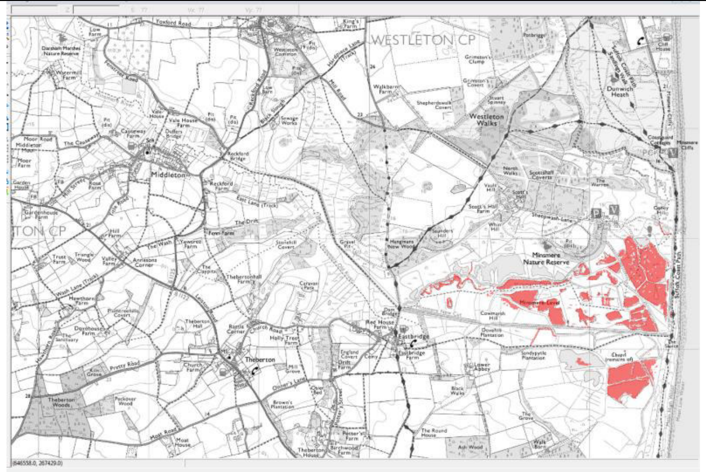
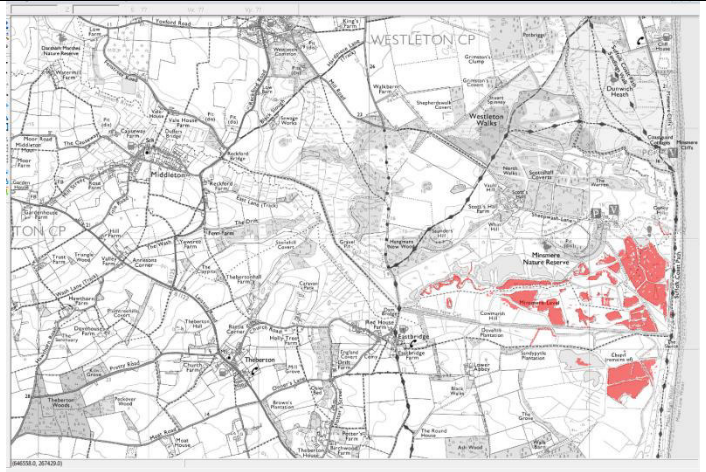
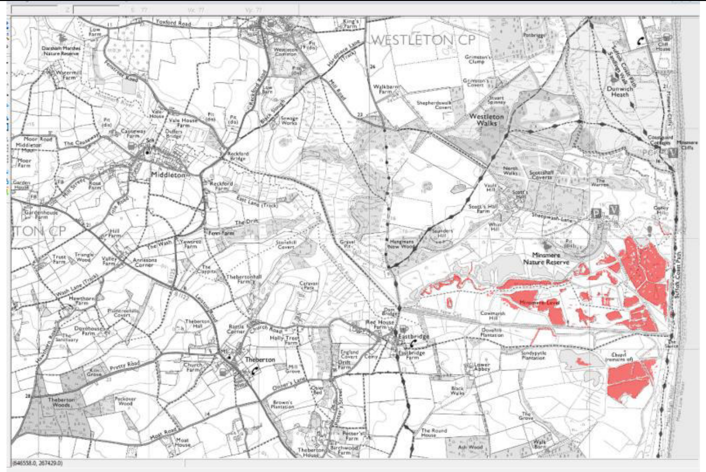
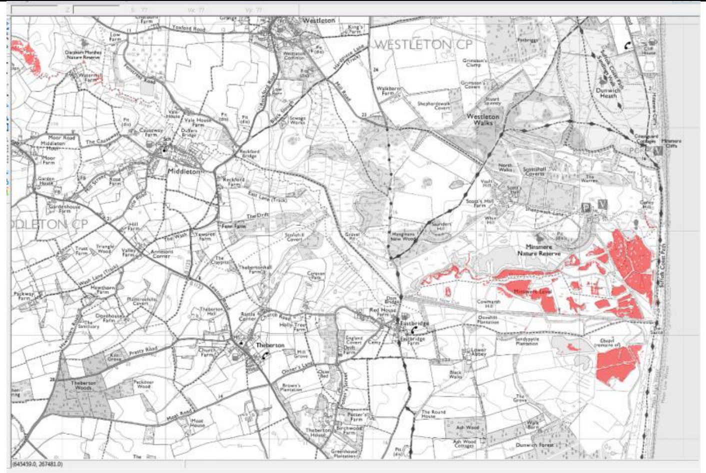
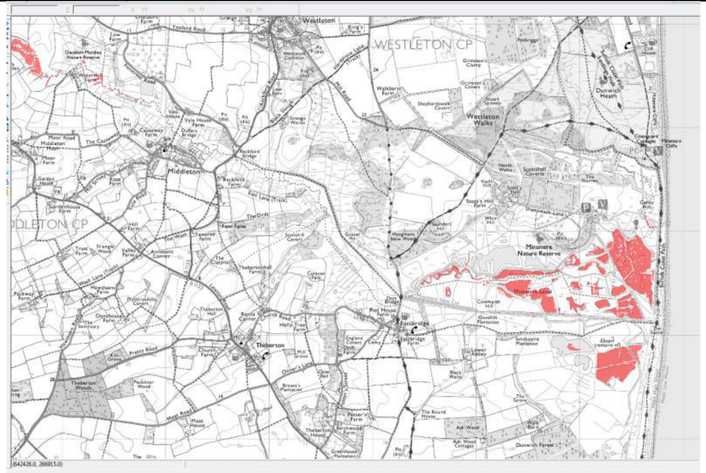
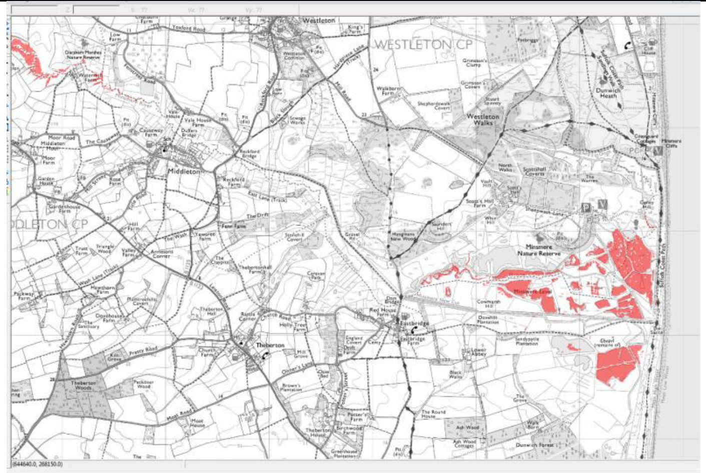
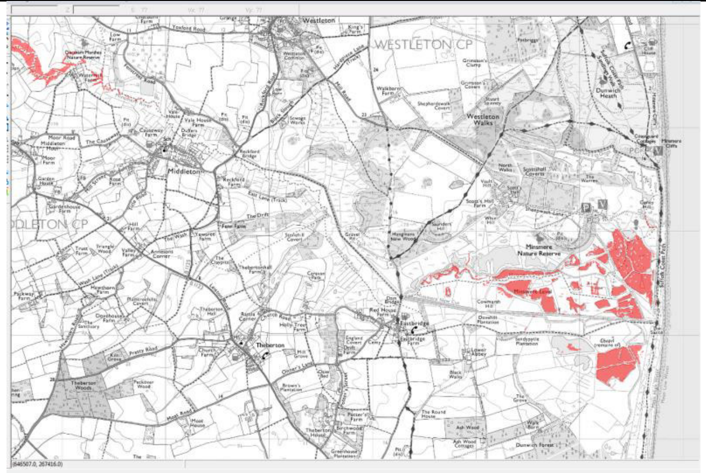
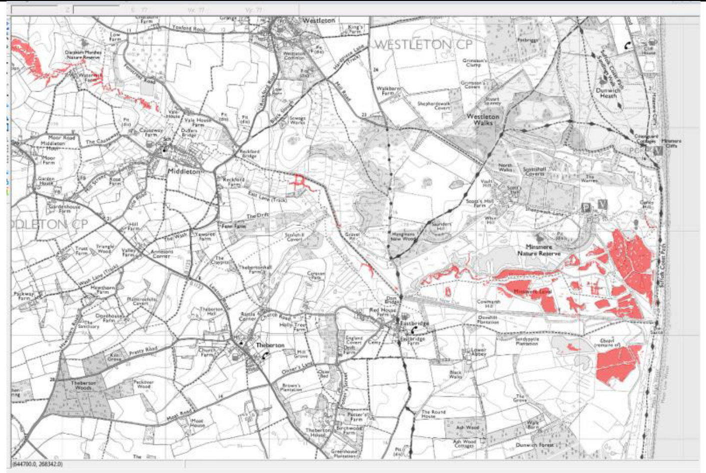
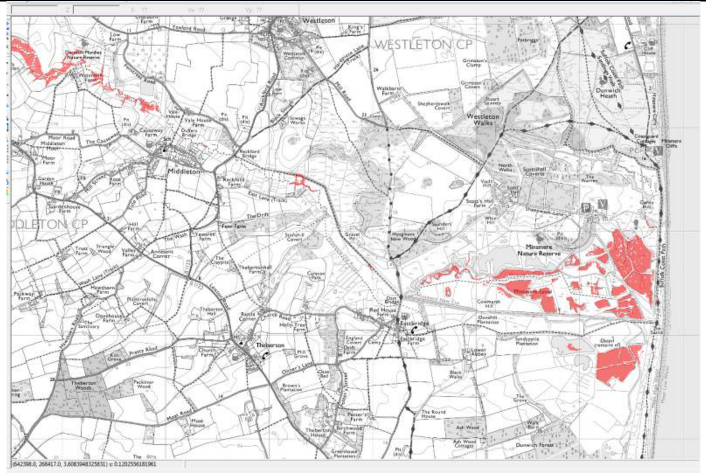
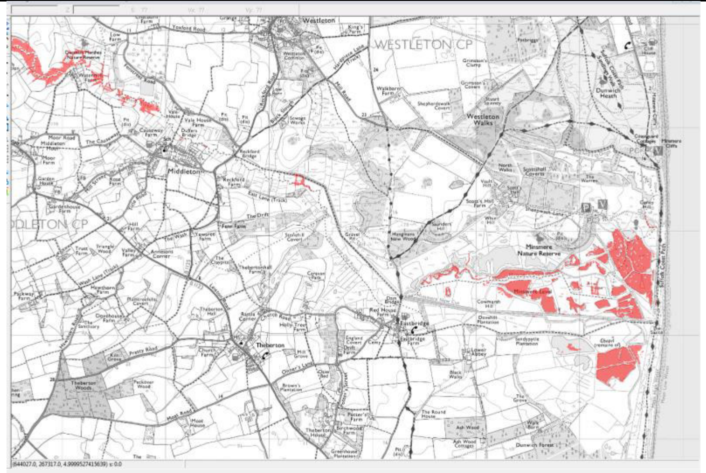
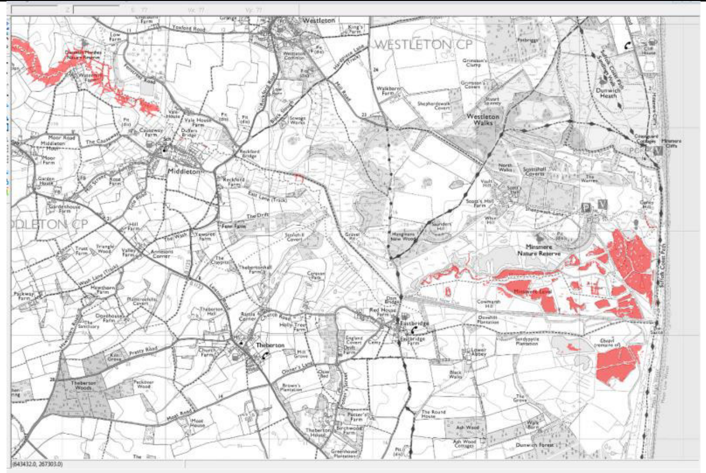
Time (hours)	Minsmere New Cut channel roughness 0.045 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I062_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.055 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I063_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.065 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I064_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.085 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I065_T027_NR_Rating.ief)
96				
120				
144				

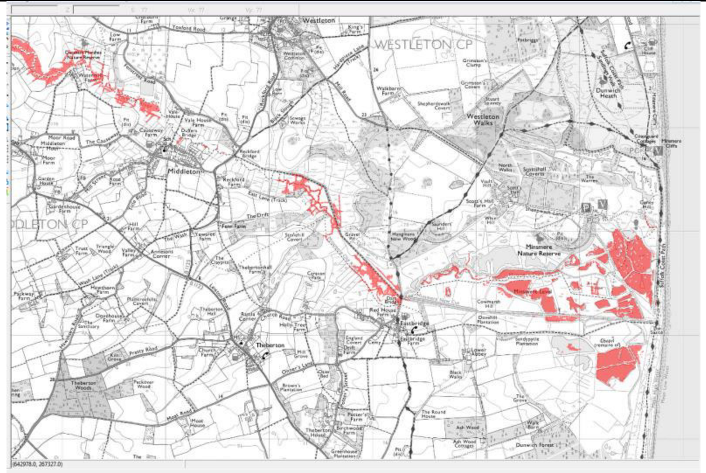
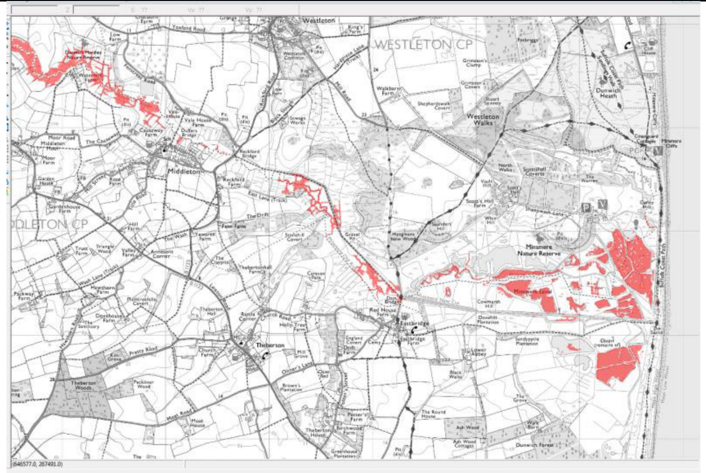
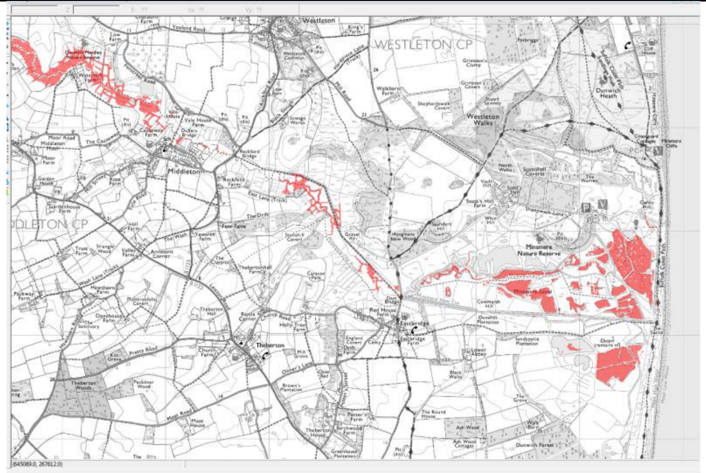
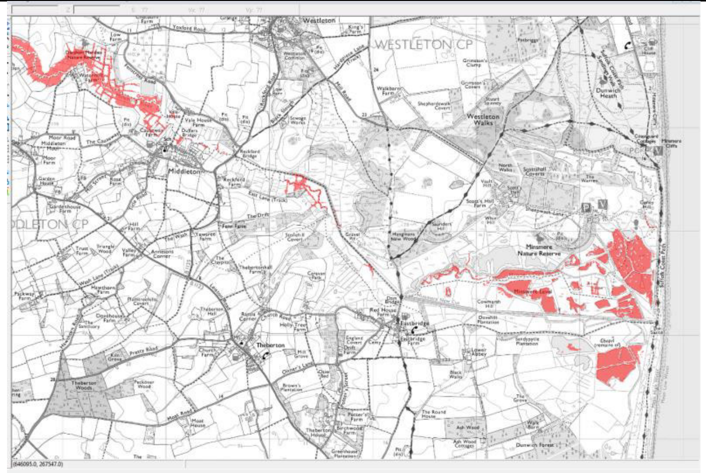
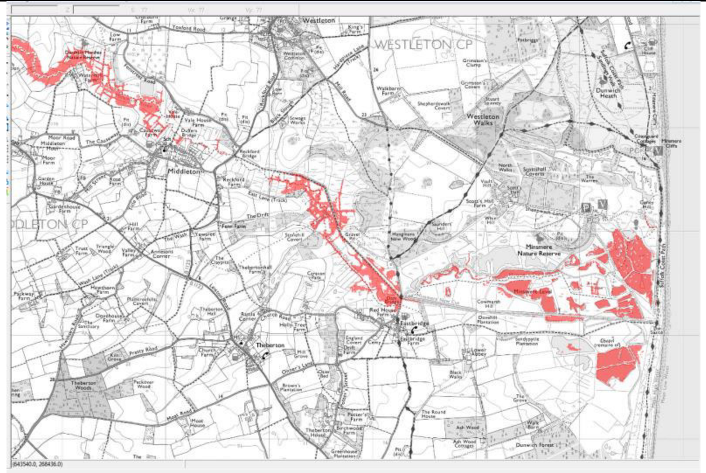
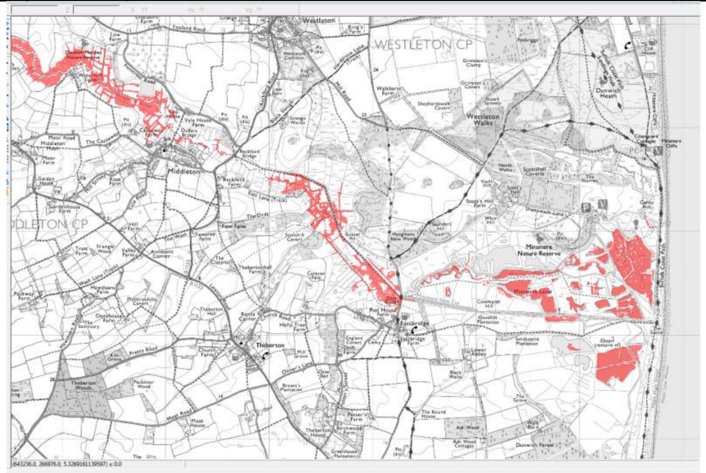
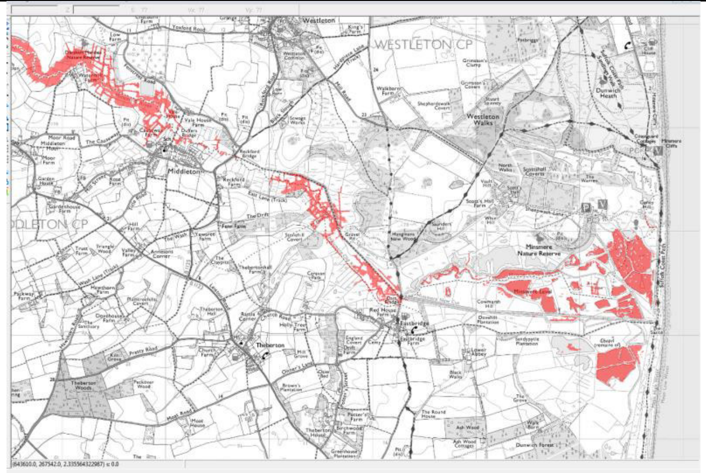
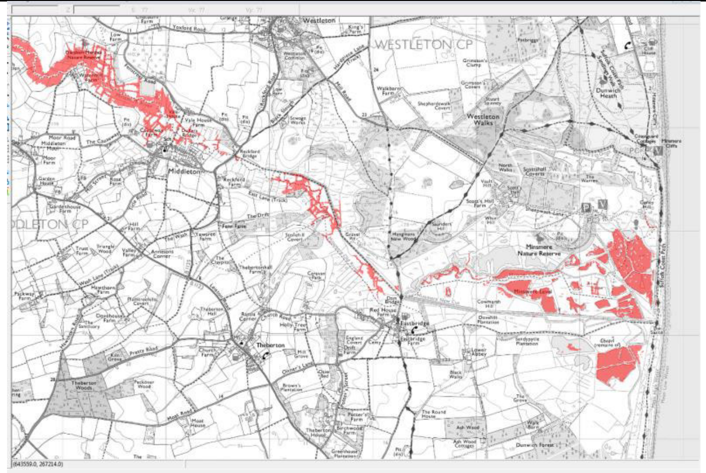
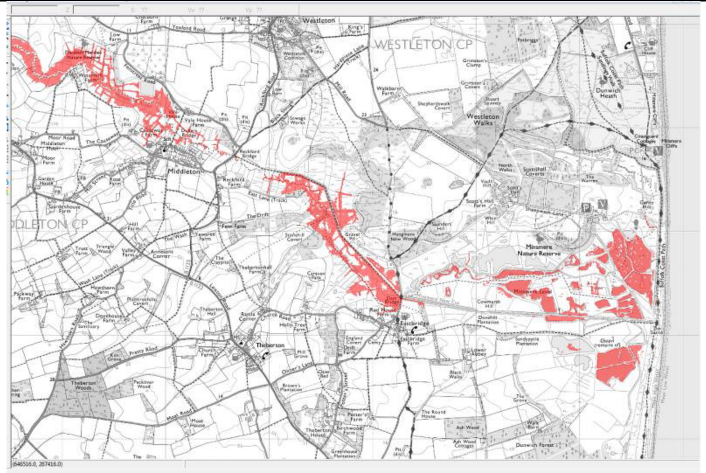
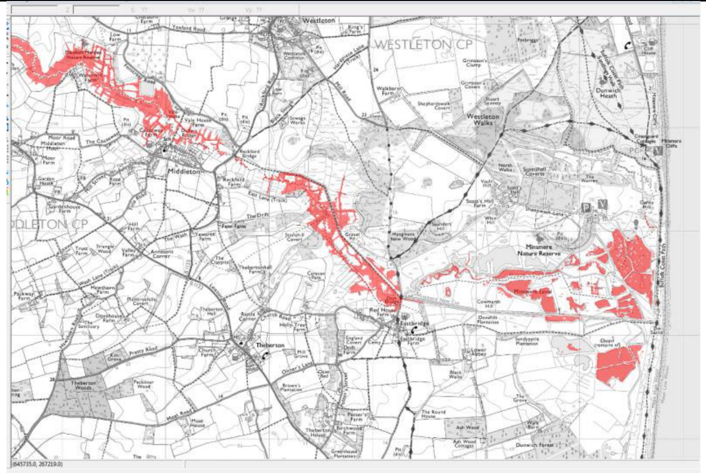
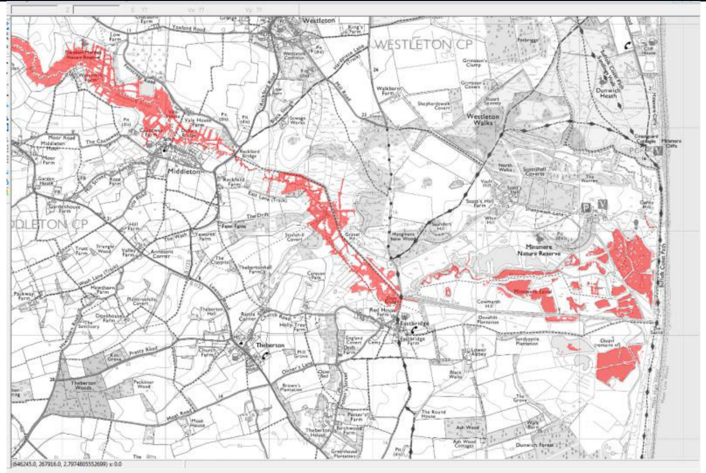
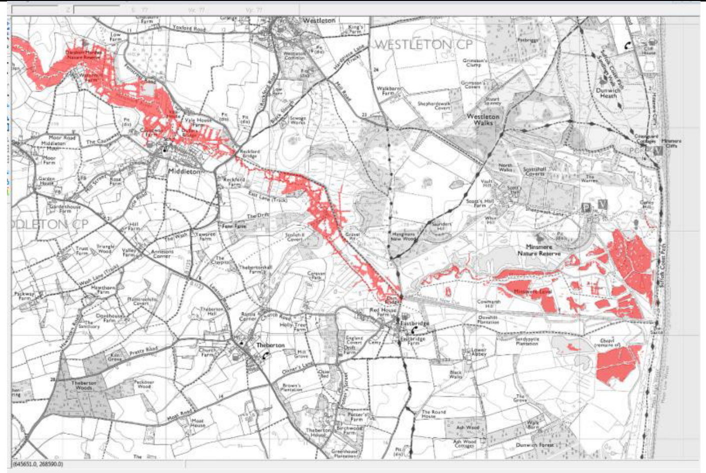
Time (hours)	Minsmere New Cut channel roughness 0.045 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I062_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.055 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I063_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.065 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I064_T027_NR_Rating.ief)	Minsmere New Cut channel roughness 0.085 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I065_T027_NR_Rating.ief)
168				
192				

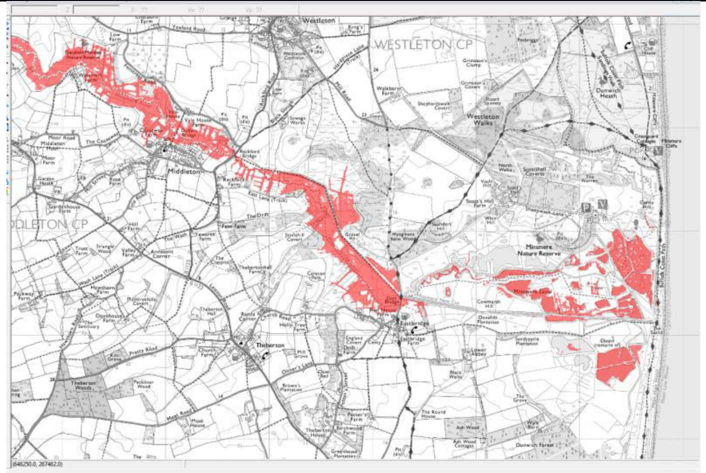
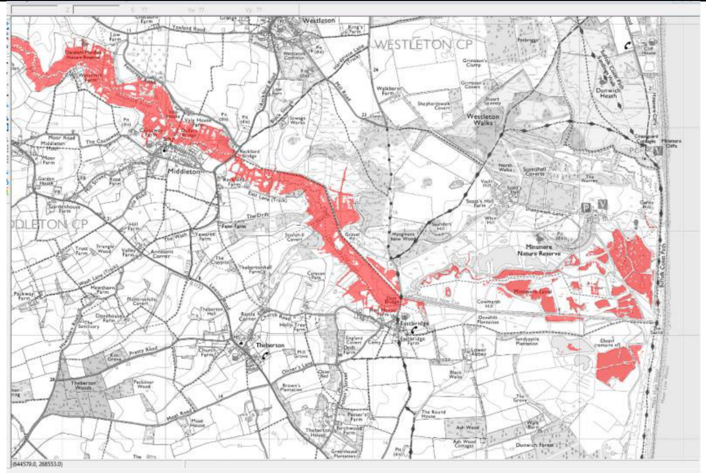
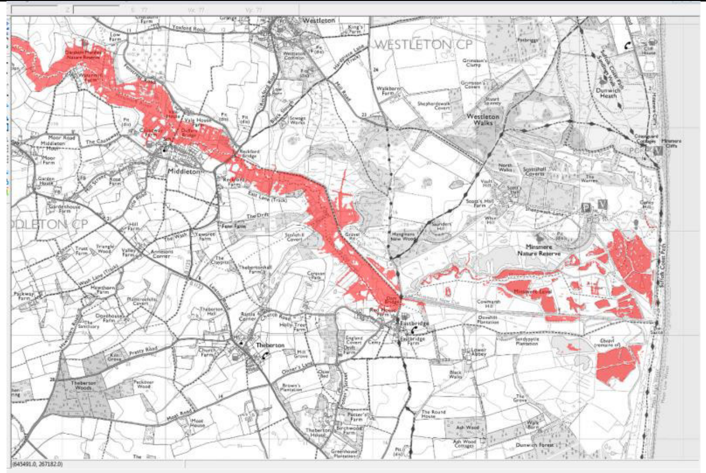
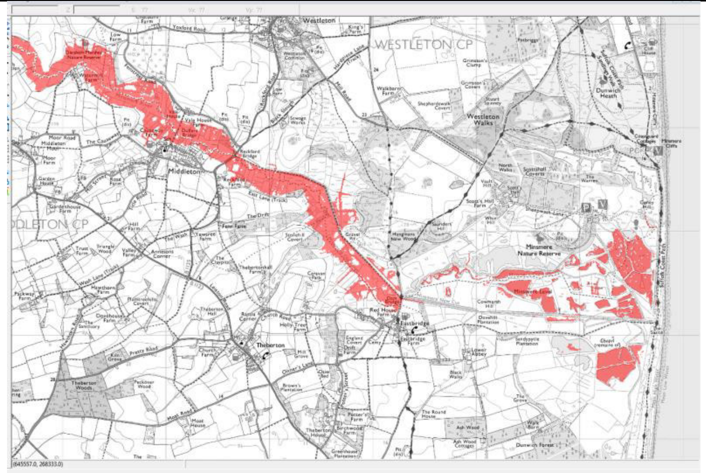
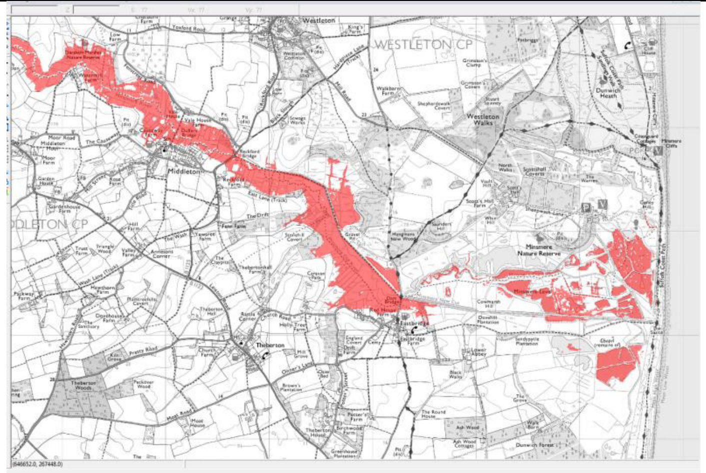
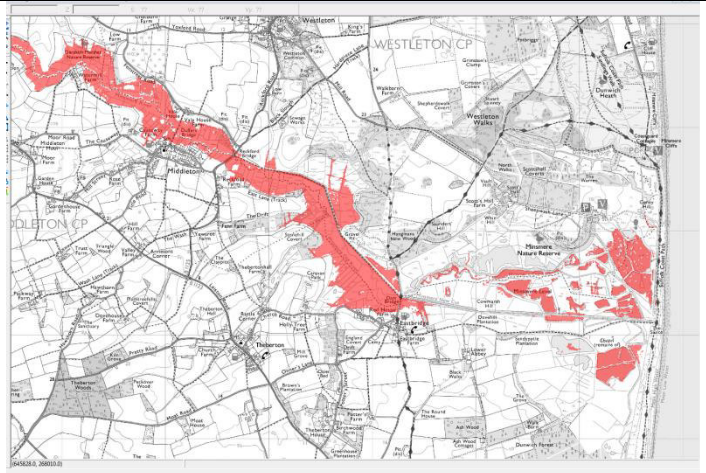
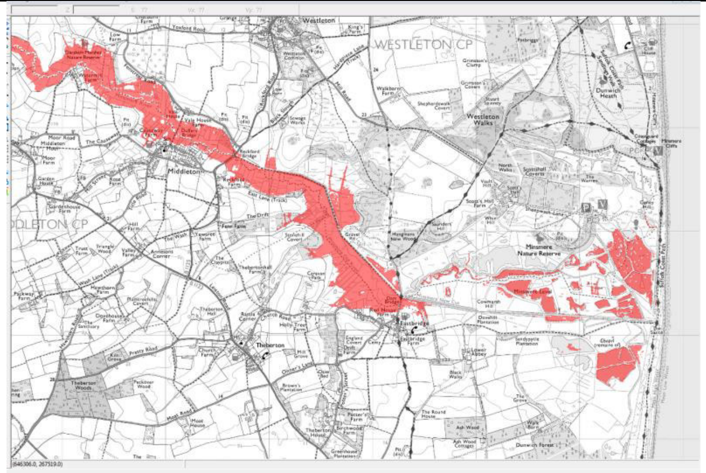
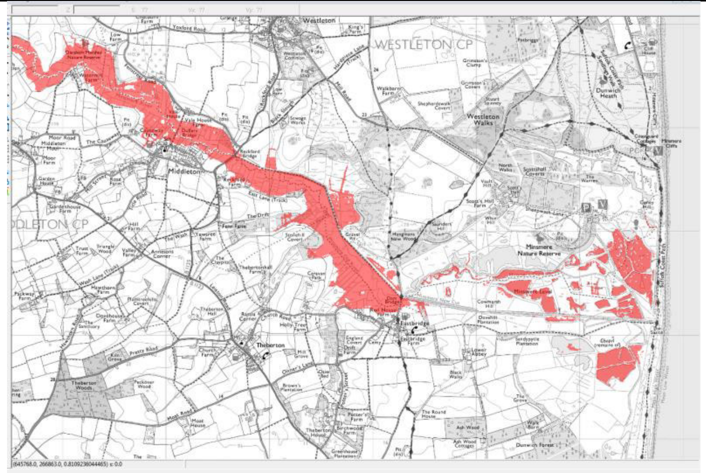
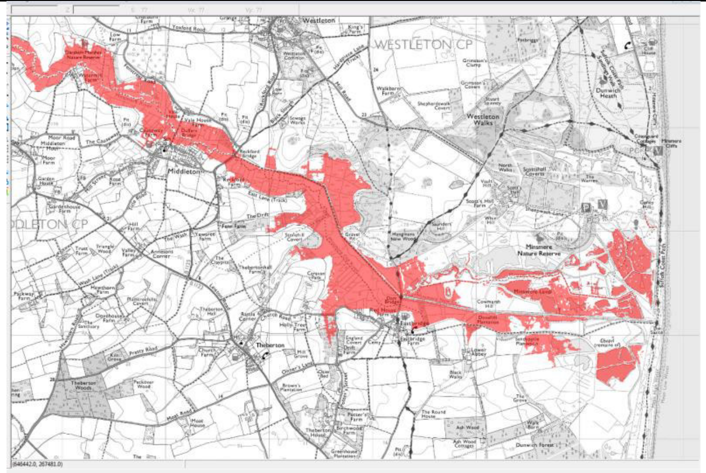
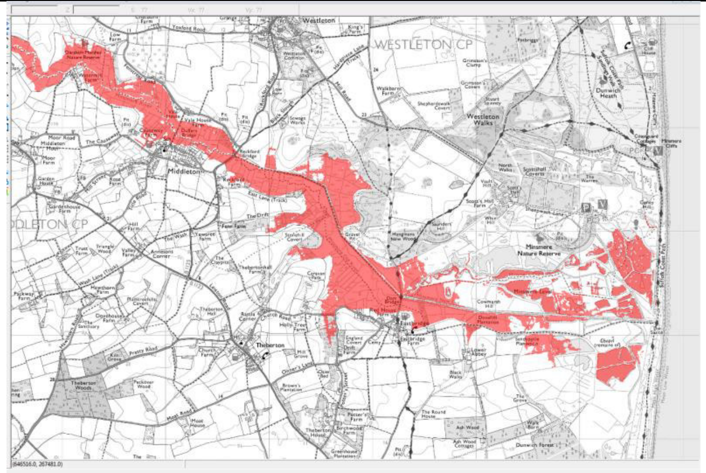
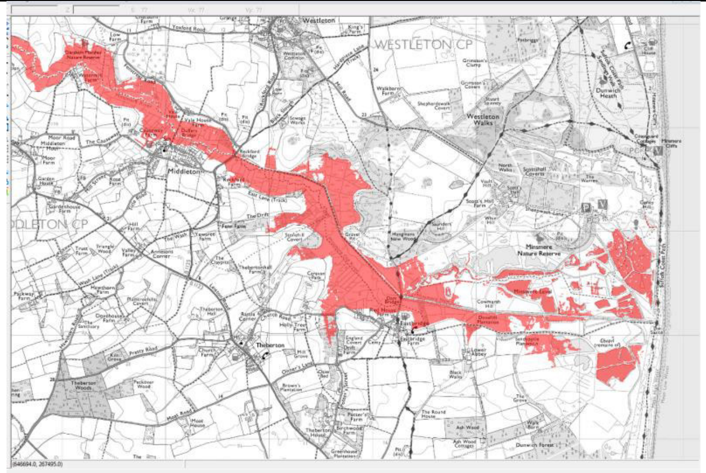
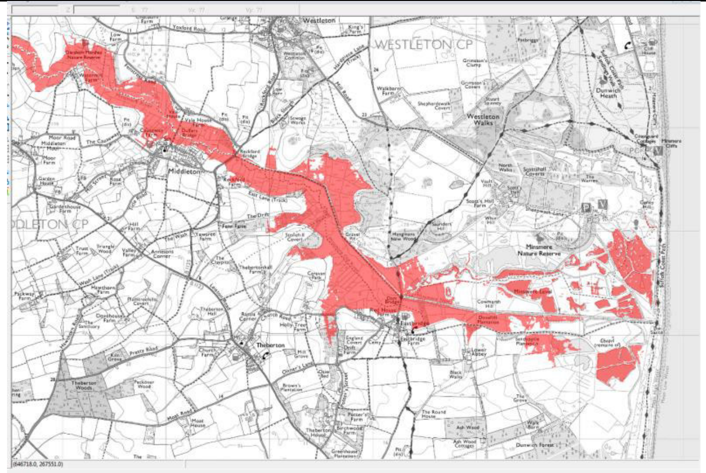
Time (hours)	Commentary
0 - 24	Initial Conditions for all model runs inundate floodplain below 0.20m AOD
29	Water spills into the washland (~600-1,000m upstream of Minsmere Sluice) on the right hand bank of the MNC in the lowest roughness coefficient (0.045) scenario only
31	Water spills into the washland in all scenarios In the highest roughness (0.085) coefficient scenario, water begins to spill from the right hand bank of the MNC between Reckford and Dam Bridges
35	In the 0.085 and 0.065 roughness coefficient scenarios, water spills from two locations on the right hand bank of the MNC between Reckford and Dam Bridges In the 0.055 and 0.045 roughness coefficient scenarios, water spills from only the observed downstream location on the right hand bank of the MNC as for the higher roughness coefficient scenarios, however water also spills over the left hand bank of the MNC into the meadows upstream of Dam Bridge Road. For the 0.065 roughness coefficient scenario spilling is observed from all locations
36	In the 0.045 and 0.055 roughness coefficient scenarios water spills over the left hand bank of the MNC opposite the washland into the RSPB reserve
44	Water spills into the RSPB reserve opposite the washland in all scenarios other than the highest roughness coefficient scenario (0.085) Water is spilling from both locations on the right hand bank of the MNC between Reckford and Dam Bridges in all scenarios Water spills into Meadow Marsh on the left hand bank of the MNC upstream of Dam Bridge in the 0.065 and 0.085 roughness coefficient scenarios
60	By 60 hours the lower roughness coefficient scenarios exhibit greater extents in the RSPB reserve than the high roughness coefficient scenarios but smaller extents in the right hand floodplain of the old course of the MNC
70	Water spills over the right hand bank of IDB Drain No 7 in the high (0.085) roughness coefficient scenario which is not seen in other scenarios
74	All scenarios spill over the right hand bank of IDB Drain No 7
96	Flood extents are similar upstream of Dam Bridge and in the Southern Levels in all scenarios There is a decreasing extent of flooding in the RSPB reserve with an increase in roughness coefficient
120	All flood extents are similar
144	All flood extents are similar
168	All flood extents are similar
192	All flood extents are similar

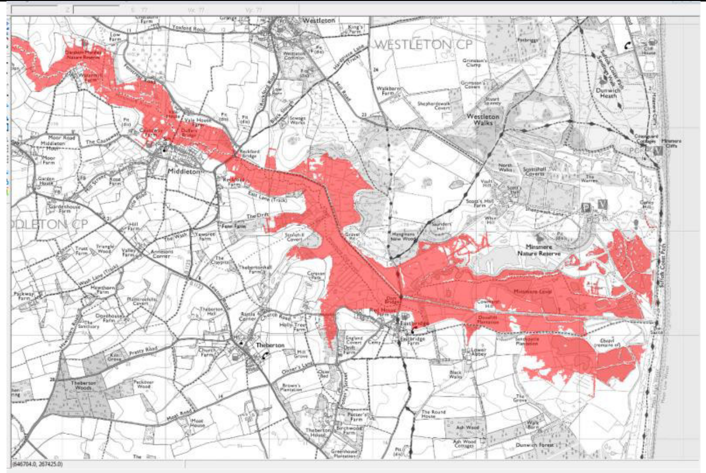
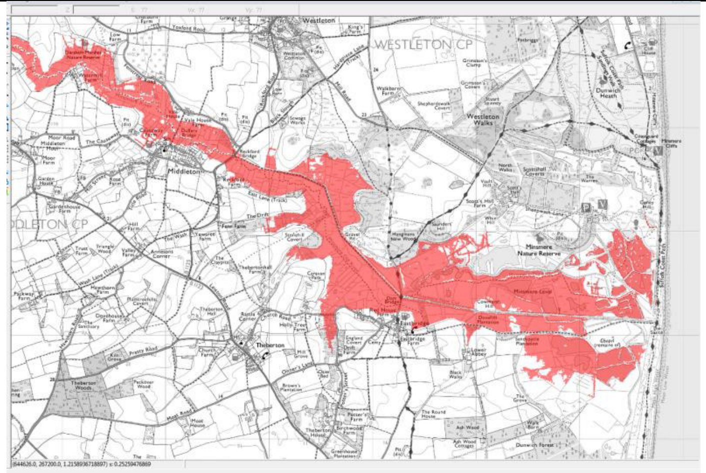
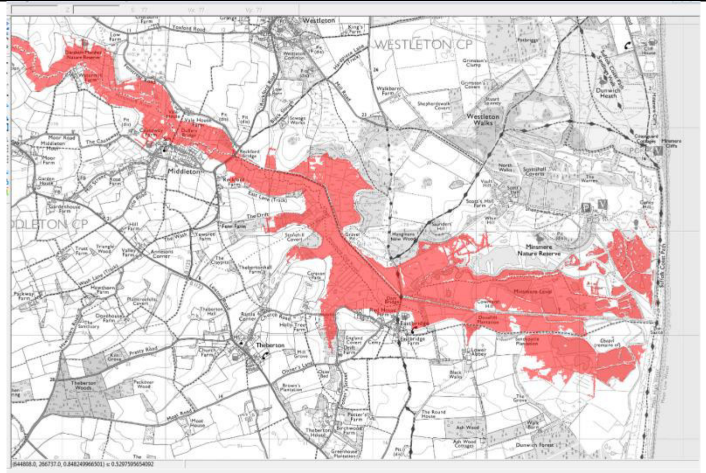
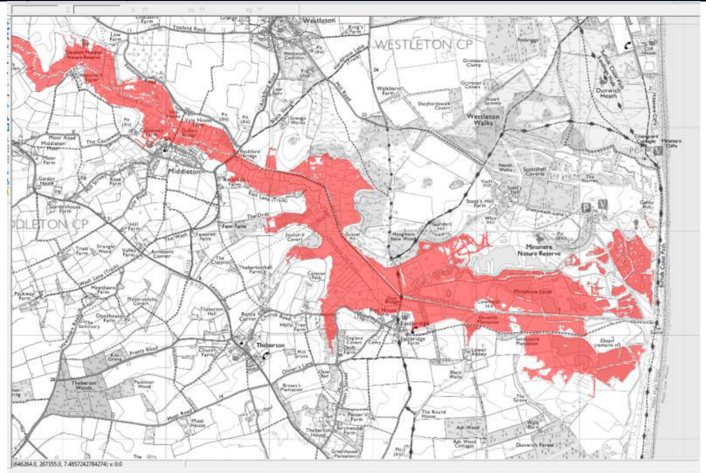
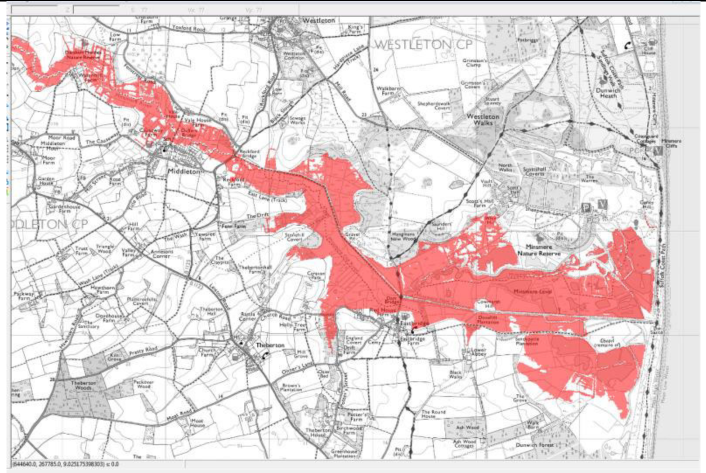
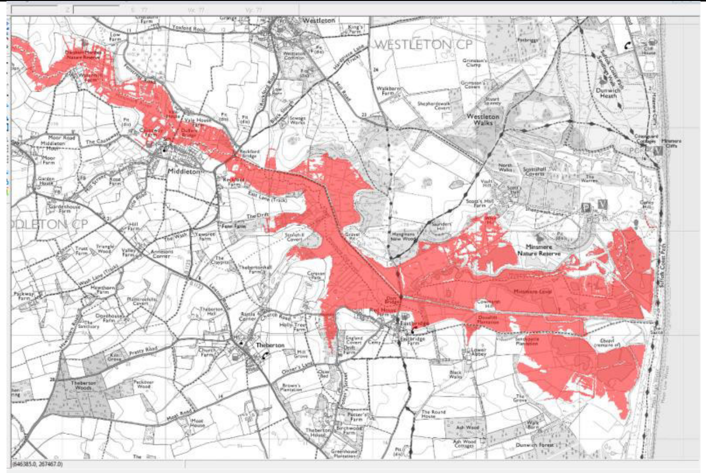
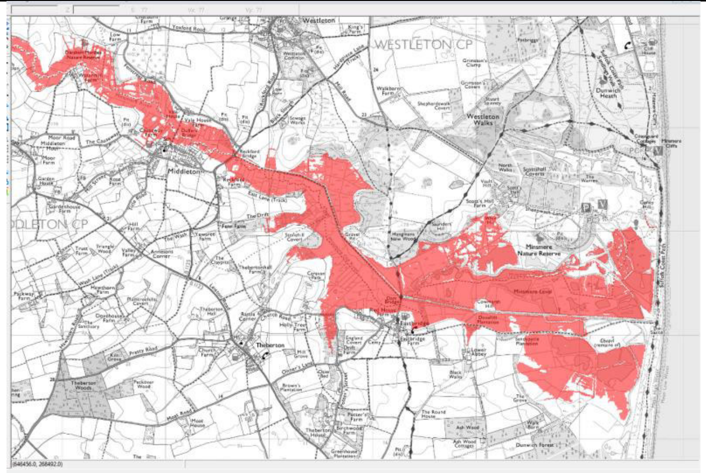
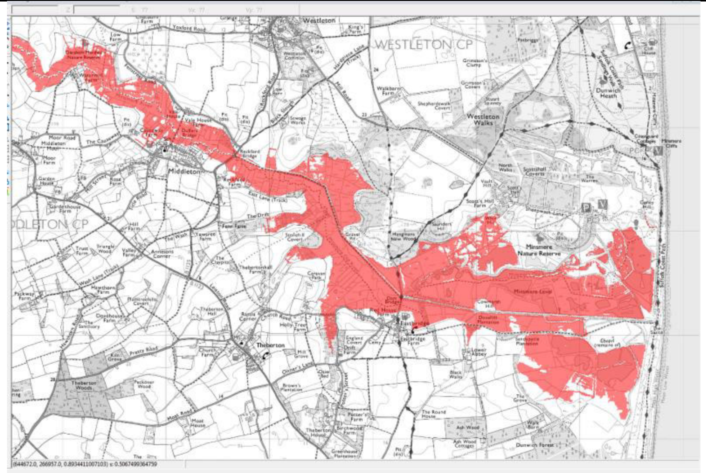
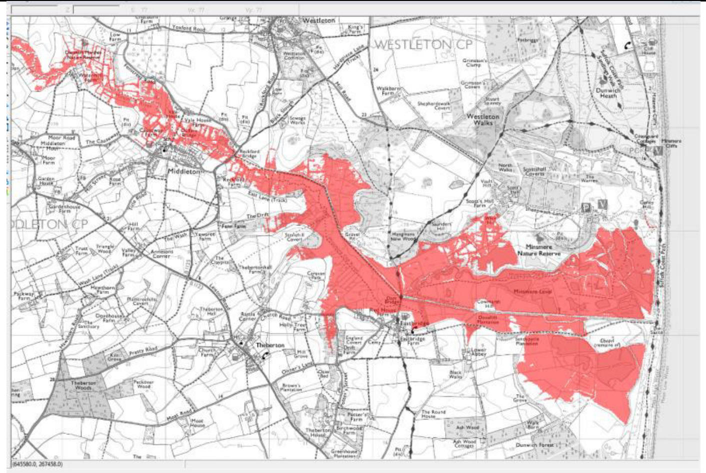
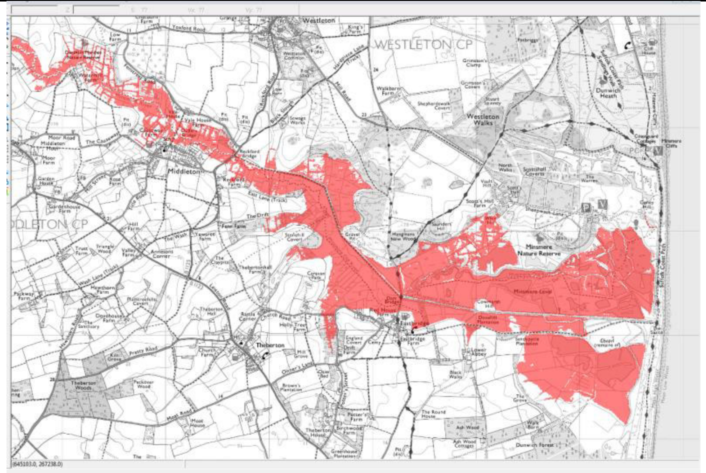
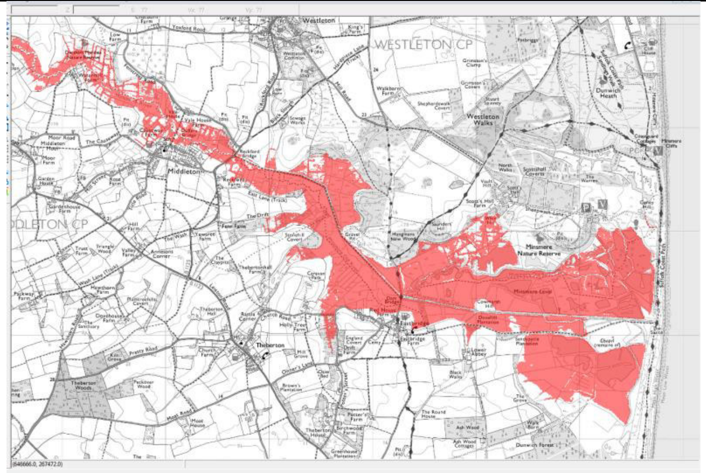
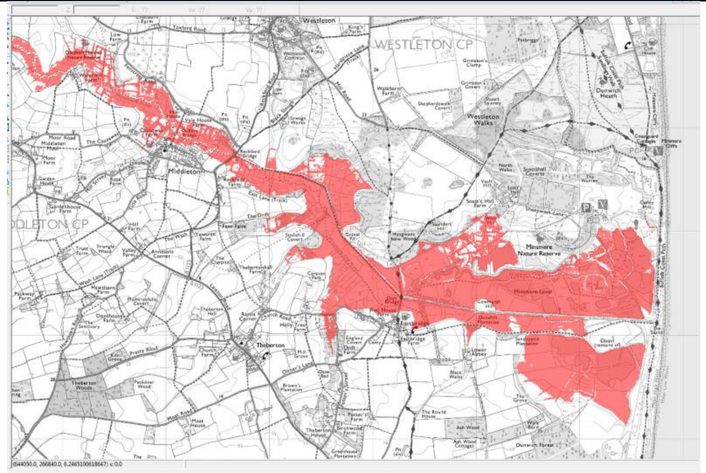
APPENDIX B - Observations of flood mechanisms

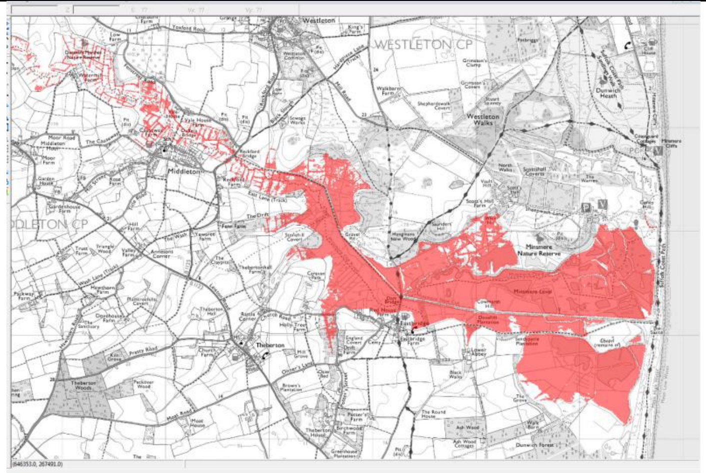
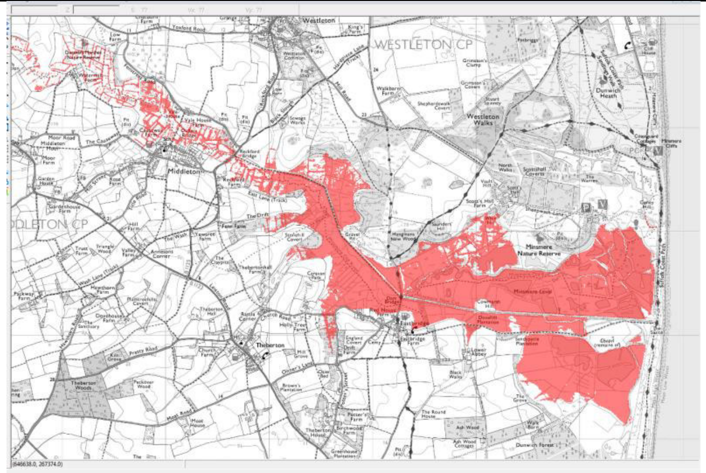
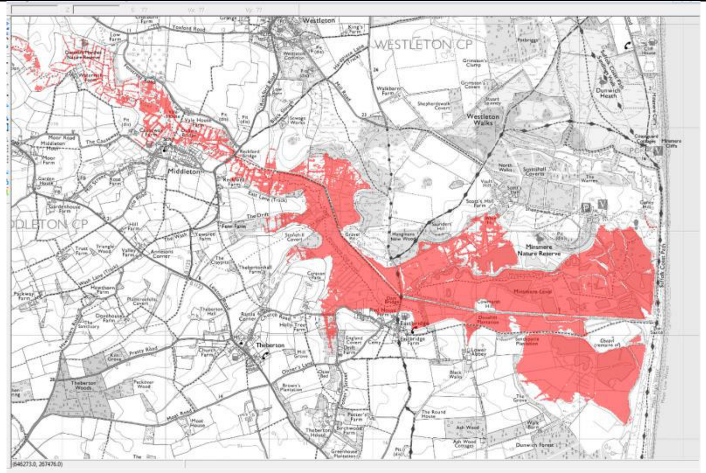
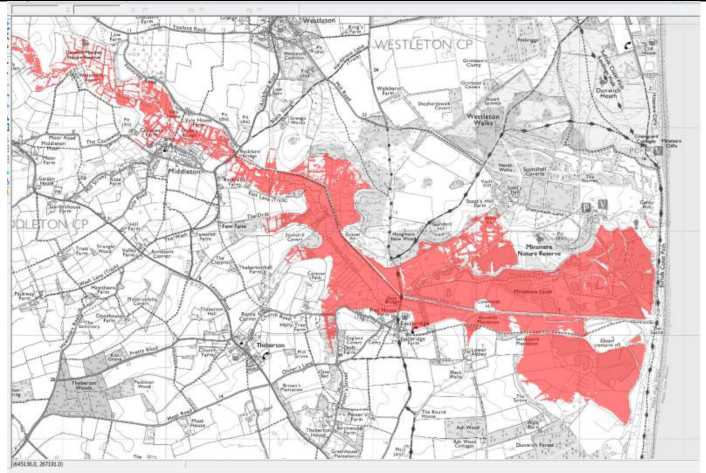
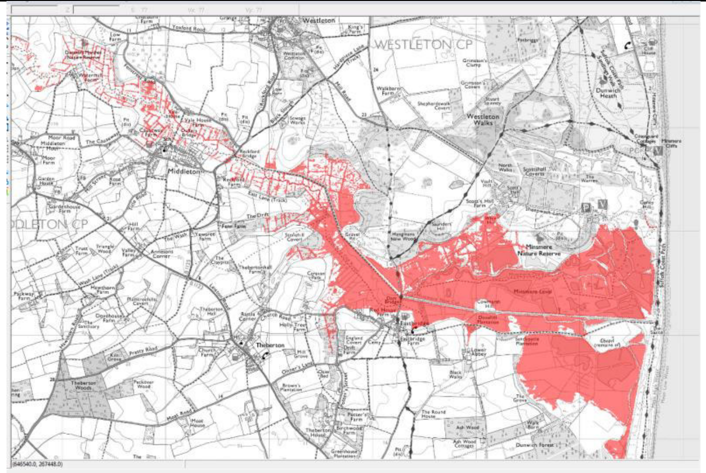
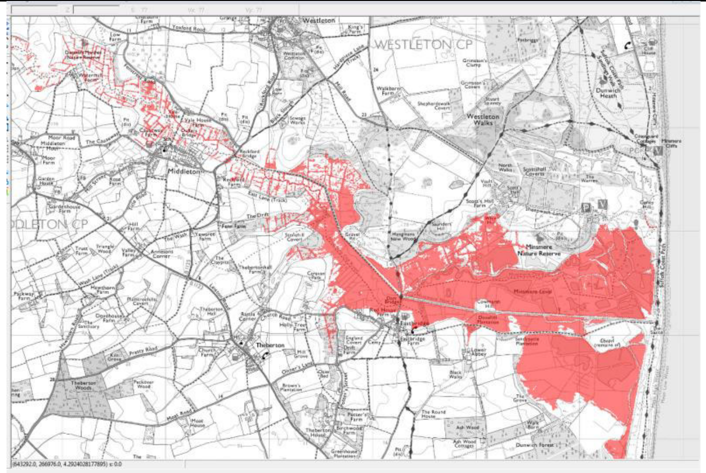
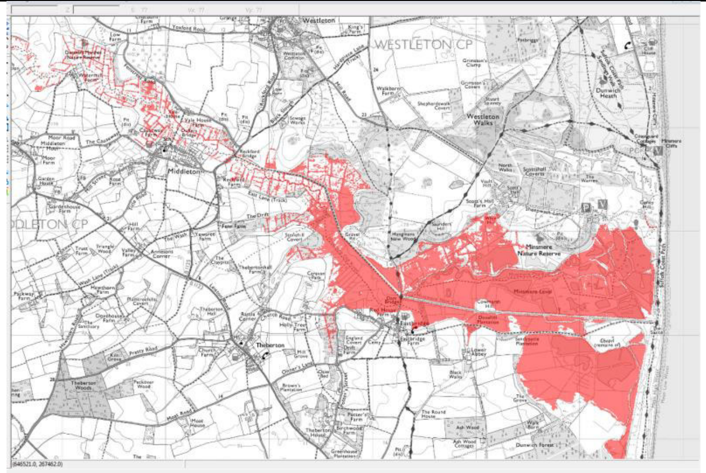
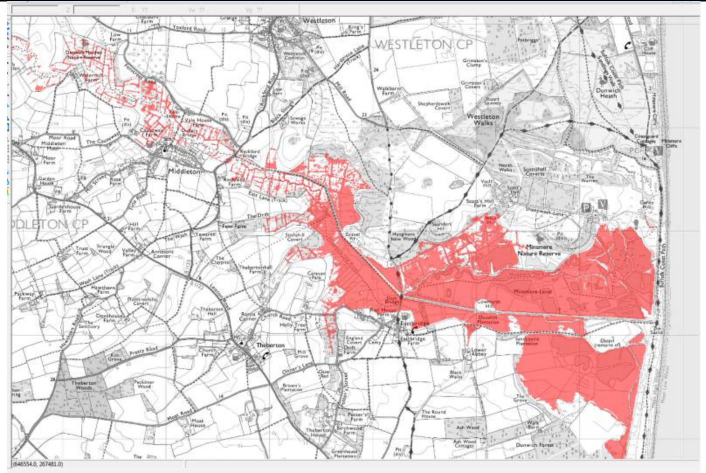
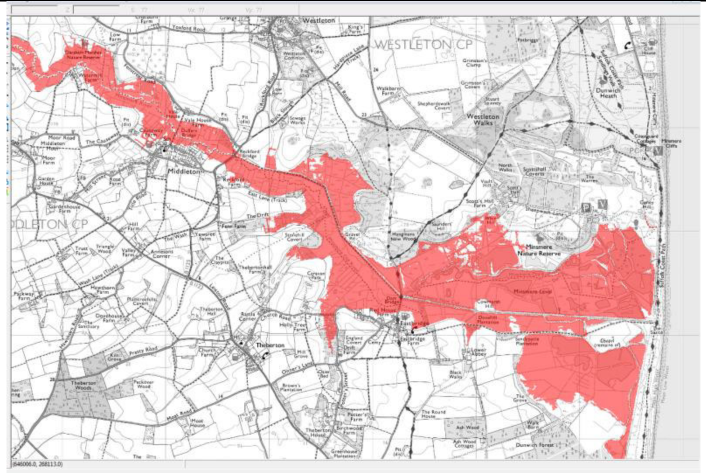
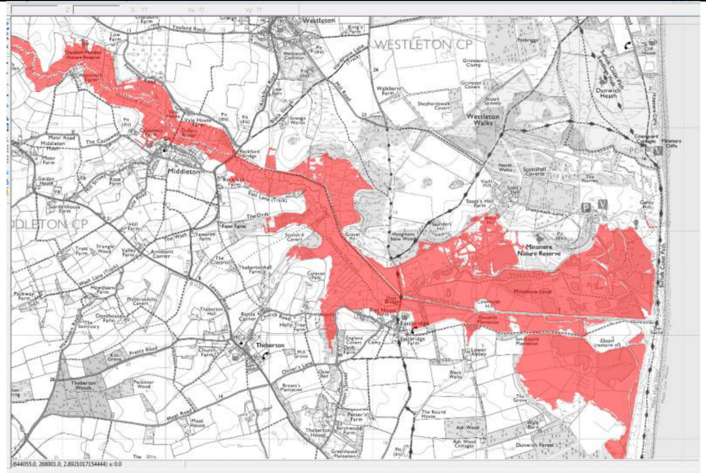
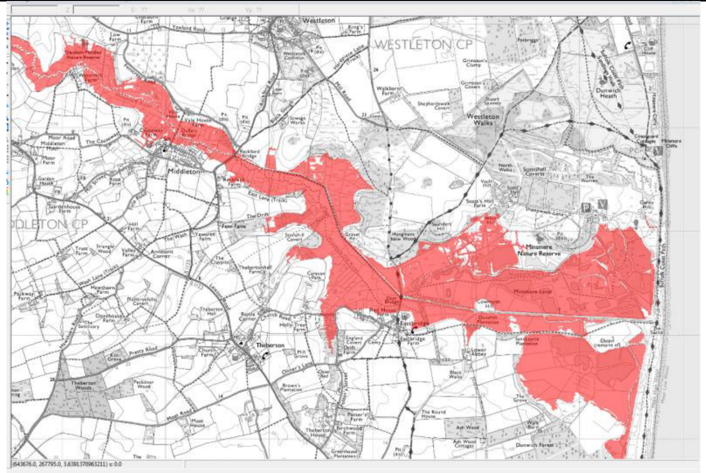
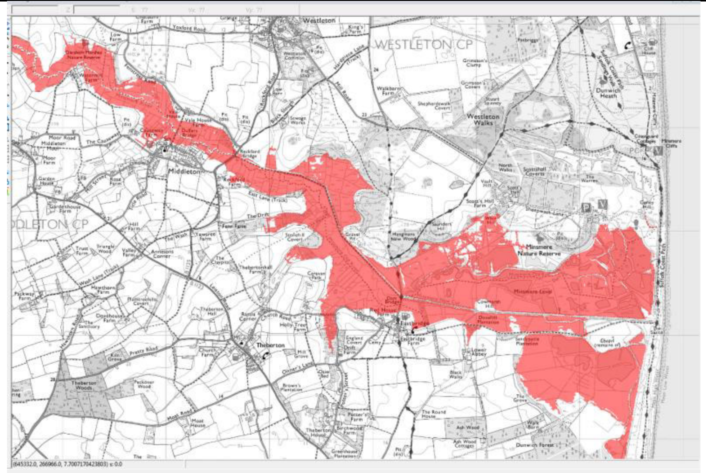
Appendix B – Observations of Flood Mechanisms from the Extended Rating Models Utilising a Hydrograph

Time (hours)	Minsmere New Cut channel roughness 0.045 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I066_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.055 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I067_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.065 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I068_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.085 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I069_T028_NR_Rating.ief)
0				
8				
9				

Time (hours)	Minsmere New Cut channel roughness 0.045 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I066_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.055 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I067_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.065 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I068_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.085 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I069_T028_NR_Rating.ief)
10				
11				
12				

Time (hours)	Minsmere New Cut channel roughness 0.045 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I066_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.055 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I067_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.065 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I068_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.085 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I069_T028_NR_Rating.ief)
14				
16				
20				

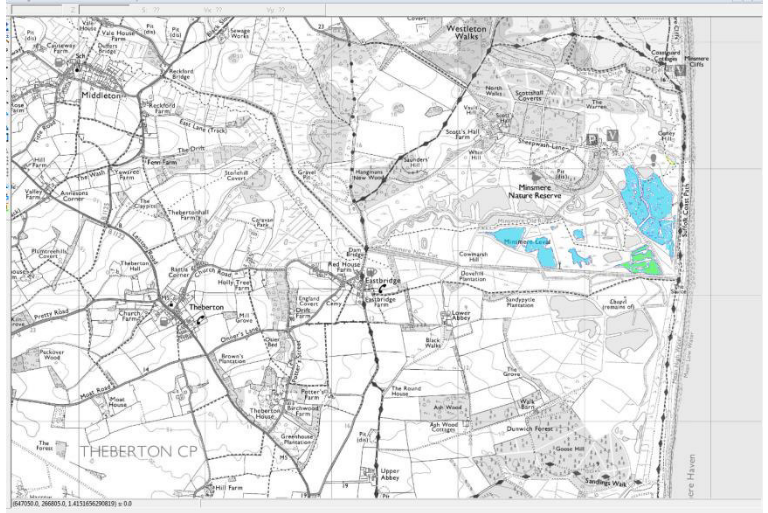
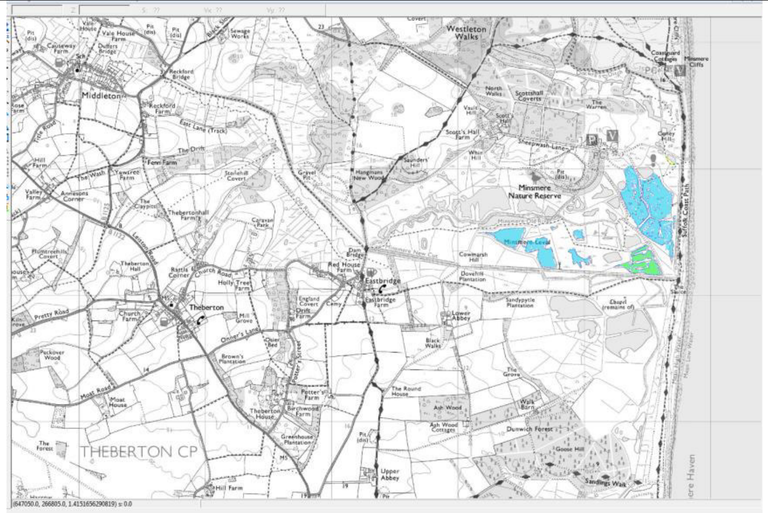
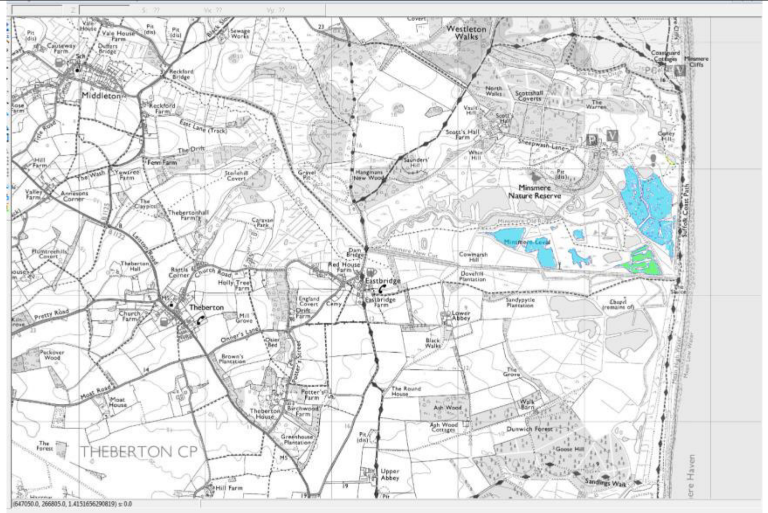
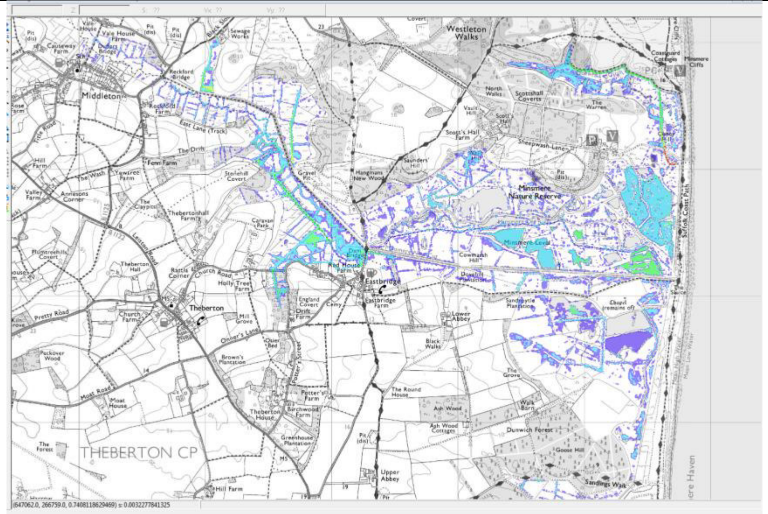
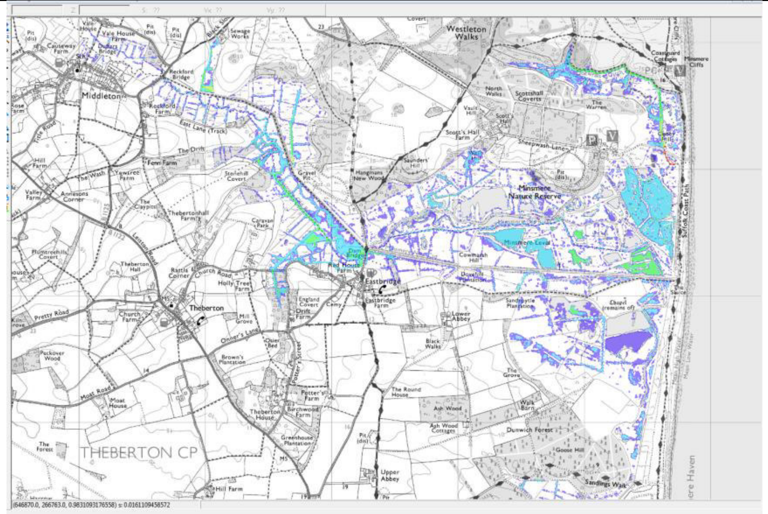
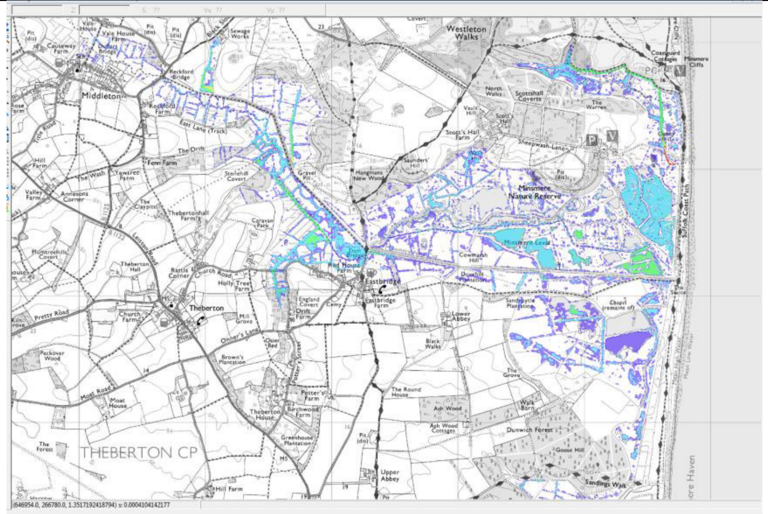
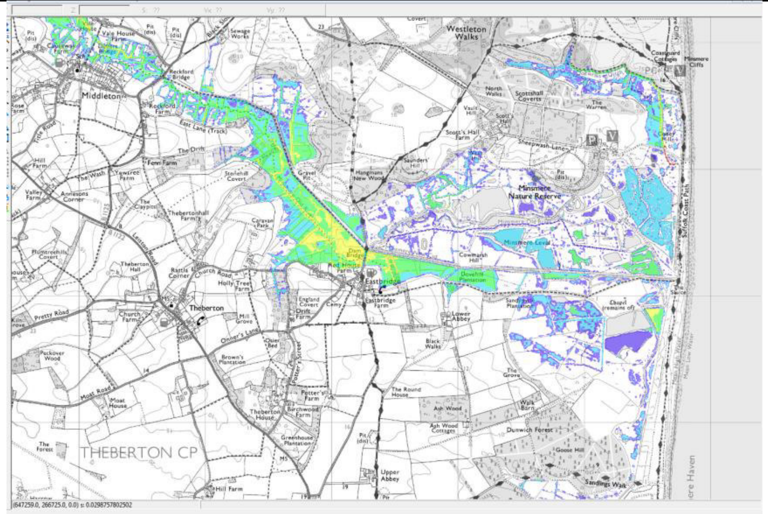
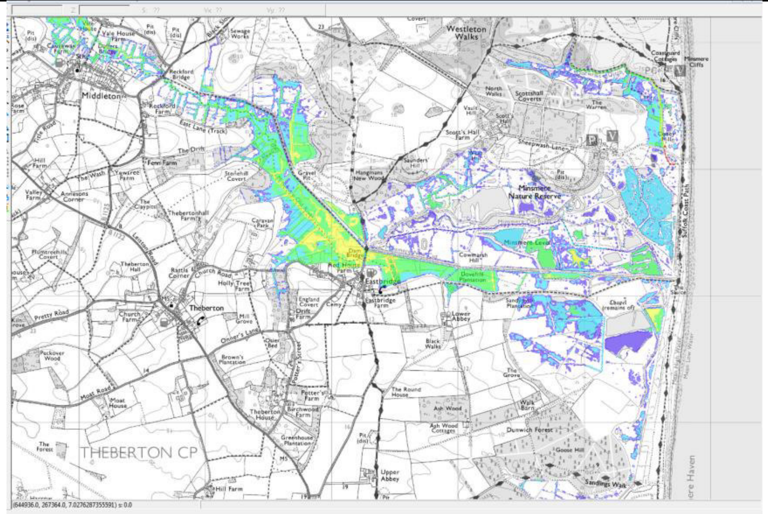
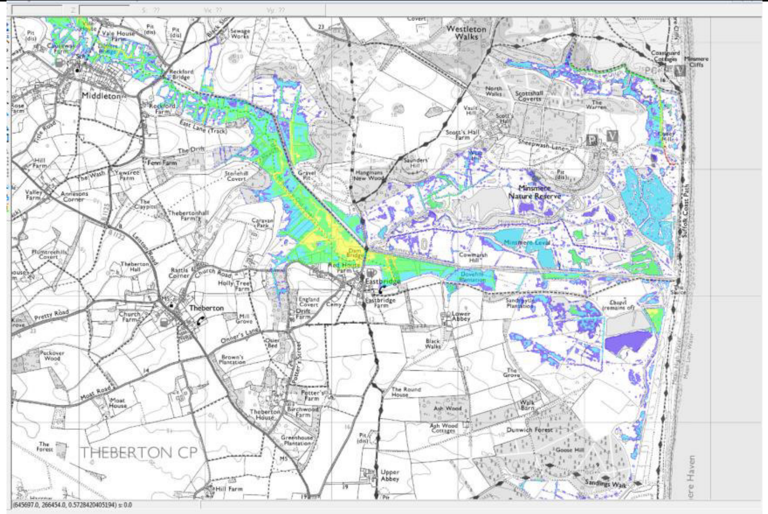
Time (hours)	Minsmere New Cut channel roughness 0.045 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I066_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.055 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I067_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.065 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I068_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.085 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I069_T028_NR_Rating.ief)
24				
28				
32				

Time (hours)	Minsmere New Cut channel roughness 0.045 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I066_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.055 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I067_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.065 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I068_T028_NR_Rating.ief)	Minsmere New Cut channel roughness 0.085 Constant tide boundary at 0.00m AOD No rainfall and constant baseflow from other inputs (Build_I069_T028_NR_Rating.ief)
36				
48				
Max				

Time (hours)	Commentary
0	Initial Conditions for all model runs inundate floodplain below 0.20m AOD
8	Flood hydrograph begins to increase Increasing flood extents upstream of Reckford Bridge are observed with increasing roughness coefficient in the MNC (roughness increases looking left to right across the diagrams)
9	Additional to mechanisms at 8 hours, flooding occurs over the right hand bank of the MNC at two locations between Reckford and Dam (at Eastbridge) Bridges The volume of water spilling at these two locations decreases with increasing roughness (due to greater attenuation upstream of Reckford Bridge with increased roughness coefficient)
10	Flood extents downstream of Reckford Bridge are greater for lower roughness scenarios For all four scenarios water begins to spill over the left hand bank of the MNC to Meadow Marsh and into the meadow upstream of Dam Bridge Road The washland on the right hand bank of the MNC (~600-1,000m upstream of Minsmere Sluice) begins to fill for all scenarios other than the high (0.085) roughness scenario
11	Flood extents increase with no new mechanisms observed
12	For the lowest roughness coefficient scenario (0.045) water spills into the RSPB reserve opposite the washland (~800m upstream of Minsmere Sluice)
14	There is extensive flooding upstream of Dam Bridge in all scenarios Spilling over the left hand bank opposite the washland still only occurs in the low (0.045) roughness coefficient scenario
16	Flood extents increase and flooding occurs over the left hand bank opposite the washland for the 0.055 roughness scenario Water spills over the lowest parts of the road to the south of Dam Bridge in all scenarios
20	Water spills over the right hand bank of Drain No 7 into the Southern Levels in all scenarios Water spills over the northern side of Dam Bridge Road into the western extent of the RSPB reserve Water does not spill into the RSPB reserve in the highest (0.085) roughness scenario opposite the washland on the MNC
24	By 24 hours the flood peak has reached the downstream extent of the reach (downstream of Eastbridge) and all scenarios exhibit a similar flood extent
28	Flood extents are similar for all scenarios
32	Flood extents are similar for all scenarios
36	Flood extents are similar for all scenarios
48	Flood extents are similar for all scenarios
Max	Flood extents are similar for all scenarios

APPENDIX C - January 2003 Selected Model Screen Shots

Appendix C – January 2003 Selected Model Screen Shots

Time (hours)	High Roughness (0.06-0.065) 50% PR Initial Level 0.00m AOD (SZC_003c_Jan03_001_IC0)	Low Roughness (0.035-0.040) 50% PR Initial Level 0.00m AOD (SZC_005c_Jan03_001_IC0)	High Roughness (0.06-0.065) 40% PR Initial Level 0.00m AOD (SZC_003c_Jan03_002_IC0)	
0				**Depth (m)** 0.05 - 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 1.00 > 1.00
12				**Depth (m)** 0.05 - 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 1.00 > 1.00
24				**Depth (m)** 0.05 - 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 1.00 > 1.00