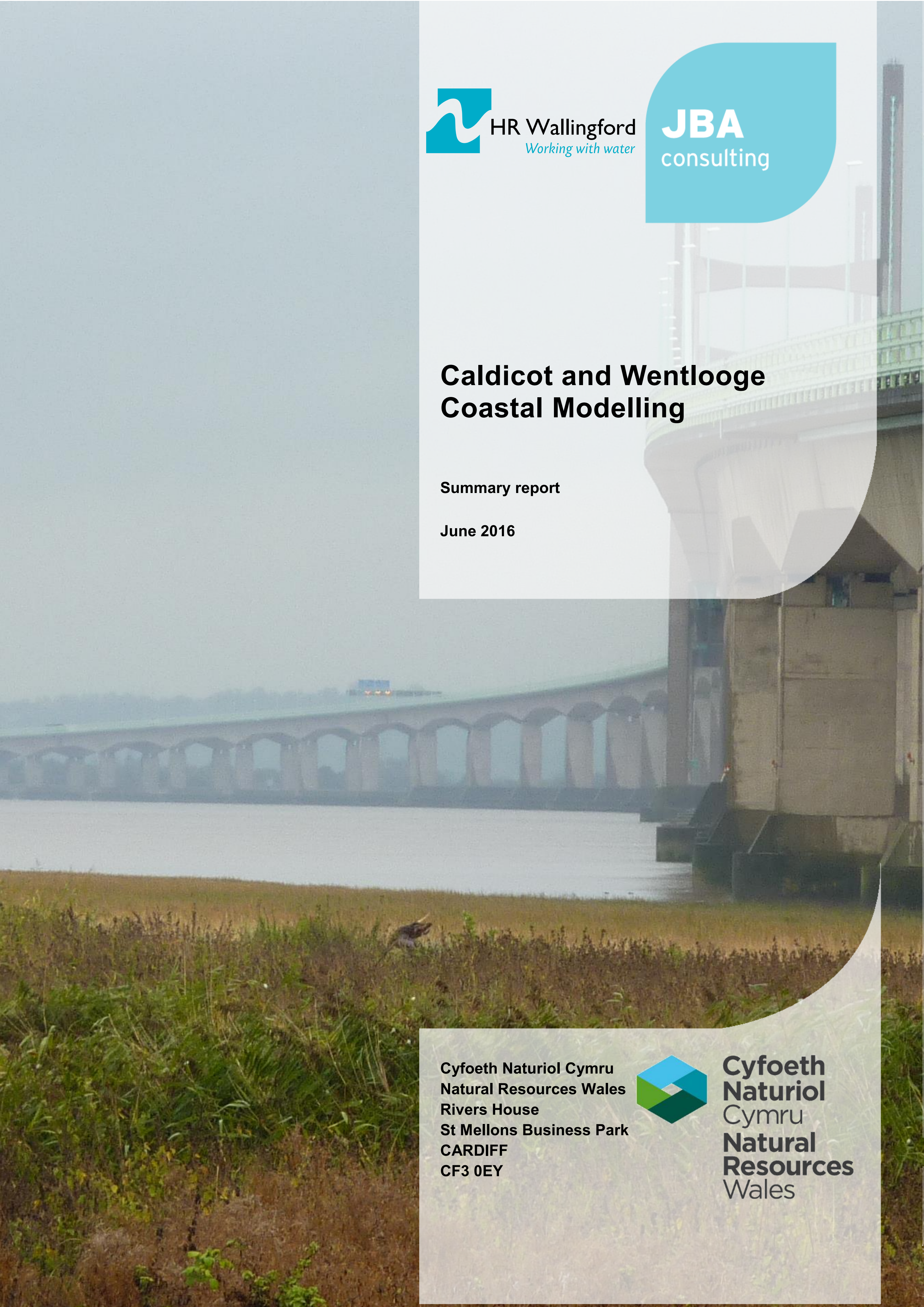


HR Wallingford
Working with water

JBA consulting

Caldicot and Wentlooge Coastal Modelling

Summary report

June 2016

Cyfoeth Naturiol Cymru
Natural Resources Wales
Rivers House
St Mellons Business Park
CARDIFF
CF3 0EY

Cyfoeth Naturiol Cymru
Natural Resources Wales

JBA Project Manager

Matthew Hird
South Barn
Broughton Hall
Skipton
North Yorkshire
BD23 3AE

Revision History

Revision Ref / Date Issued	Amendments	Issued to
Draft report/June 2016		Nick Steele
Final report/August 2016	Addressed comments from draft report	Nick Steele

Contract

This report describes work commissioned by Natural Resources Wales, by a letter dated 19th August 2014. Natural Resources Wales' representative for the contract was Nick Steele of Natural Resources Wales. Matthew Hird, Jennifer Hornsby and Kathryn Williams of JBA Consulting carried out this work.

Prepared by .... Kathryn Williams BSc MSc, Jennifer Hornsby BSc and Matthew Hird BSc MCIWEM C.WEM CEnv CSci

JBA

Prepared by .... Dominic Hames, Mohamed Ahmed Ali Mohamed Hassan

HR Wallingford

Reviewed by .... Daniel Rodger BEnvSci, MEng, PGDipGIS, PGCert Coastal Eng, CMarEng CEng MIMarEST MICE

Chartered Senior Coastal Engineer

Approved by .... Mark Lawless BSc MSc PhD CSci CEnv MCIWEM C.WEM

Director

Purpose

This document has been prepared as a Final Report for Natural Resources Wales. JBA Consulting accepts no responsibility or liability for any use that is made of this document other than by the Client for the purposes for which it was originally commissioned and prepared.

JBA Consulting has no liability regarding the use of this report except to Natural Resources Wales.

Copyright

Carbon Footprint

A printed copy of the main text in this document will result in a carbon footprint of 726g if 100% post-consumer recycled paper is used and 924g if primary-source paper is used. These figures assume the report is printed in black and white on A4 paper and in duplex.

JBA is aiming to reduce its per capita carbon emissions.

Executive Summary

Introduction

This project was commissioned by Natural Resources Wales (NRW) to improve understanding of the coastal flood risk between Cardiff and Chepstow in South Wales. The main objectives of the study were:

- To provide evidence of coastal flood risk for the communities along the Severn Estuary between Cardiff and Chepstow
- To improve understanding of flood risk with revised sea-levels and joint probability assessments
- Review guidance on climate change and breach modelling
- Improve understanding of wave related risks that will provide a better appreciation of the potential impacts of major flood events
- Improve understanding of the conditions that will warrant the issue of flood warnings and provide clarity with respect to standards of protection (SoP).

The specific objectives of the project were:

- To identify areas that would flood from the sea, without the presence of defences, during the 1 in 200-year and 1,000-year extreme sea-level events. These events are consistent with Flood Zone 3 and Flood Zone 2 of the Flood Map respectively
- To map flood defences and identify areas that are likely to benefit from flood defences (ABDs) within the coastal floodplain
- To model flood defence breaches
- To model failure of tidal outfalls for tidal and fluvial flood events
- To model a number of climate change scenarios to indicate how the susceptibility of the study area to flooding from the sea may change in the next century
- To model a number of present day and climate change scenarios using the 95% confidence limits of extreme sea-levels to indicate the impact on predicted flood risk.

The dominant source of coastal flood risk in the coastal floodplain is related to the coincident occurrence of a high astronomical tide, the passage of a storm surge (generated by an atmospheric depression) and the effects of wind related waves. There is also a risk from fluvial flooding in the coastal Flood Zone which was an additional consideration for this study, focusing on the flapped outfalls failing on the main reens. Risk from surface water flooding is not considered in this study.

Study Area

The study area covers the Wentlooge and Caldicot Levels extending from Cardiff to Chepstow on the right bank (looking downstream) of the Severn Estuary. Much of the Wentlooge and Caldicot Levels are located below the mean high water spring tide level and are protected from coastal flooding by a system of sea defences. These defences essentially separate the land from the sea. It also contains low agricultural plains which are drained and pumped by an extensive network of artificial channels, including pumping stations at Collister Pill and Greenmoor.

A number of previous studies have been undertaken in the study area to examine tidal flood risk. These include the M4 proposed route modelling study (Arup, 2014), the SWAN modelling study (Deltares, 2011), Cardiff SFCA (Atkins, 2011), Newport SFRM modelling (JBA Consulting, 2011) and Severn Estuary FRM Strategy (Atkins, 2011). This new study largely represents an update to these previous studies, which will utilise improved ground level, new defence data, revised extreme sea-level data and more advanced modelling techniques to develop a new set of coastal flood risk maps.

Modelling approach

A suite of statistical and numerical models were used within this study. A statistical analysis was undertaken to revise extreme sea-levels estimates for the coastline, using tide gauge records to 2014. The latest multivariate statistical techniques were then used to calculate the probability of extreme coastal conditions occurring simultaneously, e.g. coincident extreme sea levels, wind and offshore wave conditions. A wave transformation model was used to transform offshore conditions through the surf zone to the base of coastal defences. A detailed wave overtopping assessment was undertaken using the nearshore data and local defence characteristics, which included breach and fragility analysis. The extreme sea-level and overtopped water was then simulated spilling over the coastal foreshore using two flood inundation models.

The outputs from the model simulations were a set of inundation maps that were used to update the Flood Zones, ABDs, Flood Alert and Flood Warning Areas. A series of visualisation tools were created to present the results of the model simulations and allow the impacts of a forecast flood event to be visualised.

Impact of change in sea-levels

The incorporation of additional years of observed tide gauge data to update the extreme sea-levels, resulted in very similar results for Newport and Avonmouth, when compared to the 2011 CFBD study, but increased levels at Mumbles, of 0.1m at the 200-year and 0.2m at the 1,000-year return periods. The se-levels between Newport and Mumbles are interpolated resulting in gradually increasing levels towards Mumbles. For Cardiff, where there is a known low spot in the defences, the levels are 0.1-0.2m higher for the 200-1,000-year events. All future defence upgrades should use the updated sea-levels.

Impact on coastal communities

Coastal modelling was completed to assess the coastal flood risk under a range of scenarios which included "With" and "Without" defence conditions. The coastal communities at risk during present day "With Defences" scenarios are the Rumney area of Cardiff, Peterstone, Liswerry and Uskmouth in Newport, parts of Whitson, Goldcliff, Summerleaze, Cadlicot, Portskewett, Sudbrook, and Chepstow. There are also many individual farms and properties at flood risk throughout the Wentlooge and Caldicot Levels.

Additional communities are at risk of coastal flooding due to the impacts of climate change. On the Caldicot Levels these include Rogiet, Magor, Undy, Redwick, Greenmoor, Bishton, Wilcrick, Llanwern and Somerton. On the Wentlooge Levels these include Duffryn, Marshfield, St Brides, Lighthouse Park, St Mellons and Trowbridge.

When the defences are removed for the "No Defences" the whole of the Wentlooge and Caldicot Levels are inundated. All the communities inundated within a "With Defence" scenarios remain affected, but to a greater depth and extent

Impact of Climate Change

Climate change simulations were undertaken for the 200-year and 1,000-year events. These simulations represent the potential increase in flood risk up to the year 2115 based on the NPPF guidance for sea-level rise estimates. These simulations show the current defence structures to be highly vulnerable to the increased risk of flooding due to climate change, becoming heavily inundated as a result of increased wave overtopping. For the majority of defences the amount of wave overtopping increases by over 100% during a climate change scenarios. In the Wentlooge model 926 properties are flooded for the present day 200-year event which increases to over 6,500 in a climate change scenario, and in the Caldicot Levels the number of properties increases from 118 to over 11,500. Table 11-2 E1 shows the number of flooded properties during climate change scenarios for both models compared against the 200 and 1,000-year event property inundation.

Table E1: Property inundation for present day and climate change simulations

Event (1 in x year)	Flooded properties (Defended)	Model
200	926	Wentlooge
1,000	1,768	Wentlooge
200CC (2115 NPPF)	6,500	Wentlooge
1,000CC (2115 NPPF)	7,930	Wentlooge
200	118	Caldicot
1,000	2,497	Caldicot
200CC (2115 NPPF)	11,701	Caldicot
1,000CC (2115 NPPF)	13,468	Caldicot

Impact on critical infrastructure

The first critical infrastructure to flood in both models is on the tidal rivers of the Usk and Rhymney. During a 5-year event the railway line south of Rumney, next to the Parc Tredelerch on the Rhymney is at risk, and on the Usk the A48 roadway between the A4042 and Liswerry in Newport. As the magnitude of the events increase the amount of critical infrastructure at risk increases. When the effects of climate change are taken into account the number of infrastructure assets at risk of flooding increases from 53 in the present day 1,000-year event to 185 for the 1 in 1,000-year event with climate change conditions.

Impact of defence breaches

The Wentlooge Level defences provide a high SoP and the flood risk is limited to tidal flooding on the Rhymney at Rumney, and areas of wave overtopping on the coastal frontage at Newton Farm, Peterstone Gout and Orchard Farm. When breaches through the defences are simulated, the flood extents are significantly increased and approximately 100 additional properties are at risk under a present day scenario. When the impacts of climate change are included over 1,000 additional properties are at risk during a 1,000-year breach scenario.

The Caldicot Levels are more susceptible to flooding during the "with Defence" scenarios and the inclusion of breaches increases the number of properties at risk by approximately 150 during present day and 400 during climate change scenarios.

Impact of outfall failure

There are very few properties at risk from the outfall failure scenarios. Outfall failure was assessed with outfalls failing to close, allowing tidal ingression and separately with outfalls failing to open and locking in fluvial flows. Most failures, whether during tidal or fluvial events, result in the flooding of one to two properties. On the Wentlooge Levels all failure open and closed scenarios result in a maximum of one property flooding.

On the Caldicot Levels, failure of the outfalls to open, trapping in fluvial flows, at Fisher's Gout and Monk's Ditch result in 8 and 22 properties flooding around Goldcliff. Failure of the outfall at Magor Pill floods nine properties and failure at Caldicot Pill, 5 properties. Failure of the outfall too close, allowing tidal ingression results in flooding of 13 properties from Fisher's Gout and Monk's Ditch but only a maximum of one property from a failure at any of the other outfalls.

Standard of Protection

The SoP provided by the coastal defences was calculated for all modelled structures. The rate of overtopping was used to determine the SoP by comparing against a range of thresholds from the Eurotop manual and against published acceptable limits of overtopping from the Severn Estuary Strategy. The target SoP for the defences in the estuary, from the Strategy study, was 1 in 1,000-years in 2010. Acceptable limits of overtopping for grassed embankments and wave return walls were compared against the modelled results. On the Wentlooge Levels only defence 6, near Peterstone Gout, has a SoP less than 1 in 1,000 for a grassed embankment. On the Caldicot Levels there are several sections with a SoP less than 1 in 1,000. For grassed embankments these are defences 21, 22, 23, 25, 27, 28, 29, 30 and 31. The majority of these defences also have a SoP less than 1 in 200-years, with defence 23 having the lowest SoP of 1 in 20-years. For defences with wave return walls, all of the overtopping discharges are lower than the acceptable discharge of $0.2m^3/s/m$.

Limitations

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

A number of assumptions were made and there are elements of subjectivity throughout all stages of the modelling process. Whilst the joint probability approaches use the most advanced statistical methods they are still limited by the amount and quality of the underlying data - that being the extrapolation of 30-years of available data out to generate 10,000-years of synthetic data. As more data becomes available, the confidence in the extrapolation of the extreme values will increase.

Overall, the work undertaken to update the Flood Map should provide users with a map that can be applied with greater confidence than previous versions. In light of the limitations highlighted above there are a number of recommendations for future work and updates which could be undertaken to improve confidence in the modelling.

Recommendations

It is recommended that the modelling is periodically revisited, particularly following large flood events. Significant new event data may alter the range of extreme values in the statistical analysis and may also provide evidence to validate the performance of the coastal models, for the wave transformation, wave overtopping and flood inundation models.

All future defence upgrades should use the updated sea-levels. If new flood alleviation schemes are built, the model should be updated to account of the new defences.

The results of this study alone should not be used for design purposes and should a flood risk assessment be required for a specific location within the modelled domain, a further investigation should be undertaken to investigate the specific risks and considerations for each site.

Conclusion

This study has used the most up to date methods and data and has improved the confidence in the mapping of flood risk in the Caldicot and Wentlooge flood cells.

New visualisations have been created in terms of animations and interactive GeoPDFs for use in incident management.

The study has highlighted that the area between Sudbrook and Chapel Farm is defended to a lower standard of protection than was identified in the Severn Estuary Flood Risk Management Strategy.

Contents

Executive Summary iii

1 Introduction 1

1.1 Project objectives 1
1.2 Study extent and previous studies 1
1.3 Inception stage 2
1.4 Report structure 2

2 Qualitative description of flood response 4

2.1 Introduction 4
2.2 Mechanisms of flooding 4
2.3 Source-pathway-receptor analysis 5
2.4 Tidal flood history 5

3 Model approach and justification 7

3.1 Overview 7
3.2 Offshore multivariate analysis of waves, winds and water levels 7
3.3 Extreme sea-levels 7
3.4 Wave transformation modelling 8
3.5 Wave overtopping 8
3.6 Inundation modelling 9
3.7 Model schematisation 9

4 Input data plan 10

4.1 Data used 10
4.2 Data availability 10
4.3 Development of a database of defence actual crest levels, locations and cross sections 12
4.4 Preparation of extreme sea-level estimates and tidal-graphs 13
4.5 Climate change assumption 16
4.6 Fluvial flow estimation for the reen catchments 17
4.7 Offshore multivariate analysis 20
4.8 Wave overtopping boundaries 21

5 Technical method and implementation 22

5.1 General methodology 22
5.2 Offshore multivariate analysis 22
5.3 Wave transformation modelling 25
5.4 Emulation 29
5.5 Wave Overtopping Modelling 31
5.6 Flood inundation modelling 37

6 Breach modelling 49

6.1 An overview of the EMBREA model 49
6.2 Breach modelling 50
6.3 Model setup 50
6.4 Breach modelling results 52
6.5 Observations and Conclusions 55

7 Flood defence reliability assessment 56

7.1 Objective 56
7.2 Overview of fragility curves 56
7.3 Uses of fragility curves 57
7.4 The RELIABLE tool 58
7.5 Input data 59
7.6 Resulting fragility curves 60
7.7 Observations and conclusions 62

Contents

8 Model proving 63

8.1 Introduction 63
8.2 Wave transformation model calibration 63
8.3 Wave overtopping model validation 64
8.4 Flood inundation model calibration 66
8.5 Sensitivity testing 66

9 Results 70

9.1 Introduction 70
9.2 Modelled scenarios 70
9.3 Wentlooge model 71
9.4 Caldicot model 79
9.5 Additional modelling scenarios 90
9.6 Flood Zone and Areas Benefiting from Defences mapping 102
9.7 Standard of Protection 103
9.8 Mapping and model outputs 105
9.9 Update overtopping spreadsheet and create new visualisation tools 105

10 Flood warning 106

10.1 Updates to the Flood Alert and Warning Areas 106
10.2 Flood Alert and Flood Warning thresholds identified 107

11 Conclusions 111

11.2 Limitations and recommendations 114

Appendices I

A Updated extreme sea-levels I

B Catchment descriptors for reen catchments XII

C Multivariate extreme value method XIV

D Background to joint probability methods XVIII

E Emulator description XX

F Design point event selection XXI

G Wave climate at structure toes XXII

List of Figures

Figure 1-1: Study area 2
Figure 2-1: Components of sea-level variation that lead to coastal flooding 4
Figure 3-1: Coastal flood modelling system architecture 7
Figure 4-1: LIDAR coverage 11
Figure 4-2: Newport extreme sea-levels versus updated 2014 extreme sea-levels, with confidence limits (CL) 14
Figure 4-3: Mumbles extreme sea-levels versus updated 2014 extreme sea-levels, with confidence limits (CL) 14
Figure 4-4: Avonmouth extreme sea-levels versus updated 2014 extreme sea-levels, with confidence limits (CL) 15
Figure 4-5: 0.5% AEP design tidal graph for Newport 16
Figure 4-6: Outfall locations 17
Figure 4-7: Flow volumes for the seven storm durations 19
Figure 5-1: Overview of the main components of the coastal boundary methodology applied for the Caldicot to Wentlooge Levels 23
Figure 5-2: SWAN model location of joint probability data sets. 24
Figure 5-3: Output from the offshore multivariate extreme value analysis for Wentlooge Caldicot 25
Figure 5-4: Wave transformation model mesh 27
Figure 5-5: Water level profile used in the model grid 28
Figure 5-6: Route Mean Square Error as a function of design point simulations – comparison between a GPE and a traditional “look up” table approach 30
Figure 5-7: Example output from the emulator for section 7. Dots represent separate events obtained from the fitted multivariate statistical model 31
Figure 5-8: Photographs showing the seawall schematised as section 7. 33
Figure 5-9: Selection of profiles considered to be representative of Section 7. 34
Figure 5-10: Schematised profile for Section 7. 34
Figure 5-11: Parameter schematisation for use in BAYONET 35
Figure 5-13: TUFLOW model domains 37
Figure 5-14: Wentlooge wave overtopping profiles 39
Figure 5-15: Caldicot wave overtopping profiles 40
Figure 5-16: Caldicot wave overtopping profiles, Caldicot to Chepstow 41
Figure 5-17: Formal flood defences and infrastructure included in the Caldicot TUFLOW model 43
Figure 5-18: Formal flood defence footprints removed from the Caldicot TUFLOW model 43
Figure 5-19: Flood defence structures and inland culverts included in the Caldicot TUFLOW model 44
Figure 5-20: Pumping stations included in the Caldicot TUFLOW model 45
Figure 5-21: Formal flood defences and infrastructure included in the Wentlooge TUFLOW model 46
Figure 5-22: Formal flood defence footprints removed from the Wentlooge TUFLOW

model.. 47
Figure 5-23: Flood defence structures and inland culverts included in the Wentlooge TUFLOW model 48
Figure 7.1: Generic form of a fragility curve 56
Figure 7.2: Source-Pathway Receptor conceptualisation of flood risk systems................ 58
Figure 7-1: Potential failure modes of the Wentlooge and Caldicot defences, showing external erosion (left) and seepage through levee (right) 59
Figure 7-1: Defence W5 fragility curve. 60
Figure 7-1: Defence W9 fragility curve. 61
Figure 7-1: Defence C19 fragility curve. 61
Figure 7-1: Defence D25 fragility curve. 62
Figure 7-1: Defence C29 fragility curve. 62
Figure 8-1: Wentlooge modelled water levels resulting from three sensitivity tests: A base case, plus and minus 20% roughness values (represented as Mannings n vales). 67
Figure 8-2: Caldicot modelled water levels resulting from three sensitivity tests: A base case, plus and minus 20% roughness values (represented as Mannings n vales). 68
Figure 8-3: Wentlooge modelled water levels resulting from two sensitivity tests (roughness patches and stubby buildings) 69
Figure 8-4: Caldicot and Wentlooge modelled water levels resulting from two sensitivity tests (roughness patches and stubby buildings)... 69
Figure 9-1: With defences flood risk for the Wentlooge Levels, 200 and 1,000-year extents with no wave overtopping 71
Figure 9-2: With defences flood risk for the Wentlooge Levels, 200 and 1,000-year extents, with wave overtopping 72
Figure 9-3: No defences flood risk for the Wentlooge Levels, 200 and 1,000-year extents 73
Figure 9-4: Wentlooge flood risk with confidence intervals 74
Figure 9-5: Wentlooge model climate change flood extents 76
Figure 9-6: Wentlooge model climate change flood extents with confidence intervals 77
Figure 9-7: No defence flood risk extent for the Wentlooge model for climate change with confidence intervals 78
Figure 9-8: With defences flood risk for the Caldicot Levels, 200 and 1,000-year extents with no wave overtopping.. 80
Figure 9-9: With defences flood risk for the Caldicot Levels, 200 and 1,000-year extents, with wave overtopping.. 81
Figure 9-10: No defences flood risk for the Caldicot Levels, 200 and 1,000-year extents. 83
Figure 9-11: Caldicot flood risk with confidence intervals 85
Figure 9-12: Caldicot model climate change flood extents... 87
Figure 9-13: Caldicot model climate change flood extents with confidence intervals 88
Figure 9-14: No defence flood risk extent for the Caldicot model for climate change with confidence intervals... 89
Figure 9-15: Breach locations for the Wentlooge TUFLOW model 91
Figure 9-16: Breach locations for the Caldicot TUFLOW model 93

Figure 9-17: Wentlooge 1,000-year breach results 94
Figure 9-18: Caldicot 200-year breach results 96
Figure 9-19: Wentlooge tidal gates failure open results 98
Figure 9-20: Caldicot tidal gates failure open results 99
Figure 9-21: Wentlooge tidal outfalls failure closed flood extents 100
Figure 9-22: Caldicot tidal gates failure closed results 101
Figure 10-1: Existing Flood Warning Areas 107
Figure A. 11-1: Mumbles 2013 tide levels with erroneous data II
Figure A. 11-2: Mumbles 2013 tide levels with erroneous data removed II
Figure A. 11-3: Illustration of the skew surge III
Figure A. 11-4: Smoothing the Skew Surge Histogram IV
Figure A. 11-5: Schematic of the Generalised Pareto Statistical Distribution IV
Figure A. 11-6: Newport 2008 and 2014 sea-level return periods VI
Figure A. 11-7: Mumbles 2008 and 2014 sea-level return periods VIII
Figure A. 11-8: Avonmouth 2008 and 2014 sea-level return periods X
Figure A. 11-9: Comparison of 2014 sea-level return periods for all sites XI
Figure C.1: Example events from the time series wave, wind and water level data XV
Figure D.1: Conceptual diagram illustrating the joint exceedence approach XVIII
Figure D.2: Approximate integration method used by the joint exceedence approach XIX
Figure D.3: Monte-Carlo simulation used to integrate the joint probability density of waves and water levels XIX
Figure 1: Location of Wentlooge to Caldicot levels and sections considered in this SWAN model grid and location of joint probability data sets. XXII
Figure 2: Offshore wave climate based on the multivariate extreme value analysis for Wentlooge Caldicot XXIII
Figure 3: Wave climate at the structure toe for section 7. XXIV
Figure 4: Wave climate at the structure toe for section 28. XXV

List of Tables

Table 4-1: Summary of original and revised extreme sea-levels at Avonmouth, Newport and Mumbles: 200-year and 1,000 year return periods 13
Table 4-2: Summary of original and revised extreme sea-levels at Avonmouth, Newport and Mumbles: 200-year and 1,000-year return period 95% confidence limits.. 15
Table 4-3: Outfall locations and catchment area adjustments 18
Table 4-4: Cumulative flow volumes and comparison with the cumulative flow volumes of the 72.5hr storm duration 19
Table 4-5: Data required for offshore multivariate analysis 20
Table 5-1: Model physics summary 28
Table 5-2: Wave overtopping profiles 32
Table 5-3: BAYONET parameter description 35
Table 5-4: Caldicot flood defence structures 44

Table 5-5: Levels of operation for Greenmoor and Collister Pill pumping stations 45
Table 5-6: Wentlooge flood defence structures 48
Table 6.1: Modelled embankments 50
Table 6.2: Embankment geometry 50
Table 6.3: Failure modes per defence and return period 51
Table 6.4: Initial breach width for each defence failing by overtopping 52
Table 6.5: Soil properties 52
Table 6.6: Results summary by defence 53
Table 6.7: Results summary by return period 54
Table 8-1: Verification 1 (27 Dec 2014 0600hrs) model inputs, outputs and percentage difference 63
Table 8-2: Verification 2 (03 January 2014 2100hrs) model inputs, outputs and percentage difference 63
Table 8-3: Verification 3 (08 Feb 2014 1900hrs) model inputs, outputs and percentage difference 64
Table 8-4: Verification 4 (15 February 2014 0200hrs) model inputs, outputs and percentage difference. In this case, the tide level is that of the next hours' time step. 64
Table 8-5: Number of overtopping discharges that exceed published EurOtop thresholds for pedestrians and trained staff 66
Table 9-1: Scenarios and events required for the study 70
Table 9-2: Wentlooge property counts 74
Table 9-3: Wentlooge climate change property counts 78
Table 9-4: Caldicot property counts 86
Table 9-5: Caldicot climate change property counts 90
Table 9-6: Number of breaches per modelled return period 90
Table 9-7: Breach parameters used in the Wentlooge model 91
Table 9-8: Breach parameters used in the Caldicot model 92
Table 9-9: Details of the breaches included in the 50m breach model simulations 93
Table 9-10: Wentlooge breach property counts 94
Table 9-11: Caldicot breach property counts 97
Table 9-12: Tidal gate failure details 97
Table 9-13: Caldicot and Wentlooge tidal gate failure property counts 102
Table 9-14: Standard of Protection Estimates for defences 1-32 103
Table 10-1: Flood Alert codes and coverage 106
Table 10-2: Flood Alert codes and coverage 107
Table 10-3: Existing Flood Warning thresholds and number of threshold crossings, (the numbers in brackets represent the threshold crossing if the levels are adjusted to account for the sea-level change along the coast) 108
Table 10-4: Met Office WWIII point 625 peak wind speeds 110
Table 11-1: Property inundation for present day and climate change simulations 112
Table 11-2: Critical infrastructure at risk 113
Table A. 11-1: Data used for sea-level updates III

Table A. 11-2: Return period results for Newport V
Table A. 11-3: Return period 95% confidence intervals for Newport V
Table A. 11-4: Return period results for Mumbles VII
Table A. 11-5: Return period 95% confidence intervals for Mumbles VII
Table A. 11-6: Return period results for Avonmouth IX
Table A. 11-7: Return period 95% confidence intervals for Avonmouth IX
Table A. 11-8: Return periods at CFB locations around the coastline XI
Table A. 11-9: Key catchment descriptors XII
Table A. 11-10: Flow volumes for the seven storm durations XIII
Table C.1: Summary of statistical treatment of the sea condition variables XVII

Abbreviations

1D .... One dimensional
2D .... Two dimensional
3D .... Three dimensional
ABD................................. Area Benefitting from Defences
ACL .... Actual Crest Level
AEP .... Annual Exceedance Probability
CC .... Climate Change
CC CI .... Climate Change Confidence Interval
CCO .... Channel Coastal Observatory
CFB .... Coastal Flood Boundary
CI .... Confidence Interval
DEM .... Digital Elevation Model
DSM .... Digital Surface Model
DTM .... Digital Terrain Model
EA .... Environment Agency
EMBREA......................... EMbankment BREAch model
EurOtop........................... European Wave Overtopping Manual
GPD .... Generalised Pareto Distribution
GPE .... Gaussian Process Emulators
GIS.................................. Geographical Information Systems
JBA .... Jeremy Benn Associates
LIDAR .... Light Detection and Radar
LSE .... Limit State Equation
m .... metres
mAOD .... metres Above Ordnance Datum
MCS .... Monte Carlo simulation
MDA .... Maximum Dissimilarity Algorithm
MHWS............................. Mean High Water Springs
NaFRA .... National Flood Risk Assessment
NFCDD .... National Fluvial and Coastal Defence Dataset
NOC .... National Oceanography Centre
NRD .... National Receptor Data
NRW .... Natural Resources Wales
OS .... Ordnance Survey
RAM .... Random Access Memory
SEFRMS.......................... Severn Estuary Flood Risk Management Strategy
SFCA .... Strategic Flood Consequence Assessment

SFRM	Strategic Flood Risk Mapping
SoN	State of the Nation
SoP	Standard of Protection
SOR	Statement of Requirements
SSJPM	Skew Surge Joint Probability Method
SWAN	Simulating Waves Nearshore model
SWE	Shallow Water Equations
TUFLOW	Two-Dimensional Unsteady Flow model
UK	United Kingdom
USA	United States of America

1 Introduction

1.1 Project objectives

This project was commissioned by Natural Resources Wales (NRW) to improve understanding of the coastal flood risk between Cardiff and Chepstow in South East Wales. The main objectives of the study were:

- To provide evidence of coastal flood risk for the communities along the Severn Estuary between Cardiff and Chepstow
- To improve understanding of flood risk with revised sea-levels and joint probability assessment
- Review guidance on climate change and breach modelling
- Improve understanding of wave related risks that will provide a better appreciation of the potential impacts of major flood events
- Improve understanding of the conditions that will warrant the issue of flood warnings and provide clarity with respect to standards of protection (SoP).

The specific objectives of the project were:

- To identify areas that would flood from the sea, without the presence of defences, during the 1 in 200-year and 1,000-year extreme sea-level events. These events are consistent with Flood Zone 3 and Flood Zone 2 of the Flood Map respectively
- To identify and map flood defences, and areas that are likely to benefit from flood defences within the tidal floodplain
- To model flood defence breaches
- To model failure of tidal outfalls for tidal and fluvial events
- To model a number of climate change scenarios to indicate how the susceptibility of the study area to flooding from the sea may change in the next century
- To model a number of present day and climate change scenarios using the 95% confidence limits of extreme sea-levels to indicate the impact on predicted flood risk.

The dominant source of coastal flood risk in the tidal floodplain is related to the coincident occurrence of a high astronomical tide, the passage of a storm surge (generated by an atmospheric depression) and the effects of wind related waves. There is also a risk from fluvial flooding in the tidal flood zone which is an additional consideration for this study, focusing on the failure of flapped outfalls on the main reens. Risk from surface water flooding was not considered in this study.

1.2 Study extent and previous studies

The study area covers the coastal region on the right bank of the Severn Estuary extending from Cardiff to Chepstow, as shown on Figure 1-1. Two major flood cells are included within the study extent. These are characterised by very large areas of flat, low lying land. The Wentlooge Levels are bounded by the River Rhymney in the west and the River Ebbw in the east while the Caldicot Levels are bounded by the River Usk in the west and the River Wye in the east.

Much of the Wentlooge and Caldicot Levels are located below the mean high water springs tide level (MHWS), so are protected from flooding by the sea by a system of sea defences. These defences essentially separate the land from the sea. The low agricultural plains are also drained and pumped by an extensive network of artificial drains and channels, including pumping stations at Collister Pill and Glenmore.

A number of previous studies which examined tidal flood risk have been completed in the study area and are referenced throughout this report where relevant. Of most direct relevance is the M4 proposed route modelling study (Arup, 2014). The combined extent of this previous studies is nearly identical to the study area for this project and the objectives were also similar. Other previous studies also are highly relevant to this study, including the SWAN modelling study (Deltares, 2011), Cardiff SFCA and Severn Estuary Flood Risk Management Strategy (SEFRMS). This new study largely represents an update to these previous studies, which will utilise improved ground level and defence data, more advanced modelling techniques and in particular will develop a new set of tidal flood risk maps associated with the revised extreme sea-level and wave data.

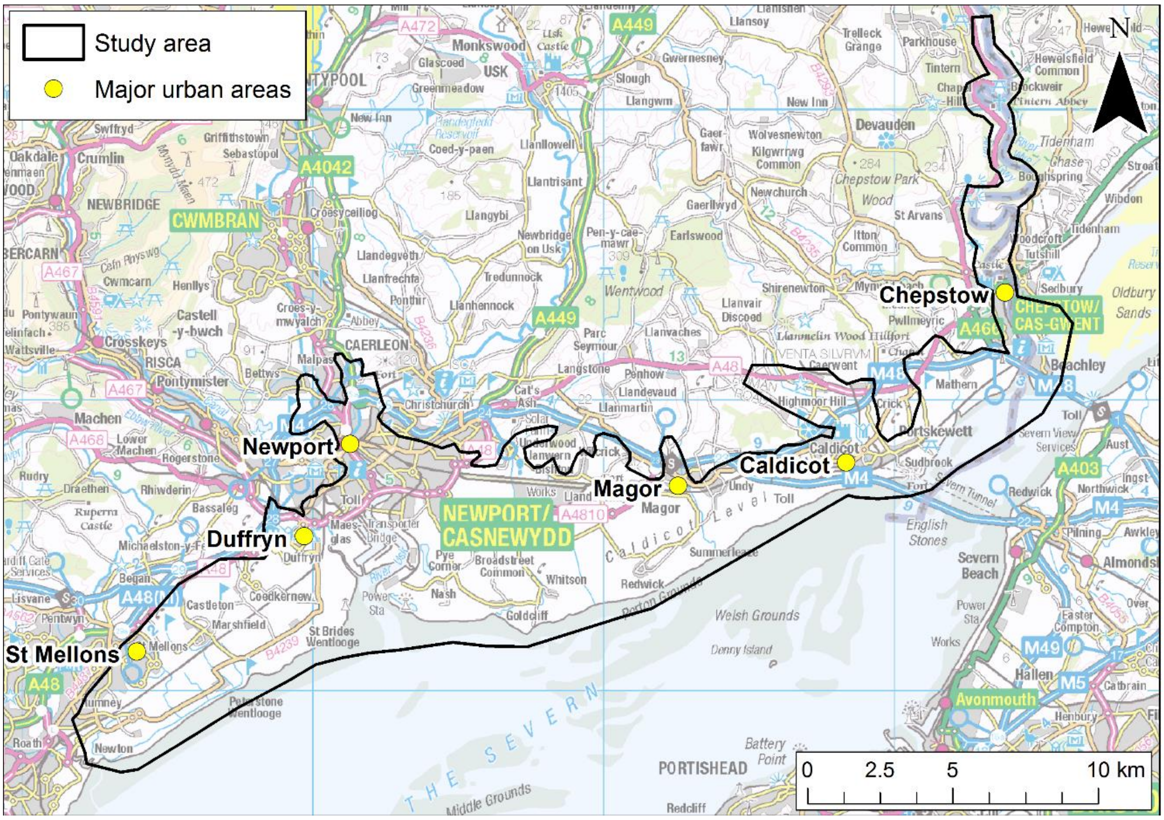


Figure 1-1: Study area

1.3 Inception stage

The project was completed in two stages, an inception stage and a main assessment stage. The specific objectives of the inception stage were:

- To collect, collate and review data on the flood defences, available flood models and previous reports
- To review the current understanding of the mechanics of flood risk in the study area and to investigate the relationship between flood risk and flood defence
- To propose a methodology for the main assessment stage.

The findings and data gathered as part of the Inception Stage were then used during the main assessment stage to model and map the coastal flood risk.

1.4 Report structure

This report summarises the key elements from the study, including a detailed methodology to meet the objectives of the study. The report consists of six chapters:

- **Chapter 1 (Introduction)** puts the study into context, defines the boundaries of the study area, outlines the objectives of the study and summaries the approach.
- **Chapter 2 (Study area, flood defences and flood history)** describes the configuration of watercourses in the study area and systems for land drainage. It also describes the flood defences that have been constructed to protect people and property from tidal flood risk and summarises the flood history of the study area.
- **Chapter 3 (Extreme sea-levels and fluvial flow estimation)** discusses the characteristics of extreme sea-levels and waves in the study area to provide an understanding of local flood risk mechanisms and to set a framework within which previous models of coastal flooding can be assessed.

- **Chapter 5 (Previous flood modelling)** describes previous flood models that have been developed in the study area, investigating their basic mechanics and suitability for use in the main assessment stage.
- **Chapter 6 (Offshore multivariate analysis)** describes the methods applied to analyse the offshore water level, wind and wave conditions and produce an event set for use throughout the different stages of modelling.
- **Chapter 7 (Wave transformation modelling)** describes the process for transforming offshore waves into the nearshore zone.
- **Chapter 8 (Wave overtopping modelling)** describes the process for calculating the rate of water overtopping the coastal defences.
- **Chapter 9 (Breach modelling)** describes the fragility and breach modelling that was undertaken to inform the breach widths and depths within the flood inundation simulations.
- **Chapter 10 (Flood inundation modelling)** describes the flood inundation modelling for the two models constructed to cover the Wentlooge and Caldicot Levels.
- **Chapter 11 (Model results)** summarises the findings of the modelling and describes the results in terms of flood extents, depths, hazards, property counts and standard of protection.
- **Chapter 12 (Conclusions and recommendations)** details the findings of the modelling stages, the limitations of the investigations and the conclusions and recommendations.

2 Qualitative description of flood response

2.1 Introduction

The first stage in the development of any flood inundation model involves consideration of three factors:

1. The local mechanisms of flooding (source);
2. Floodwater pathways and conveyance (pathway); and
3. The impacts of the flooding (receptor).

It is essential that the model developed accounts for these processes in as realistic a manner as possible; otherwise the results of the model are less reliable. In this chapter, the principal mechanisms of flooding, its flow paths and its impacts will be discussed.

2.2 Mechanisms of flooding

The study area for this project is diverse, containing saltmarshes, hard and soft defences and the tidal river inlets of the Usk, Rhymney and Wye. The Severn Estuary has one of the largest tidal ranges in the world. This, coupled with the low lying land of the Wentlooge and Caldicot Levels requires flood defences to prevent the Levels from being reclaimed by the sea. This has resulted in a flood defence network that is well developed in the study area. This defence system, assuming that it remains intact during an extreme event, effectively separates the sea from the coastal and estuary floodplains for all but the most extreme still water level storm events. When the flooding from wave overtopping is taken into account the actual standard of the defences is much lower.

Flooding from fresh water sources (i.e. from fluvial flooding and the arterial drainage network) is usually insignificant when compared to floodwaters from the coastal events. The primary goal of this study was to simulate, as realistically as possible, the mechanisms and consequences of major coastal flood events. Clearly, the key driving mechanism during a coastal event is the sea. However, the manner in which the sea floods the land is also highly dependent on the topography of the land and its defences, as well as the presence of tidal rivers, which can act as important conduits of saltwater flooding.

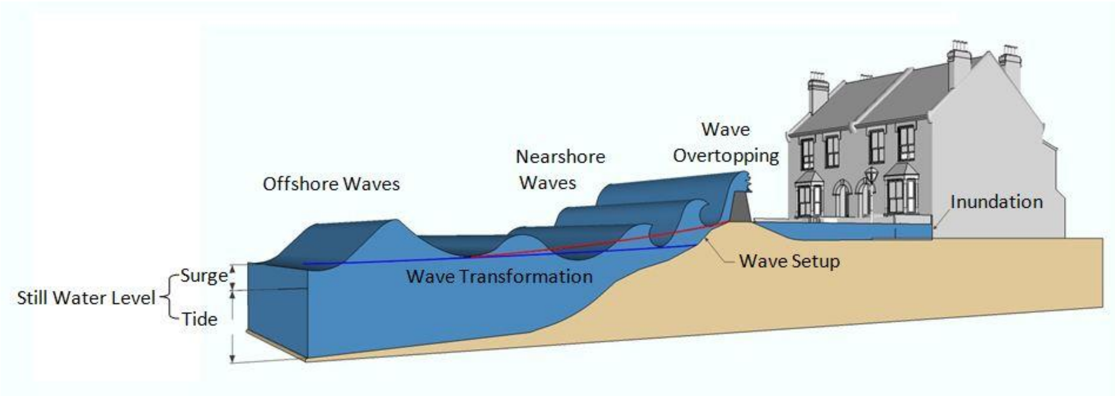


Figure 2-1: Components of sea-level variation that lead to coastal flooding

Figure 2-1 illustrates the components of sea-level variation that contribute to coastal flooding during a storm event. The base sea-level, often referred to as the 'Still Water Level', comprises of the underlying astronomical 'tide' and the passage of a large scale storm 'surge'. These two components determine the average sea-level for a particular location and time. Whilst this variable is very important in terms of coastal flooding, still water-induced flooding is limited to sheltered locations such as tidal rivers and harbours. Not surprisingly, the sea is not "still" and for the more exposed locations, the presence of waves needs to be accounted for. Given the well-developed flood defence network, the risk from still water flooding is limited to the most extreme events and in many locations most flooding occurs through 'wave overtopping', rather than still water flooding.

Wave action is a complex process controlled by a number of factors. The manner in which these factors combine determines the magnitude of any wave induced flood impact. Waves generate in deep water and then propagate towards land. As they do so, they enter shallower bathymetry where wave transformation processes occur, including shoaling, diffraction, refraction, depth limitation and breaking. The consequence of these processes is that the properties of the waves, when they reach the base of flood defences, are quite different to the waves in deep water. It is these nearshore waves that are of most importance because they interact with beaches and defences and lead to wave overtopping.

Wave overtopping is a complex process controlled by the state of the sea (depth, wave properties) and the geometry of local flood defences. There is a long history of flood defence development in the study area and a wide range of flood defence types. These defences include a variety of embankments, revetment, vertical seawalls, return-walls and rock armour.

Unfortunately, no one numerical model is capable of simulating both still water flooding and wave overtopping. Therefore, it is necessary to evaluate wave overtopping discharges separately and then to include these as inputs to a coastal flood inundation model. Similarly, it is necessary to derive the still water and fluvial hydraulic boundaries for the model separately and then to include these as inputs.

2.3 Source-pathway-receptor analysis

The source-pathway-receptor is the conceptual model of flood risk broken into the sources of flooding including the water levels, offshore wind and wave conditions, the pathway features including the flood defences and flow paths over land and the receptors, including people and properties. The sources detailed in section 2.2 are caused by a combination of still water levels, wave overtopping and wave set-up. The floodwater induced from these different flood mechanisms can potentially be conveyed through the study areas in the following ways:

- The open coastal frontages, which are liable to flooding resulting from the combination of extreme still water sea-levels, wave set-up and wave overtopping; and
- The principal tidal estuaries in the study area (including Usk, Rhymney and Wye), where the mechanism of flooding is still water flooding resulting from the inland propagation of the tide and surge along these estuary channels; these estuaries are largely sheltered from significant wave set-up or overtopping.

2.4 Tidal flood history

The Severn Estuary region has a long history of flooding, with notable events occurring in 1607, 1770 and 1809 and more recent events in 1981, 1984, 1990, 1994, 1997, 1999, 2000, 2007[1], 2012 and 2014. Historical flood map data was collated from Geostore, this contains the flood extents for previous flood events within the Wentlooge and Caldicot Levels regions. These show a significant number of properties were flooded along the River Usk and River Wye in the 1981 events. Areas particularly affected include Shafesbury (Newport) and Chepstow. Flooding in Chepstow was also reported in 1994 and 1997[2].

3-6 January 2014

The most recent storm event to affect the region occurred on 3-6 January 2014 and was the result of deep low pressure system moving in from the North Atlantic. Atmospheric pressure at the centre of the system dropped to nearly 950 hPa[3], which is relatively rare for the UK. This storm event combined with a period of high tides to produce the highest water levels recorded at Newport in the 16 years of gauged data. The sea-level of 8.03mAOD was the highest recorded since 1981 and was 0.2m higher than the 1997 level[4].

Severe flood warnings were issued and 1,050 properties were advised to evacuate in four regions in Wales including the Newport region. Despite the exceptionally high sea-levels, the number of properties flooded was relatively low. The only areas to report flooded properties were Tintern which reported less than 10 properties flooded and the Goldcliffe locality which reports less than 10 properties indirectly affected by flooding. According to the Wales Coastal Flooding Review Phase 1 Report, sea defences protected 50,000 properties and £2 billion of damages were avoided

[1] Severn Estuary Flood Risk Management Strategy (Atkins, ABP Mer, March 2011)
[2] http://www.severnestuary.net/sep/partnership/docs/ClimateChangeReportCard5.pdf
[3] http://www.metoffice.gov.uk/climate/uk/interesting/2013-decwind
[4] Wales Coastal Flooding Review Phase 1 Report - Assessment of impacts (NRW, January 2014)

across Wales. Of these properties protected, nearly 50% were located in the Wentlooge and Caldicot Levels region reflecting the importance of building and maintaining flood defences in the region. An estimated £25,000 damage was made to sea-defences within the region including an earth embankment and several sea walls.

13 December 1981

Prior to the recent 2014 event, the most significant recent flood event occurred in 1981. The event was estimated at the time to have a return period of 1 in 100-years and was due to a combination of weather conditions and high spring tides. A maximum water level of 8.40mAOD was recorded at Newport Docks, although a continuous record was not recorded. The Newport Tidal Defence Scheme Feasibility Report[5] (1982) by the Welsh Water Authority provides the single most informative document on the December 1981 event. Three main areas of flooding were identified:

1. The Marshes / Crindau Pill area on the west bank of the River Usk between the M4 motorway and railway bridges. This was the worst affected area for flood damage with over 430 houses, 30 retail trading premises and 20 commercial properties flooded to an average depth of about 0.3m.
2. Bond Street on the east bank of the River Usk, just upstream of the railway bridge; 12 houses flooded to a maximum depth of about 0.6m.
3. Bell Ferries on the east bank downstream of the transporter bridge. Although a large area was flooded the majority of the area consists of a dry dock and open storage area for sealed containers. The only appreciable damage was to the Bell Ferries offices where flooding occurred to an average depth of about 0.6m.

The following chapters describe the modelling approach to simulate the past flooding and map the flood risk for a range of extreme events.

[5] Welsh Water Authority, Usk Division. April 1982. Newport Tidal Defence Scheme Feasibility Study.

3 Model approach and justification

3.1 Overview

The Wentlooge and Caldicot Levels comprise of low lying land that is protected by flood defence networks. Overtopping or breaching of these defences will result in floodwaters spreading out across the land. To help understand this flood risk, several modelling tools were used to transform offshore waves into the nearshore, to calculate volumes of wave overtopping based on the nearshore wave heights, and to simulate flood inundation from both extreme still water level flooding and wave overtopping volumes.

The model system architecture that was used can be seen in Figure 3-1. The individual components of this system are discussed in detail below.

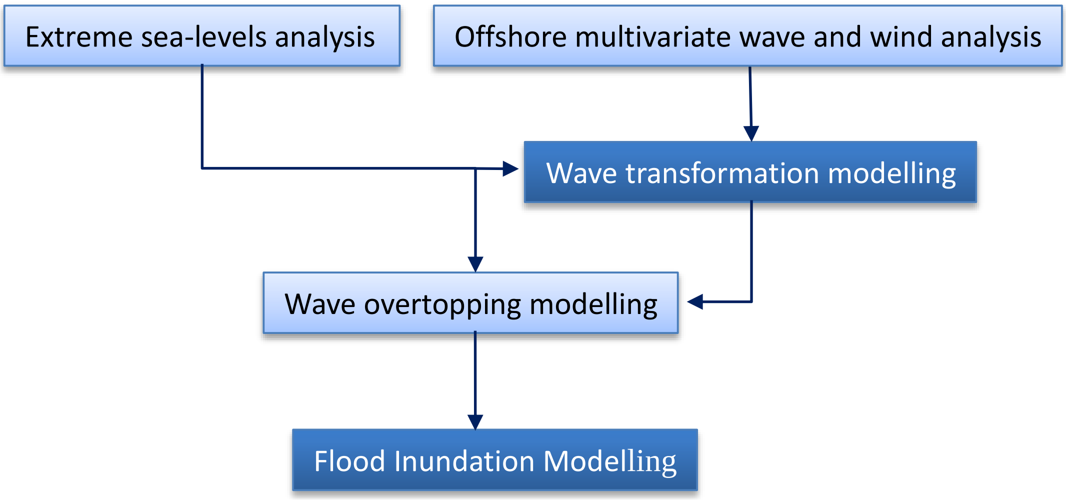


Figure 3-1: Coastal flood modelling system architecture

3.2 Offshore multivariate analysis of waves, winds and water levels

The first stage of the modelling requires analysis of waves, winds and water levels to generate storm conditions to force the wave transformation model. HR Wallingford were commissioned to apply the extreme value method developed by Heffernan and Tawn[6] as is currently being used by HR Wallingford for the Environment Agency's State of the Nation national flood risk assessment[7]. The method uses observed waves, winds and water level time series information, extracts significant wind, wave and water level events, applies statistical models to the data and extrapolates extreme conditions for rare events. The output is a Monte-Carlo simulation of waves, wind and water level information that represent a synthetic event set of 10,000-years of event data including extremes. This method is described more fully in the accompanying report by HR Wallingford[8].

The following time series data was used in the analysis:

- Measured water levels: Obtained from the “National Tidal and Sea Level Facility (NTSLF) National Class “A” Tide Gauge Network
- Modelled waves: from the Met Office Wave Watch III NAE 8km reanalysis (Euro8) dataset; significant wave height (Hs), wave period (Tm), wave direction (θ), wind speed (U), wind direction (θ_U) and direction spreading.

3.3 Extreme sea-levels

Extreme sea-levels were required to input to the wave overtopping and the two-dimensional (2D) flood inundation models to enable mapping of the flood risk areas on the land.

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

[7] Environment Agency (2016). State of the Nation Flood Risk Analysis. HR Wallingford

[8] HR Wallingford (2015) Joint probability application for Caldicot and Wentlooge

3.3.1 Review of existing extreme sea-levels

In February 2011, the Environment Agency's Evidence Directorate published the study *Improved Coastal Flood Boundary (CFB)*[9] *Conditions for the UK mainland and islands*. This project, which resulted in the derivation of new extreme sea-level estimates for the whole of the UK, was undertaken as part of the joint Environment Agency/Defra Flood and Coastal Erosion Risk Management Research and Development Programme (SC060064). The CFB data study produced sea-level estimates for a range of return periods at 2km intervals around the coast based on tide gauge data up to the end of 2008. Within the study region, the CFB data only extends upstream to the Severn crossing and therefore requires extending to the most upstream location of the study area at Chepstow.

3.3.2 Updates to extreme sea-levels

As part of this study, the impact of updating the sea-levels in the CFB from the current base year of 2008 to 2014 was investigated. The principal driver for the investigation was the availability of new data incorporating significant additional tidal event peaks.

Large coastal storm events in December 2013, January and February 2014 resulted in the highest ever recorded sea-levels at many locations around the UK, including sites where the sea-levels exceeded the 1953 event records. In addition to these recent large events, there is also an additional five years of tide gauge data available which can be used to add to the datasets used in the original CFB data study and enable the calculation of revised extremes and to reduce the uncertainty in the extreme sea-level estimates. The updates to the sea-levels are discussed in section 4.3.

3.3.3 Extension of extreme sea-levels to Chepstow

Modelling coastal flood risk within the study area requires information on offshore conditions along the full stretch of coastline between Cardiff and Chepstow. The CFB dataset, including the revised data, only extends up the Severn Estuary as far as the second Severn crossing at chainage point 382. Additional extreme sea-levels must therefore be derived further upstream at Chepstow.

To extend the Severn Estuary CFB data upstream from chainage point 382, an existing Estuary ISIS model was used. The ISIS model was extended to incorporate the River Wye as far as Chepstow. The model was extended using an existing Estry model of the River Wye to provide cross section data of the river channel. The downstream boundary of the ISIS model was updated using the revised CFB data. Tidal graphs were created for each return period from the 1-year to 10,000-year events, and these were added as boundaries to the ISIS model. The model was then used to propagate the downstream sea-levels further up the estuary, which were extracted and used within this study.

3.4 Wave transformation modelling

To transform offshore wave heights into the nearshore zone, wave transformation modelling was undertaken using the SWAN (Simulating Waves Nearshore) wave model. SWAN is a third generation wave model incorporating complex physics for the description of nearshore processes. It is an open source package (no licence required) used widely for research and commercial applications, developed by internationally recognised experts at the Delft University of Technology[10]. An existing SWAN model developed by Deltares in 2011 for NRW was used within this study. This model extends around the entire offshore region of the Welsh coastline and was further refined for the output locations along the defences of the Wentlooge and Caldicot Levels.

3.5 Wave overtopping

Wave overtopping has to be calculated separately, as no one numerical model is capable of simulating both wave propagation, overtopping and still water flooding. The techniques used to calculate wave overtopping rates are discussed in Chapter 8 and are based on European Wave Overtopping Manual (EurOtop) methods[11]. There are some uncertainties inherent in this process

---

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

[10] SWAN User Manual, SWAN Cycle III version 40.81, Delft University of Technology, 2010

[11] EurOtop Manual: Pullen, T., Allsop, N.W.H., Bruce, T., Kortenhaus, A., Schüttrumpf, H., van der Meer, J.W. (2007): EurOtop - Wave Overtopping of Sea Defences and Related Structures: Assessment Manual. http://www.overtopping-manual.com/manual.html

such as the manual schematisation of flood defences, the initial wave heights, the storm duration and the output results being estimates of the mean overtopping discharge rather than exact values.

3.6 Inundation modelling

Flood inundation modelling was undertaken using TUFLOW[12]; a fully hydrodynamic flood model well suited to modelling the progression of flood waves across a 2D floodplain. Whilst TUFLOW is a well proven model, its key limitation in terms of this study is with respect to the conveyance of flow along river channels such as the Usk and Rhymney, where the flow is largely 1 dimensional (1D). For these types of channel, 2D models such as TUFLOW, on their own, tend to underestimate conveyance resulting in uncertainty in the model outputs. However, the use of TUFLOW alone avoids unnecessary complexity and represents the real world processes to a suitable level of accuracy to achieve the objectives of this study. To help improve the accuracy, the models have been set-up with the boundaries located within the Usk and Rhymney channels and model output data from 1D models of these two watercourses have been used to provide adjustments to the water levels along the channels. TUFLOW is particularly good at representing the progression of floodwaters in the out of bank situations brought about by extreme events.

3.7 Model schematisation

The study extent covers a wide area and two separate TUFLOW models were required to map the main flood risk in the area. The models were created with a 10m grid resolution for the Caldicot model and 5m grid resolution for the Wentlooge model. At this resolution the models are able to accurately represent the general topography and bathymetry and provide detail on possible flow routes. However, the respective grids will not accurately represent some of the finer topographic details (i.e. anything smaller than 10m in the Caldicot model and 5m in the Wentlooge model), however, the important features such as defences will manually be read into the models to ensure they are represented. The models are described in more detail in Chapter 7 and the Model Development Report that accompanies this Summary Report.

[12] TUFLOW (Two-dimensional Unsteady Flow) simulates depth-averaged, two and one-dimensional free-surface flows. TUFLOW's fully two-dimensional solution algorithm solves the full two-dimensional, depth averaged, shallow water equation.

4 Input data plan

4.1 Data used

The flood models developed for this study required data from a wide range of sources. These can be categorised as ground data, flood defence data and inflow boundary data. The nature of these data and their sources is described below.

4.2 Data availability

4.2.1 Ground data

The largest overall dataset required for the study was the ground level data. These datasets were required to represent the surface upon which the progression of the flood wave moves across and interacts within the study area. The ground level data used in the study consisted of a variety of types, including terrain data, bathymetry data, and surveyed defence crest data. It was necessary to attribute the terrain data with estimates of the expected roughness of these surfaces to allow the appropriate progression of the flood wave in the model. The surface type was derived from Ordnance Survey (OS) maps.

Terrain data

A catalogue of the airborne Light Detection and Radar (LIDAR) data for the project was obtained from the Environment Agency's Geomatics Group during the Inception Stage. The LIDAR coverage is shown in Figure 4-1. Overall, this figure shows that there is near complete LIDAR coverage of the tidal floodplain. The entire study area is covered by either 1m or 2m horizontal resolution LIDAR. The 2m horizontal resolution data was used as the principal source of ground level data for high-resolution floodplain inundation modelling. The 1m horizontal resolution data was used to provide ground level data but its primary use was to analyse missing defence levels or floodplain features. Where used in the model, the higher resolution data was used in preference to the lower resolution data to make the model grid (the 1m LIDAR is layered on top of the 2m).

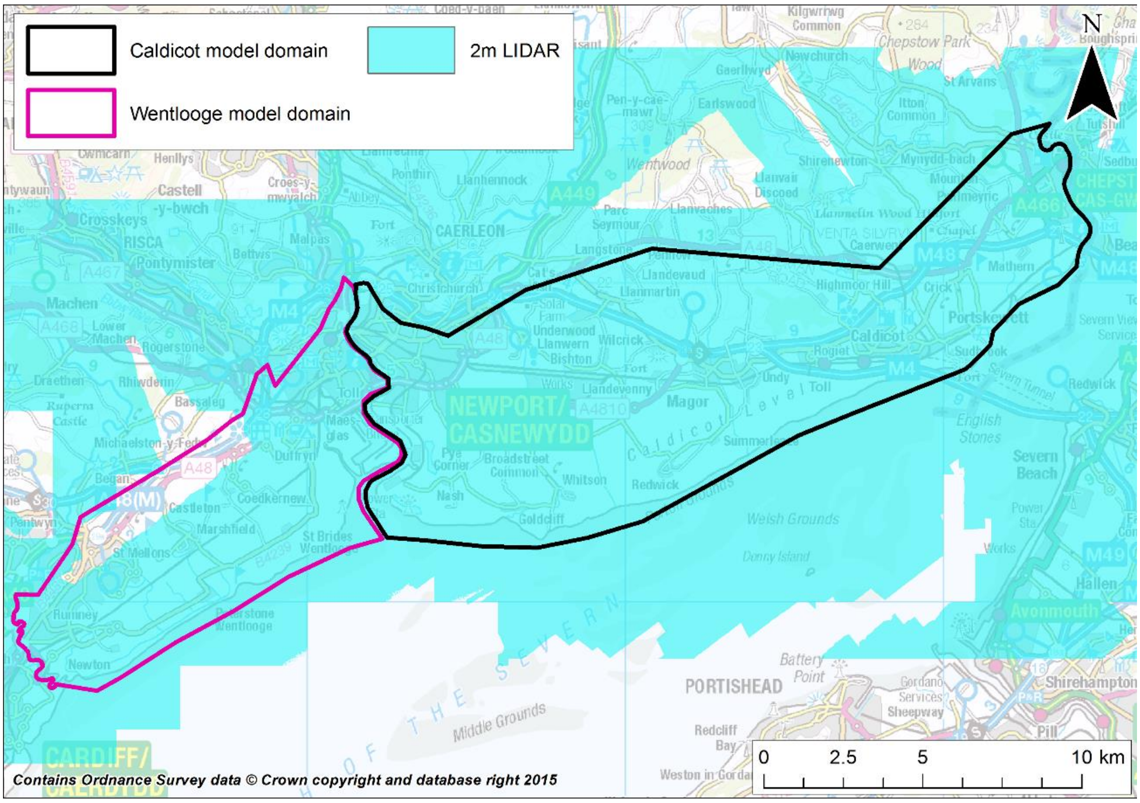


Figure 4-1: LIDAR coverage

4.2.2 Seamless 2m horizontal resolution grid of the study area

Two LIDAR products are available from Geomatics. A Digital Surface Model (DSM) which is built from unfiltered LIDAR data and a Digital Terrain Model (DTM) which is built from filtered LIDAR data. In brief, the DTM has had objects such as buildings, flood defences and vegetation removed, whilst the DSM has not been edited. The DSM and DTM of the study area were obtained during the Inception Stage. The DTM is used for the flood inundation model.

In the north of the Wentlooge model domain there is an area where no LIDAR data at any resolution was available. Areas affected include the northern extent of Newport, the hamlet of Castletown and the surrounding rural areas. At this location LIDAR information was extracted from the previous M4 model (as 'Z points') and included in the model for this study. This resolution of this data corresponds to the previous model resolution, which was 5m. No step changes between the new LIDAR and this data were apparent.

Smaller holes in the DTM were filled using interpolation techniques. Once complete, the grid was inspected for the presence of bridges and culverts. The openings that are associated with these structures on the ground were not represented in the terrain data used to construct the DTM. Instead, these structures are represented as solid features which would impede flow routes. It was therefore necessary to 'cut through' the DTM at the location of bridges and culverts to allow the flow of floodwater on the floodplain was not unrealistically constrained. These modifications were not made directly to the DTM itself, instead using 'gully lines' drawn across the structures in a Geographical Information System (GIS) that represent the inlet and outlet elevations, and interpolate a gradient along a nominal width. These gully lines were then used by TUFLOW, during a model simulation, to allow flow through the structures.

As discussed in section 2.2, the study area has large flat areas of marshland, which includes a dense arterial drainage network. The extensive network of artificial drains, known as reens, that characterise the study area are not fully represented in the model grids. In most cases the drains are only a few metres wide and therefore it is not possible to fully model them in a 5m or a 10m grid. However, ground elevations of these drains will inevitably be present in the model where model grid points fall across them. This representation will not be continuous and therefore will not form a flow path, but will contribute to the overall flood storage. The flood storage within these channels is in any case insignificant compared to the overall volumes of floodwater due to coastal flooding during an extreme sea-level event.

Many of the culverts associated with these small drains are of a similar width. The resolution of the modelling for this study was 5-10m; if gully lines were used for these structures the flow allowed through would be unrealistically large, being determined by the resolution of the model grid cell rather than the width of the channel. For these smaller structures such as culverts on drainage ditches and river channels less than 5-10m wide, 1D culvert units were used within the flood inundation model to take account of the dimensions of the structures, invert levels, estimates of internal roughness and loss coefficients.

4.2.3 Process DEMs

The seamless 2m LIDAR data were provided in ASCII grid tiles. The data therefore require processing to create seamless grids that could easily be interrogated and used for flood inundation modelling. This was completed by first converting each ASCII grid tile into an ESRI grid, and then merging all of the grids together. This was undertaken for four Digital Elevation Models (DEM): the 2m DSM, the 2m DTM, the 1m DSM and the 1m DTM. The merged DEMs were then converted back into ASCII format to be read directly into the TUFLOW models.

4.2.4 Bathymetry data

The Severn Estuary has an extremely large tidal range. As such much of the foreshore bathymetry is captured at low tide in the 2m LIDAR data. For the flood inundation model the offshore model boundary was located within the foreshore zone and so no additional bathymetry data was required. For the wave model, the model was provided with bathymetry that had been obtained from SeaZone Solutions Limited. Where the model was refined around the defence toes, the bathymetry was supplemented with LIDAR data.

4.2.5 Land use data

The rate and extent to which floodwater will flow across a floodplain is controlled partly by the roughness, which varies as a function of land cover type (for example, a woodland will offer more resistance to floodwater flow than short grassland). It was therefore necessary to attribute the terrain data used in the modelling for this study with roughness estimates. This attribution was undertaken using OS MasterMap data, which contains a highly detailed topography layer where individual features detail land objects and uses. This dataset was used to generate a polygon dataset of different land uses in the study area. Each of these different land uses was then assigned a roughness value, based on the commonly used Manning's *n* parameter. The assignment of the roughness values was based on standard values given in Chow (1959)[13] and professional judgement. As discussed in the Model Development Report, these roughness polygon datasets were read into TUFLOW during each model simulation. The DTM data over which the roughness polygons lie are assigned the relevant roughness value to best represent the manner in which floodwater will interact with land objects and varying terrain.

4.3 Development of a database of defence actual crest levels, locations and cross sections

Defence information is needed to undertake "With Defences" flood inundation modelling and to undertake wave overtopping modelling. For "With Defences" modelling a database of flood defence Actual Crest Levels (ACLs) is required whilst for wave overtopping modelling detailed survey information on flood defence cross-sections is needed. The methods that will be adopted to collate these data are described below.

4.3.1 Development of a Database of Defence Actual Crest Levels

To build high quality models that simulate flood inundation "With Defences" it was necessary to obtain accurate information on the ACLs. The available information on flood defence ACLs is reviewed in section 5.6. Since higher quality ACL information may be available from sources other than National Fluvial and Coastal Defence Dataset (NFCDD) it was recommended that a GIS dataset of flood defence ACLs was constructed that collated the best available ACLs for each flood defence. The sources of ACL information that were used are those that are described in section 5.6. Most crest level data were available from the 2013 survey dataset whilst other sources of

[13] Chow, V.T. (1959). Open-Channel Hydraulics. McGraw-Hill: Auckland.

data include design drawings for Tabbs Gout and Portland (under construction during the study) and from previous modelling studies such as the Newport model defences along the River Usk. This defence data is recent and of good quality through thorough investigation carried out for these studies.

4.3.2 Collation of cross-section survey data

Cross sections for the wave overtopping profiles were extracted from the LIDAR data. These cross sections provided the defence slope angles and elevations. The crest levels were verified against the surveyed crest level data.

Further information on the modelled defences are provided in Chapter 5.

4.4 Preparation of extreme sea-level estimates and tidal-graphs

The original CFB study calculated sea-levels to a base year of 2008. There are now several years of additional gauged data including significant storm events from the winter of 2013-2014. This additional data was used to update the CFB sea-levels. The new recorded sea-level data (2009-2014) for the Class A gauges at Newport, Mumbles and Avonmouth was collated and the same Skew Surge Joint Probability (SSJPM) method developed as part of the CFB data study was applied to calculate revised sea-level and confidence intervals. A full description of the method applied is given in Appendix A.

A comparison between the original CFB and revised extreme sea-levels for the 200-year and 1,000-year return periods at each gauge is given in Table 4-1 **Error! Reference source not found.**[14] and are plotted Figure 4-2 and **Error! Reference source not found.**. Associated confidence limits are given in Table 4-2**Error! Reference source not found.**[15].

Table 4-1: Summary of original and revised extreme sea-levels at Avonmouth, Newport and Mumbles: 200-year and 1,000 year return periods

Gauge	Return period (years)	Original 2008 CFB levels (mAOD)	Updated 2014 CFB levels (mAOD)	Difference (m)
Newport	200	8.41	8.41	**0.00**
	1,000	8.72	8.73	**0.01**
Mumbles	200	6.15	6.26	**0.11**
	1,000	6.39	6.63	**0.24**
Avonmouth	200	9.11	9.10	**-0.02**
	1,000	9.43	9.43	**0.00**

It can be seen from the results and the plots that the general trend at Newport and Mumbles is an increase in the return period sea-level estimates when the additional years of gauged data has been included. However, this increase is within the confidence limits of the return period estimates and so is not considered a significant change. Similarly, the slight reduction in the return period sea-level estimates observed at Avonmouth is not considered significant as this well within the confidence limits.

The relatively small changes in return period sea-levels suggest the method of estimating extremes is fairly robust. Though the winter events of 2013/2014 were indeed extreme sea-level events, they were also predicted within the original CFB dataset and hence the return period sea-levels have not significantly changed. Extreme sea-level events are the result of both high tides and extreme surge. Whilst the probability distribution of tides is fully known, the probability of extreme surge must be predicted using recorded surge data. The extreme tail of the skew surge probability distribution was extrapolated as part of the original CFB sea-level estimates and the additional data from these recent events was predicted within the extrapolated zone. As a result, the skew surge distribution and return period estimates are not significantly affected by the inclusion of the more recent additional data.

[14] A complete set of sea-levels for return periods ranging from 1 year to 10,000-year is given in Appendix A.
[15] A complete set of confidence limits for return periods ranging from 1 year to 10,000-year is given in Appendix A.

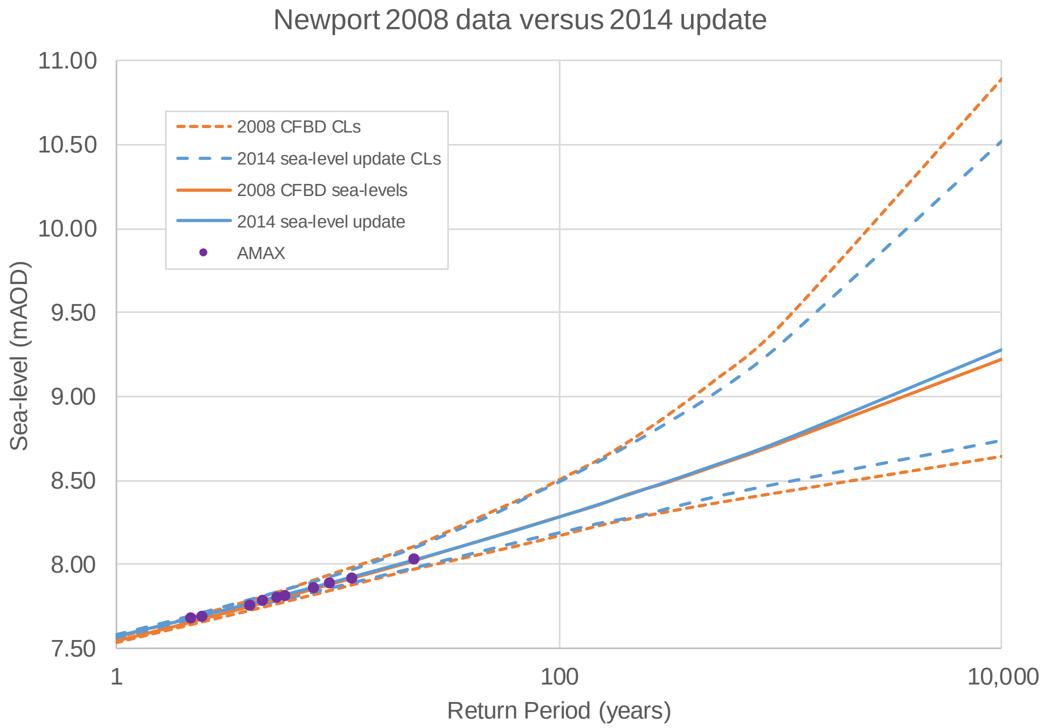


Figure 4-2: Newport extreme sea-levels versus updated 2014 extreme sea-levels, with confidence limits (CL)

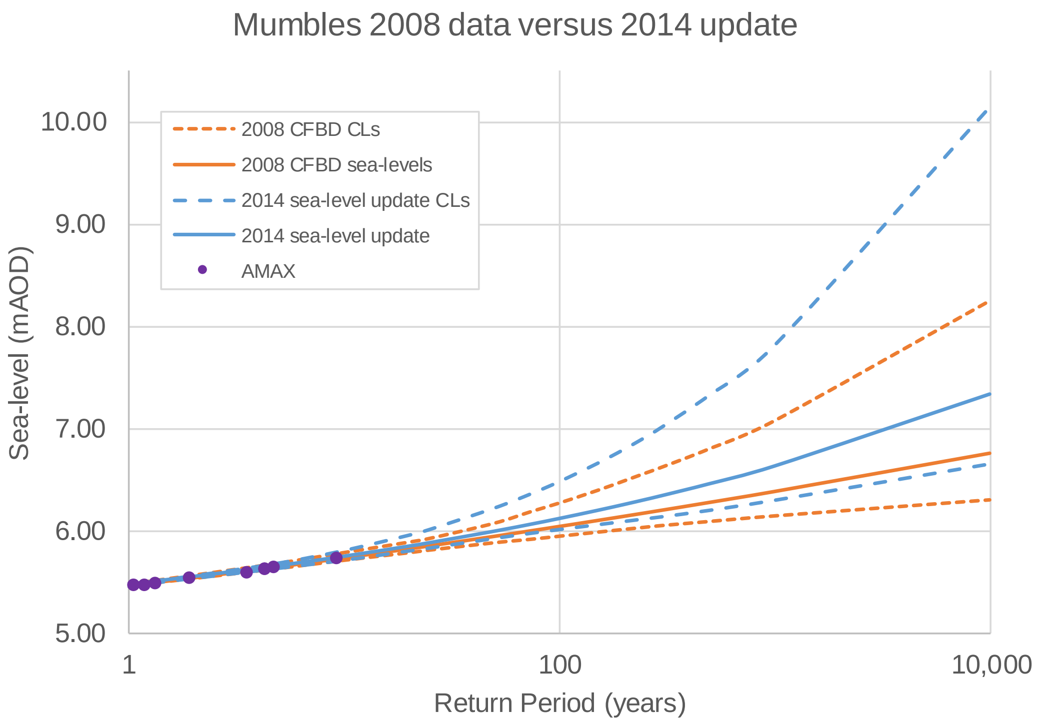


Figure 4-3: Mumbles extreme sea-levels versus updated 2014 extreme sea-levels, with confidence limits (CL)

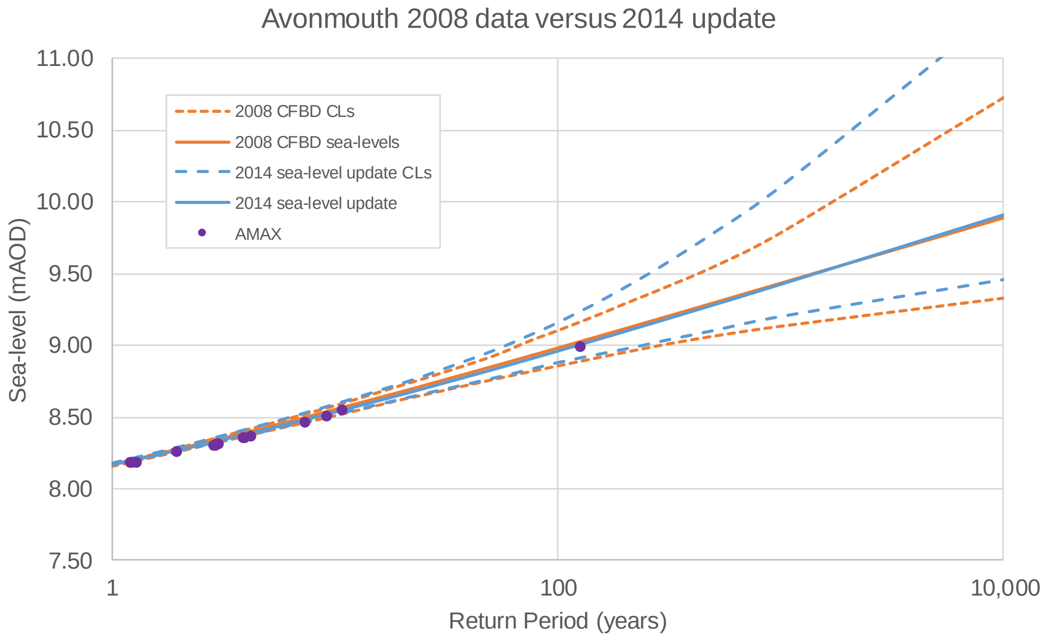


Figure 4-4: Avonmouth extreme sea-levels versus updated 2014 extreme sea-levels, with confidence limits (CL)

Table 4-2: Summary of original and revised extreme sea-levels at Avonmouth, Newport and Mumbles: 200-year and 1,000-year return period 95% confidence limits

Gauge	Return period (years)	Original 2008 CFB confidence bounds (m)	Updated 2014 CFB confidence bounds (m)	Original 2008 CFB confidence intervals (m)	Updated 2014 CFB confidence intervals (m)	Difference (m)
Avonmouth	200	0.34	0.43	0.2	0.2	0.0
	1,000	0.65	0.91	0.3	0.5	0.1
Newport	200	0.45	0.43	0.2	0.2	0.0
	1,000	0.99	0.83	0.5	0.4	-0.1
Mumbles	200	0.48	0.70	0.3	0.4	0.1
	1,000	0.94	1.51	0.5	0.8	0.3

The return period confidence intervals, shown in Table 4-2, have also not significantly changed. For the 200-year return period, there is no change at Newport and Avonmouth and only 0.1m at Mumbles. The 1,000-year confidence intervals show greater variation from the original analysis but this is expected given the much greater uncertainty with the higher return periods. With the inclusion of additional data in the extremes analysis, it might be expected that the confidence intervals would decrease. This appears to be the case at Newport, which had the shortest gauge record of the three sites and where the inclusion of new data has increased the gauge record from 15 years to 21 years. However, the addition of new data at Avonmouth and Mumbles, in particular, has slightly increased the confidence intervals. When additional extreme event data is added to a dataset, such as the 2014 event, which was the largest recorded at Mumbles, the tail of the distribution is extended. When the extension is based on a single value the limited number of events at the upper end of the tail increases the uncertainty in the results and hence the confidence intervals.

The results of this analysis were discussed. Although the difference in the updated in extreme sea-levels and original CFB data is small, it was agreed that the updated extreme sea-levels would be used in the modelling as these include the most recent and relevant data.

4.4.1 Preparation of tidal-graphs

To force the flood inundation model at the offshore boundary, design tidal graphs were required. A design tidal graph is a time-series that quantifies how sea-levels are expected to change through time during an extreme event. It is these design tidal graphs which are used to drive the still water component of the flood inundation model at its offshore boundaries. Creation of design tidal graphs requires three principal sources of information: an ESL estimate for the return period of interest; a design surge shape, and; a design astronomical tide.

The ESLs that were used in the derivation of the design tidal graphs were based on the revised CFB data that was updated with an additional 6 years of tide gauge data for this study. The CFB data was also extended to Chepstow by re-running the Severn Estuary ISIS model with updated CFB data applied at the downstream boundary.

The design surge profile that was used is based on historical surge events recorded at the closest Class A tide gauge sites, Newport and Avonmouth, as derived during the CFB Project. The underlying tide was exported from predictions based on harmonic analysis of the tide gauge data and calculated by the Admiralty Total Tide Software. Prediction sites were available at each of the extreme sea-level sites, including Cardiff, Newport, Sudbrook and Inward Rocks.

As an example, the present day design tidal graph derived for a 1 in 200-year event for Newport is shown in Figure 4-5.

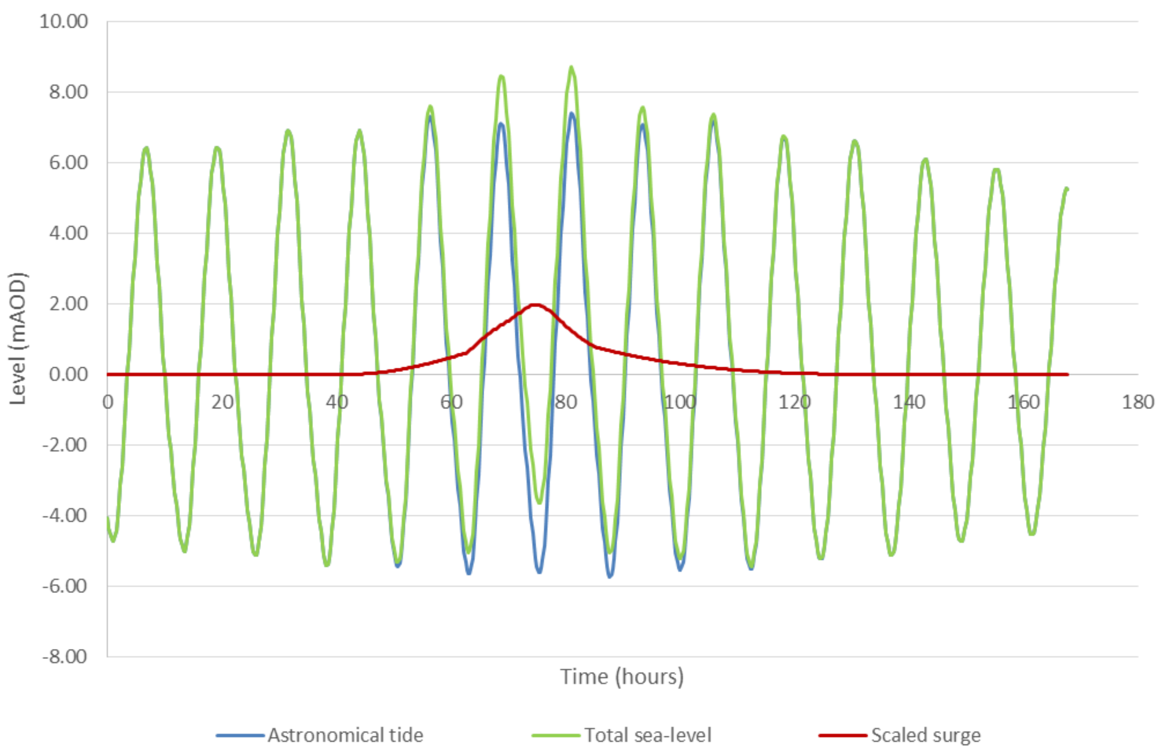


Figure 4-5: 0.5% AEP design tidal graph for Newport

4.5 Climate change assumption

Climate change scenarios were simulated to assess coastal flood risk in the event of sea-level rise and increased wave heights. For the future climate change simulations, the Monte-Carlo simulation output wave conditions for present day conditions were re-run with elevated sea-levels in line with [16] (FCDPAG3) climate change guidance. The 1 in 200-year and 1 in 1,000-year "With Defences" and "No Defences" scenarios were simulated for the climate change horizon year 2115. The sea-level increase expected to 2115 (relative to 2014) was 1.06m and this was added to sea-levels in the SWAN wave transformation model.

To account for possible changes in wind climate due to climate change, 10% was added to the offshore wind speeds modelled in SWAN. In the nearshore an increase in sea-level results in an increase in water depth, thereby allowing larger waves to arrive at the toe of flood defences.

[16] Flood and Coastal Defence Appraisal Guidance: FCDPAG3 Economic Appraisal. Supplementary Note to Operating Authorities – Climate Change Impacts; October 2006; Department for Environment, Food and Rural Affairs.

4.5.1 Climate change (2115) tidal-graphs

The tidal-graphs for climate change scenarios were calculated by simply adding the appropriate sea-level increase to the tidal-graphs that were derived for the present day conditions. Effectively this represents a constant increase in the Astronomical Tide Level and no change in the surge. This approach is consistent with the latest guidance set out in the CFB Project on tidal graph generation. A value of 1.06m was added to the present day scenario water levels in the tidal graphs to account for sea-level rise based on NPPF guidance for planners, as agreed.

4.6 Fluvial flow estimation for the reen catchments

In addition to the flood risk from coastal drivers, this study also considered flooding due to tidal gate failures. There were 15 outfalls identified with diameters greater than 1m that discharge flows from the Caldicot and Wentlooge reen drainage systems into the Severn estuary. This section details the hydrological calculations that were undertaken for the estimation of design flows in each of the reen catchments.

Point inflows were required for each of the 15 catchment areas to provide fluvial boundary conditions within the TUFLOW hydraulic model at the tidal outfall. The TUFLOW model will then simulate failures at each outfall and the results will be used to present an overview of which outfalls are likely to cause the greatest amount of inland flooding in the event of a blockage or the failure to open.

4.6.1 Outline method

Catchment descriptors for each catchment, or a nearby/similar catchment were extracted from the FEH CD-ROM (version 3). The catchment areas were manually adjusted within GIS to take into account the drainage network. Each of the extracted reen catchments were allocated an ID from 1-15 with ID1 located in the west and ID15 in the east towards Chepstow (Figure 4-6).

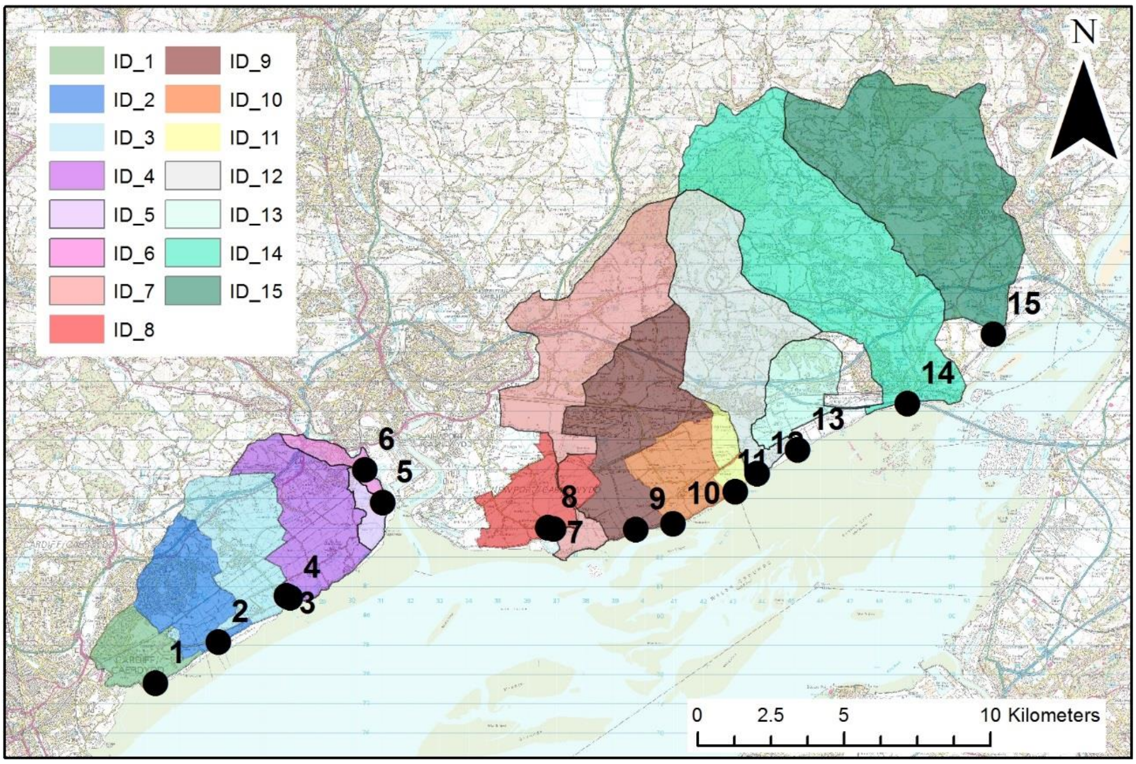


Figure 4-6: Outfall locations

Following the manual adjustments to the catchment areas the catchment descriptors were adjusted to ensure they were representative of the manually adjusted catchments. Table 4-3 details the outfall locations and the adjustments applied to the catchment areas. Appendix B details the important catchments descriptors with the adjustments made.

Table 4-3: Outfall locations and catchment area adjustments

Site code	Site	Easting	Northing	AREA on FEH CD-ROM (km2)	Revised AREA if altered
1	Rumney Great Wharf	322644	178796	0.67	5.85
2	Peterstone Great Wharf	324331	180901	1.40	10.31
3	Peterstone Gout	326187	182561	1.90	12.44
4	Peterstone Gout	328518	183399	4.70	13.70
5	New Gout	330558	183540	2.03	1.92
6	Duffryn	329497	185367	2.04	2.14
7	Goldcliff Pill	336349	184005	20.40	24.09
8	Goldcliff Pil	338140	189437	6.45	8.82
9	Great Porton	339644	186730	11.70	19.29
10	Porton Grounds	341238	184969	1.57	7.39
11	Cold Harbour Pill	342798	185610	0.52	2.39
12	Magor Pill	342828	190091	16.27	23.36
13	Collister Pill	345244	187760	2.16	7.28
14	Caldicot Pill	346063	192468	46.39	43.03
15	St Pierre Pill	349452	194738	40.02	39.62

4.6.2 Calculation of design flows

The ReFH unit within ISIS modelling software was used to calculate flows from the adjusted catchment descriptors. In ISIS, ReFH units were created for each outfall catchment. Design flows were required for the 100-year return period. The storm area of 221.64km^2 was set in all outfall units to account for the whole area of the Caldicot and Wentlooge catchments. The storm duration was set to a multiple of 6 (+0.25 as must be an uneven number of intervals when divided by timestep), with a timestep of 0.25. Seven different storm durations were set-up, ranging from 6.25 to 72.25 hours. Appendix B and Figure 4-7 details the flow volumes for each of the outfalls and the storm durations.

4.6.3 Results

The results at each of the outfall locations are presented for each of the storm durations in Figure 4-7. The longer duration storm always results in a greater flow volume at every outfall.

In the tidal gate failure scenario, the gate remains locked until it can be manually opened/unblocked by maintenance teams. Since the fluvial water has nowhere to drain to, the fluvial flood extent will continue to increase the longer the tidal gate remains blocked during the storm event. There is currently no guidance on blockage duration to limit the amount of time the flood outline is able to increase. However, it may be assumed that the maximum flood extent can be obtained by assuming the tidal gate is blocked for the entire duration of the storm hydrograph and the longest storm duration. This event will be modelled at each of the 15 outfalls for the tidal gate failure scenarios.

This assumption is conservative. In the case of the tidal gate being opened before the end of the storm event, model output grids were produced based on the maximums. For example, if the tidal flood gate was opened at 42 hours (rather than 72.5 hours) then the maximum flood outline may be derived from selecting the flood cells that flood before 42 hours in the flood simulation. After this time, the tidal gate is simulated to have been opened and the flow would discharge into the Severn Estuary.

In the case of the storm duration being shorter than the maximum modelled (72.5 hours), the cumulative flood volume may be related to the cumulative flow volumes of the longer storm duration as shown in Table 4-4. For example, the maximum cumulative flow volume of the 6.25 storm duration is reached at 37 hours in the 72.5 hour flow time series. Therefore, to derive the maximum flood extent of the 6.25 hour storm event would require all cells less than 37 hours in the "time to flood" output grids to be selected.

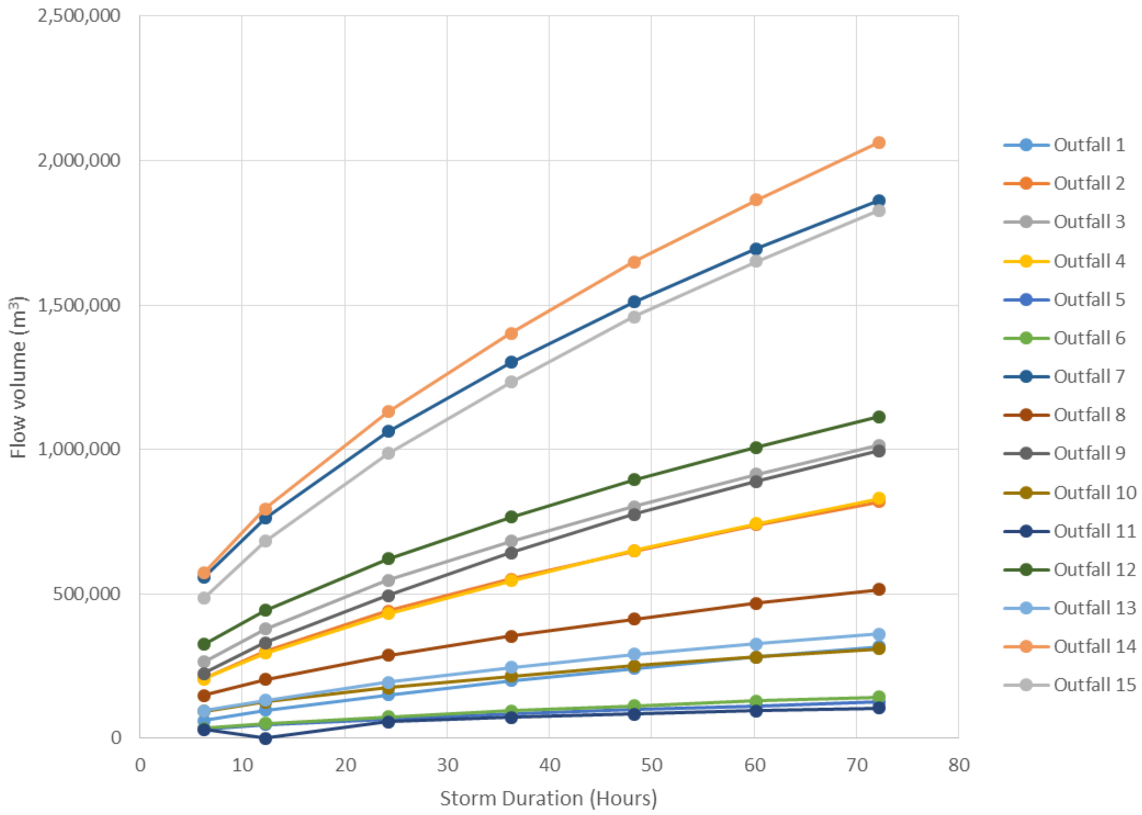


Figure 4-7: Flow volumes for the seven storm durations

Table 4-4: Cumulative flow volumes and comparison with the cumulative flow volumes of the 72.5hr storm duration

Rainfall/storm duration (Hours)	Cumulative flow volume (m^3)	Time the cumulative flow volume is reached in the 72.25hr storm duration flow series
6.25	61,996	37.00
12.25	97,268	40.25
24.25	151,038	46.00
36.25	199,799	52.00
48.25	242,523	58.25
60.25	282,501	65.50
72.25	318,413	74.25

4.7 Offshore multivariate analysis

For the multivariate joint probability, a range of water level, wave and wind data were required. The primary sources of data used within the multivariate analysis are summarised in Table 4-5.

Table 4-5: Data required for offshore multivariate analysis

Variable	Source	Comment
Sea levels time series	NTSLF National Class A tide gauge at Newport.	The Class A tide gauge at Newport was established in 1993, and consists of 15 minute records since this date.
Sea levels extremes	Updated Coastal Flood Boundaries Study extremes supplemented with extremes estimates further upstream.	Environment Agency (2011) have provided extreme sea levels around the whole of the UK at a 2km resolution. These were revised by JBA to account for the extreme sea level events over the winter 2013/14 period, and extended 30km upstream to cover the model area.
Wave conditions	WaveWatch III hindcast	Hindcast run by the Met. Office. Model grid is 8km resolution for a timespan from January 1980 to June 2014 so it includes the winter storms of 2013/2014. The model wave point used in this study is point 650 located at 51.29ºN 4.37ºW. This point was chosen to be representative of wave conditions at the offshore boundary.
Wind conditions	WaveWatch III hindcast	As for the wave conditions. However, a different model point of 51.39ºN 3.09ºW, point 625, was chosen so as to be representative of wind conditions across the model grid.

The offshore wave and wind data used in this study has been generated from a hindcast simulation undertaken by the Met Office. The hindcast uses the WaveWatch III wave model and has been run from January 1980 to June 2014. The WaveWatch III model has an 8km grid resolution, the specific grid points used in the analysis are detailed in Table 4-5. The variables analysed were:

- significant wave height (Hs)
- wave period (Tm)
- wave direction (θ)
- wind speed (U)
- wind direction (θ_U)
- directional spreading.

Prior to use in the multivariate analysis, the Met Office wave data was compared with offshore measurements at Scarweather, located just east of the Met Office prediction point at 51.43°N 3.93°W. The offshore measurements Met Office wave data showed good correlation to the offshore measurements at Scarweather, indicating there was no inherent bias within the data.

The water level data used within the analysis was obtained from the NTSLF National Class "A" Tide Gauge at Newport.

The time-series water level data were matched with the wave and wind data prior to undertaking the multivariate extreme value analysis, accounting for the approximate time lag between the model boundary and the tide gauge. Prior to implementation within the multivariate analysis, the water level data was de-trended and updated to the present day. The method used in this process is detailed in Appendix C.

4.8 Wave overtopping boundaries

Although still water levels often provide the background conditions resulting in a flood event, most of the flooding in open coastal areas occurs through the overtopping of defences due to wave action. Along with the still-water boundaries, the flood inundation models required the calculation of wave overtopping discharges to be input along wave overtopping inflow boundaries.

Along the Caldicot and Wentlooge coastline there are several communities that are exposed to swell and wind waves from the south-west and wind waves from the south-west, south and south-east, therefore, the accurate representation of the effects of wave overtopping is crucial. If defences are overtopped the impact on the flood extents may be significant.

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

The most important assumption is that wave conditions, and therefore wave set-up, remain consistent throughout the progression of the tidal curve. This approach is appropriate for modelling design events as it simulates the conditions at the boundary of the model where extreme tides, surge levels and waves occur at the same time. Changes in overtopping rates are therefore a result of the changing water level conditions rather than any changes in the incident wave conditions. Environment Agency Flood and Coastal Risk Management Modelling Guidance recommends modelling wave action over a 12-24-hour period, as the waves will then diminish as the storm moves and the wind changes direction. It was assumed that the storm continues with constant wind speeds and direction for the entire progression of the tidal curve, concurrent with the wave action. If water levels at the toe of the defence fall below the water depth physically required to support the incident wave height, then the waves are assumed to be 'depth limited' and the waves are reduced to the maximum possible height within the available water depth.

A detailed description of the delineation of the wave overtopping profiles and the wave overtopping calculations are provided in Chapter 5.

4.8.1 Wave conditions

Wave conditions at the toe of each defence were required for use in the wave overtopping calculations. These were derived through wave transformation modelling, as described in more detail in Chapter 5.

4.8.2 Flooding from other sources

The TUFLOW models constructed for this study were used to assess the tidal flood risk. Consideration was given to the flood risk associated with outfall failure on the reens but the fluvial flood risk was not considered within the study. The models also do not cover surface water or groundwater flood risk.

5 Technical method and implementation

5.1 General methodology

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

5.2 Offshore multivariate analysis

The requirement to undertake joint or multivariate probability assessments when considering coastal flood risk analysis is well-established. There are generally two approaches that are employed in practice:

- Approach 1 – Simplified joint exceedence approach;
- Approach 2 – Robust statistical approach.

These two methods have been widely used in coastal engineering practice for more than two decades. The simplified joint exceedence method was considered for use on the UK National Flood Risk Assessment (NaFRA) 2004. It was however, rejected at that time due to the known limitations of the method, Hawkes et al (2002)[17]. This method was developed for "broad-brush" applications and results in an underestimation of overtopping rates (Hawkes et al 2002). The "underestimation" arises as a result of the method of probability integration implicitly assumed in the simplified joint exceedence approach[18].

The more robust statistical approach used within previous NaFRAs and the State of the Nation (SoN) project has therefore been adopted for this project.

The objective of the method is to provide return period estimates of wave overtopping rates and associated flood volumes at each coastal defence structure along the Caldicot to Wentlooge Levels. It is based upon the methodology described by Gouldby et al (2014)[19], which is summarised in Figure 5-1 and comprised of three main components:

- A multivariate (joint) probability analysis is undertaken on offshore wave and wind data, which is then combined with sea level data and extrapolated to extreme values.
- The offshore wave conditions are translated into the nearshore zone to the base of a defence structure. As waves propagate from offshore to nearshore they undergo well-known physical process transformations that include refraction, shoaling and wave breaking, meaning the nearshore waves will have different characteristics that the offshore multivariate conditions.
- Wave, sea level and structure parameters are used to calculate the overtopping discharges using published formulae from the EurOtop Manual. These conditions are then analysed to determine extreme statistics that are used in the subsequent flood modelling.

This analysis was carried out for 32 defence sections, identified in Table 5-2. Further details of how these sections have been defined is given in section 5.4.

[17] Hawkes PJ, Gouldby BP, Tawn JA and Owen M (2002) 'The joint probability of waves and water levels in coastal defence design'. J. of Hyd. Res. Vol.40, Issue 3.

[18] Background information on the simplified joint approach and the reasons for adopting the more robust statistical approach used in this project is given in Appendix E.

[19] Gouldby B, Mendez F.J., Guanche Y, Rueda A, Minguez R (2014) "A methodology for deriving extreme nearshore sea conditions for structural design and flood risk analysis", Coastal Eng. 01/2014, 88:15–26.

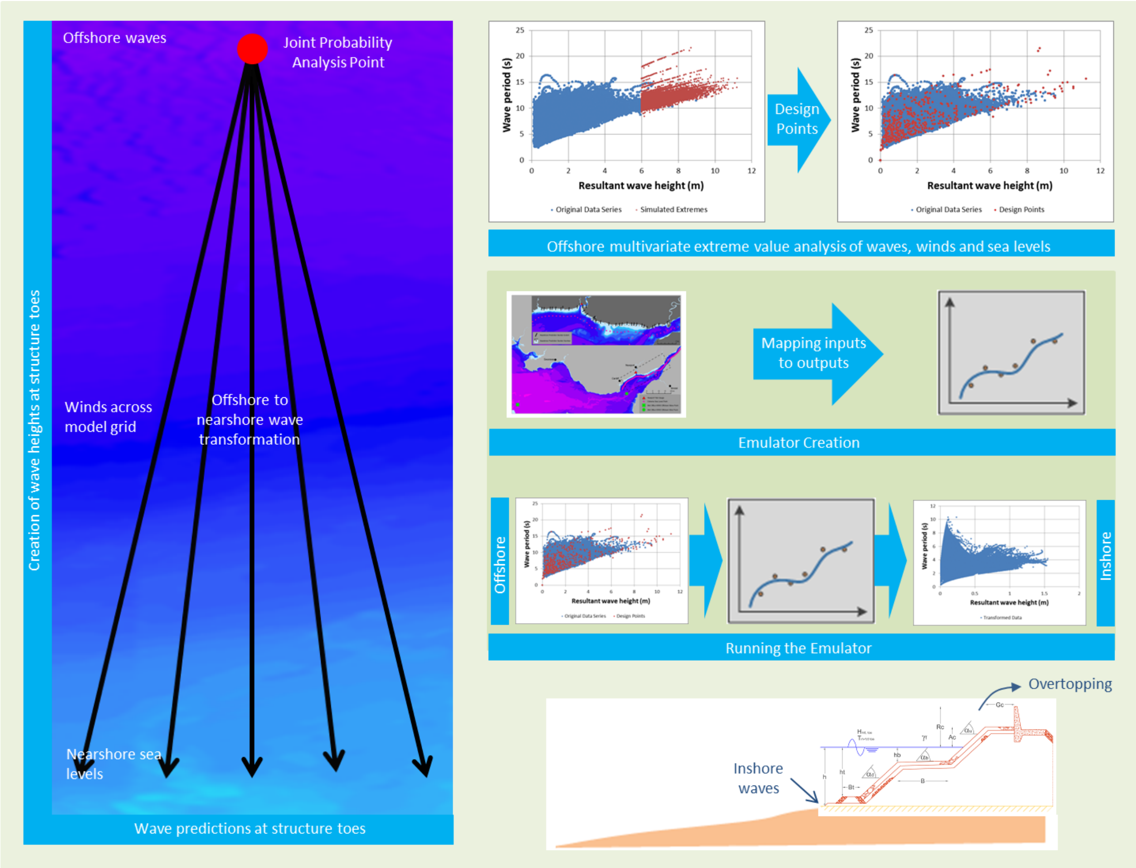


Figure 5-1: Overview of the main components of the coastal boundary methodology applied for the Caldicot to Wentlooge Levels

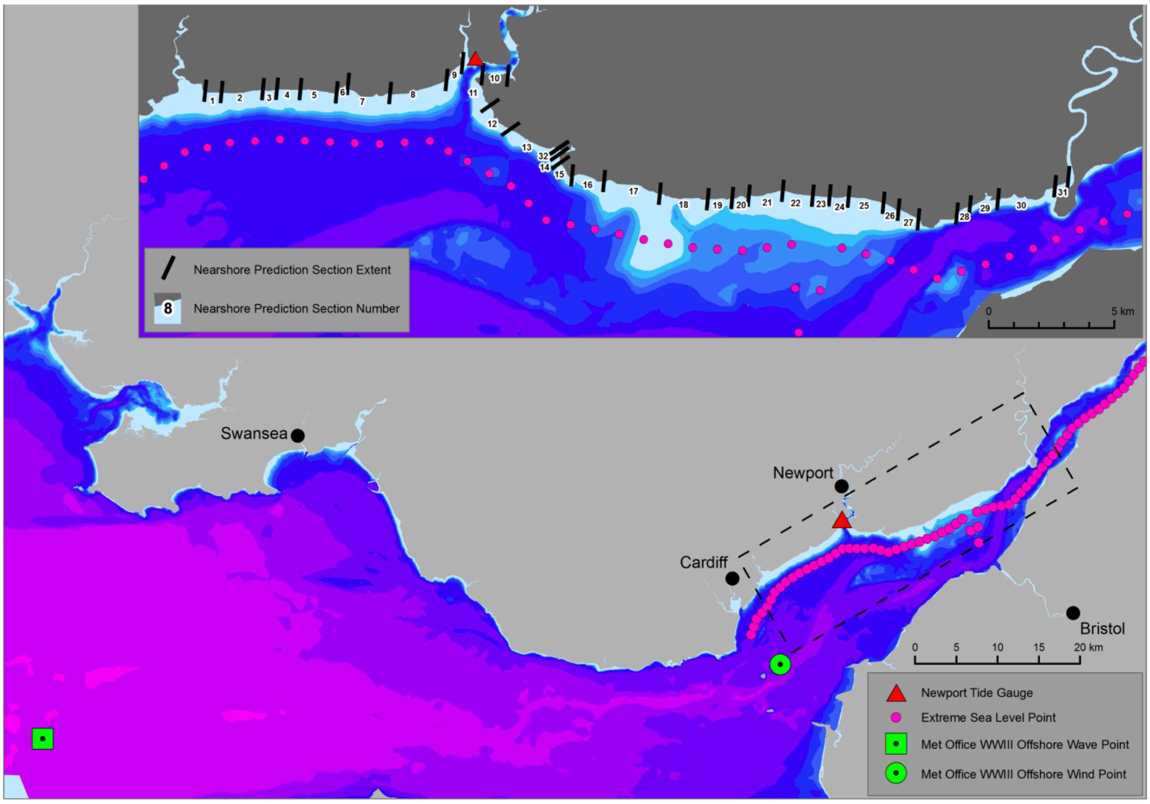


Figure 5-2: SWAN model location of joint probability data sets.

5.2.1 Multivariate extremes method

The statistical model of Heffernan and Tawn (2004) was used to undertake multivariate extreme value analysis of waves, winds and water levels. This methodology, which allows the extrapolation to extremes of any number of variables is acknowledged to give a more robust assessment of extremes than previous methodologies, such as the JOIN-SEA methodology applied in previous National Flood Risk Assessments. This section outlines the results of this model used in this study. A summary description is provided below, with further details in Appendix D.

The Heffernan and Tawn method involves fitting statistical models that enable the extrapolation of a series of variables to extreme values. The variables included within this analysis comprise:

- Significant wave height (Hs)
- Wave period (Tm-10)
- Wave direction (θ)
- Directional spreading parameter
- Wind speed (U)
- Wind direction (θu)
- Water level.

A large sample of events are then statistically simulated using a Monte-Carlo procedure. The output from this analysis is shown in Figure 5-3. The different coloured dots are as follows:

- Grey original time series
- Red peak wave height events
- Green peak wind speed events
- Blue peak water level events

The peak events are the data points that are output from the fitted multivariate extreme value statistical model, through the Monte-Carlo procedure. Grey points are underlying time-series data.

The red, green and blue points are simulated points from the fitted and extrapolated statistical model.

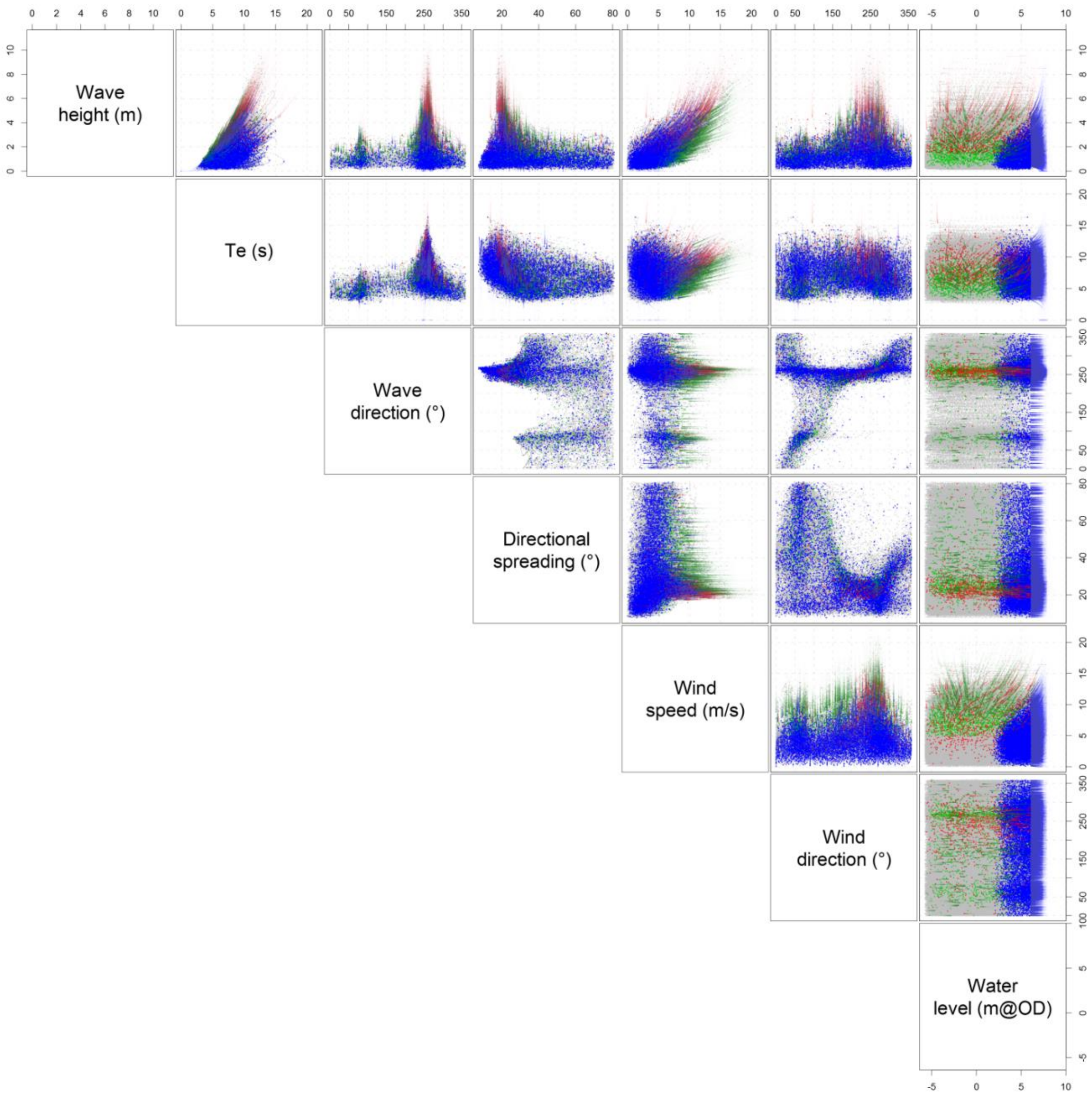


Figure 5-3: Output from the offshore multivariate extreme value analysis for Wentlooge Caldicot

5.3 Wave transformation modelling

Wave transformation modelling is required to transform the offshore waves into the nearshore to a number of defence structure toes. These are considered to be representative of sections of the Caldicot and Wentlooge coastal defences (see Figure 5-2). The model simulates the transformation of wave conditions supplied at the ocean boundary. As wave energy propagates over increasingly shallower bathymetry towards the line of coastal defences the impact of friction, refraction and shoaling become more important than in the open ocean environment. These processes alter the angle of wave approach and limit characteristics such as the height and period of the wave climate. The growth of waves due to winds is also modelled within the study area.

5.3.1 Wave modelling software

The modelling package used to consider wave transforation processes was the SWAN (Simulating WAves Nearshore) model. SWAN is a third generation wave model incorporating complex physics for the description of nearshore processes. It is an open source package used widely for research and commercial application, developed by internationally recognised experts at the Delft University of Technology[20]. The model is capable of simulating all of the following nearshore wave transformation processes:

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

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

- SWAN does not calculate wave induced currents,
- The quadruplet wave-wave interactions can give a poor approximation for long crested waves,
- The approximations for the triad wave-wave interactions have been obtained from observations in a narrow wave flume and depend on the width of the directional distribution of the wave spectrum.
- Reflection has not been directly modelled within this study.

5.3.2 Model domain and mesh development

The wave transformation model covers an area of 23,614km^2 extending around the whole coast of Wales from the Dee Estuary in the north to the Severn Estuary in the south. The original model was created to provide forecasts of wave conditions at 44 forecasts sites around the coast. The model was refined to provide high resolution at all output points around the coast but it was not sufficiently refined within the Severn Estuary to provide the required wave outputs for the defences of the Caldicot and Wentlooge Levels.

The original model was developed using a number of scripts to automatically generate the mesh based on the depth of the water, the areas of rapid depth transition and the location of the output points. Due to the size of the model, re-using the existing model and refining the mesh further in the Severn Estuary was not feasible, due to the large increase in model nodes and the resulting impact on the computation demands. To create a model with a manageable number of model nodes and manageable simulation times, the model mesh was re-created. For the Caldicot and Wentlooge modelling, the scripts were used to create a new mesh with the same model boundaries but with a higher resolution in the Severn Estuary and lower resolution for north and west Wales. The grid resolution in the refined mesh in the region around the output points was approximately 20 meters. The resolution in the coastal waters in the southern portion of the mesh is around 1,000 meters and the resolution in the offshore regions to the north and west are around 2,500 meters. Resolution was increased in regions showing either rapid depth transitions or very shallow depths. For the Isle of Anglesey, the island has not been removed from the mesh, instead the mesh includes Anglesey and picks up the positive elevations on the land. The final mesh had 123,372 nodes and is shown in Figure 5-4.

[20] SWAN User Manual, SWAN Cycle III version 40.81, Delft University of Technology, 2010

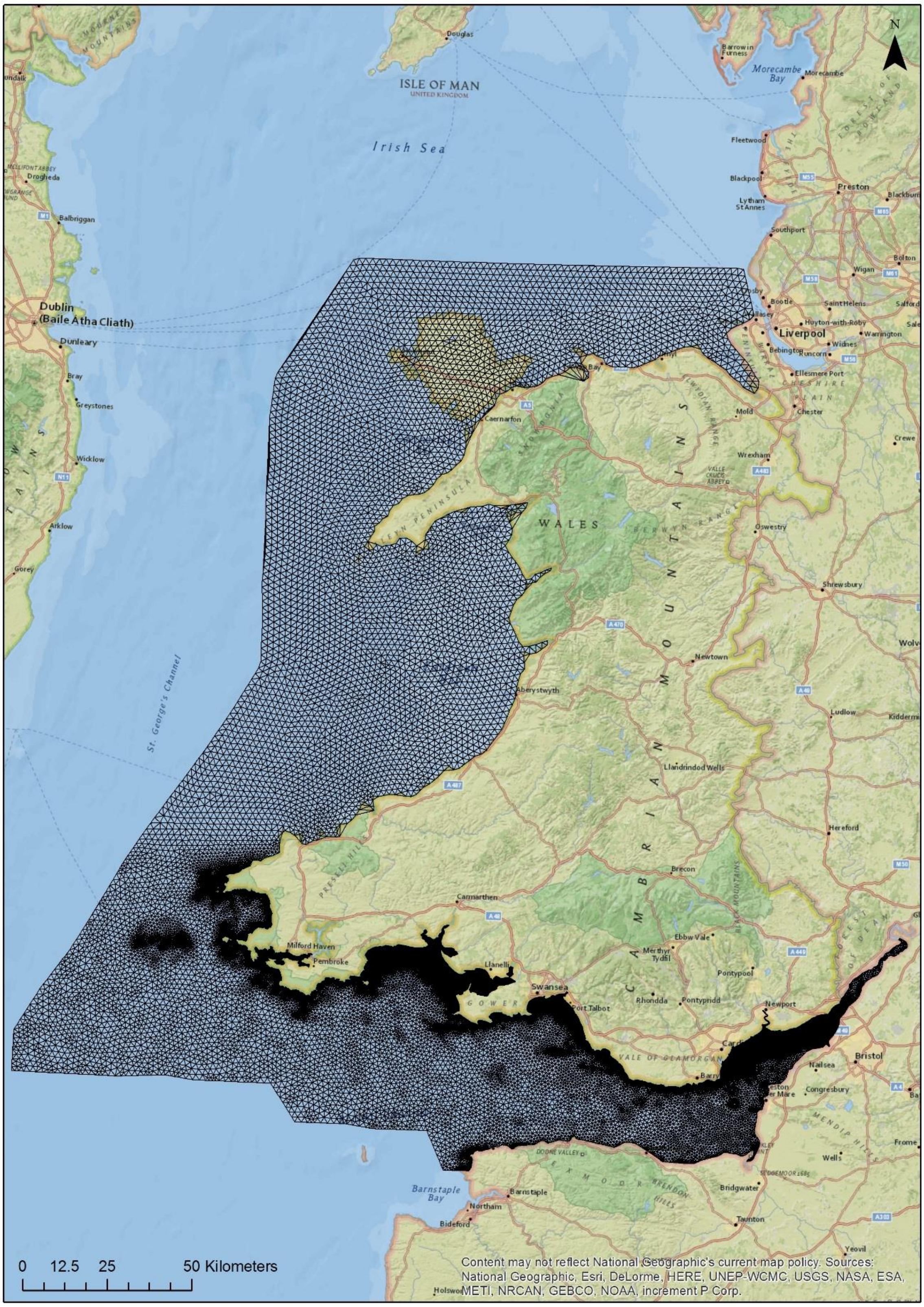


Figure 5-4: Wave transformation model mesh

5.3.3 Terrain and bathymetry data

The bathymetry from the original model was made up of bathymetry data from SeaZone Solutions Limited and ground level data from LIDAR. The bathymetry from the original model was interpolated to the new mesh. For the refined areas in the Severn Estuary, the bathymetry was improved using LIDAR data flown during low tide.

5.3.4 Model physics summary

Control files for SWAN were set up with the physics parameters left unchanged from the original model. These settings, shown in Table 5-1.

Table 5-1: Model physics summary

Physics	SWAN Syntax
Third generation model	GEN3 WESTH
Wave breaking	BRE CONSTANT 1.0 0.73
Seabed friction	FRICTION JONSWAP 0.038
Triad interactions	TRIAD

5.3.5 Sea-level boundary data

The wave transformation model must be given boundary information to represent the surface of the sea. This can be supplied either as a single level across the model grid or as a varying grid across the model domain. The spatial variation in water levels was included due to the large area of the model domain, the significant tidal range within the Severn Estuary and the potential impacts on waves propagating into the nearshore area. For all model simulations, varying water level grids were created based on the relative sea level change from the Newport tide gauge. These were created by defining an algebraic relationship between the varying water levels around the coastline and the tide gauge at Newport, based on the water levels from a 1 in 50-year return period. This relationship is given by equation 5.1, with the difference relative to Newport shown in Figure 5-5 below. This equation is applicable for all water levels, on the assumption of the same exceedance probability across the model grid, and is typically accurate to about 5cm.

The water level at a point *i* meters east of Newport (*wli*) is given by (5.1)

$$wl_i = \begin{cases} wl_N + (0.000389d^2 + 0.0318d)min\left(1, \frac{wl_N}{7.519}\right) & d > -12 \\ wl_N + (0.0268d)min\left(1, \frac{wl_N}{7.519}\right) & d \leq -12 \end{cases} \quad (5.1)$$

where:

wl_i = water level at point of interest (mAOD)
wl_N = water level at Newport (mAOD)
d = distance east from tide gauge at Newport (km)

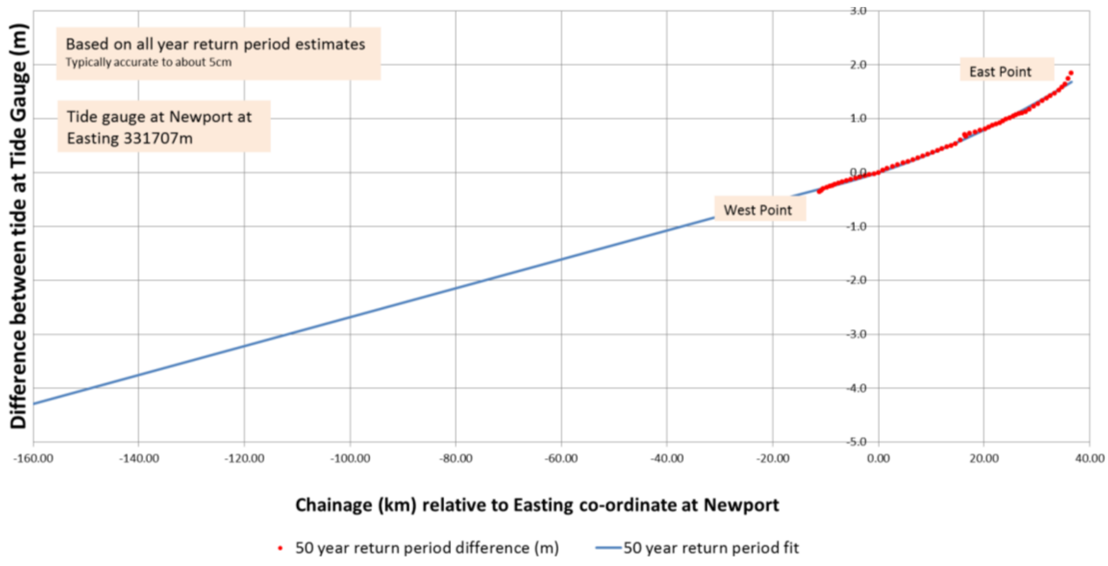


Figure 5-5: Water level profile used in the model grid

The water level profile has been used within the SWAN model to generate spatially coherent extreme water level distributions that are consistent with the existing CFB extreme water levels.

5.3.6 Wave boundary data

The offshore boundary location was unchanged from the supplied Deltares model. Boundary wave data was extracted from the Met Office WWIII hindcast model point at 51.295° N, 4.371° W. The wind data, applied as a constant wind field over the entire model domain was extracted from the Met Office WWIII hindcast model point at 51.388 ° N, 3.093 ° W, as used in the multivariate analysis.

The offshore boundary conditions were applied around the entire boundary with no variation along the perimeter. The verification wave parameters and the water levels used as an input for the wave model were obtained from the Channel Coastal Observatory (CCO) and UKMO WWIII records.

5.4 Emulation

In the wave modelling, it is required to transform all of the events output from the offshore Monte Carlo simulation (Figure 5-3) through to the structure toes for each of the defence's sections highlighted in Figure 5-2. However, SWAN is computationally time consuming to run, given the number of events that require simulation, which for Wentlooge is of the order of 200,000. The SWAN model takes 20 minutes to compute, which equates to 7.5-years of processing time if run consecutively. Therefore, rather than attempt to run SWAN for all of these events an emulator was used.

An emulator is similar in concept to a traditional "look-up table" approach used in coastal flood forecasting systems. The process involves running the SWAN model for a subset of events (known as the design points). Interpolation techniques are then applied to predict the results for other events not run in SWAN. Traditional look up table approaches are typically applied using linear interpolation techniques. As the output from SWAN is generally not a linear function of the inputs, these traditional look-up tables can be inefficient and require a large number of design point simulations. There has however, been extensive research into more sophisticated interpolation techniques, in particular Gaussian Process Emulators (GPE's), Kennedy et al (2006)[21], for example. These more sophisticated approaches have been shown to be efficient when used in the context of wave transformation modelling, Camus et al (2011a and 2011b)[22]. Figure 5-6 shows the efficiency gains that are possible, compared to a traditional "look-up table" approaches when applying a GPE to the SWAN wave model. It is evident that the same root mean squared error was achievable with 70 runs of the SWAN model using a GPE when compared to more than 48,000 runs using the traditional look-up table approach.

[21] Kennedy, M.C., Anderson, C.W., Conti, S. and O´Hagan, A., 2006. Case studies in gaussian process modelling of computer codes. Reliability Engineering & System Safety, 91(10): 1301-1309.

[22] Camus, P., Mendez, F.J. and Medina, R. (2011a). A hybrid efficient method to downscale wave climate to coastal areas. Coastal Engineering, 58(9): 851-862.

Camus, P., Mendez, F.J., Medina, R. and Cofiño, A.S. (2011b). Analysis of clustering and selection algorithms for the study of multivariate wave climate. Coastal Engineering, 58(6): 453-462.

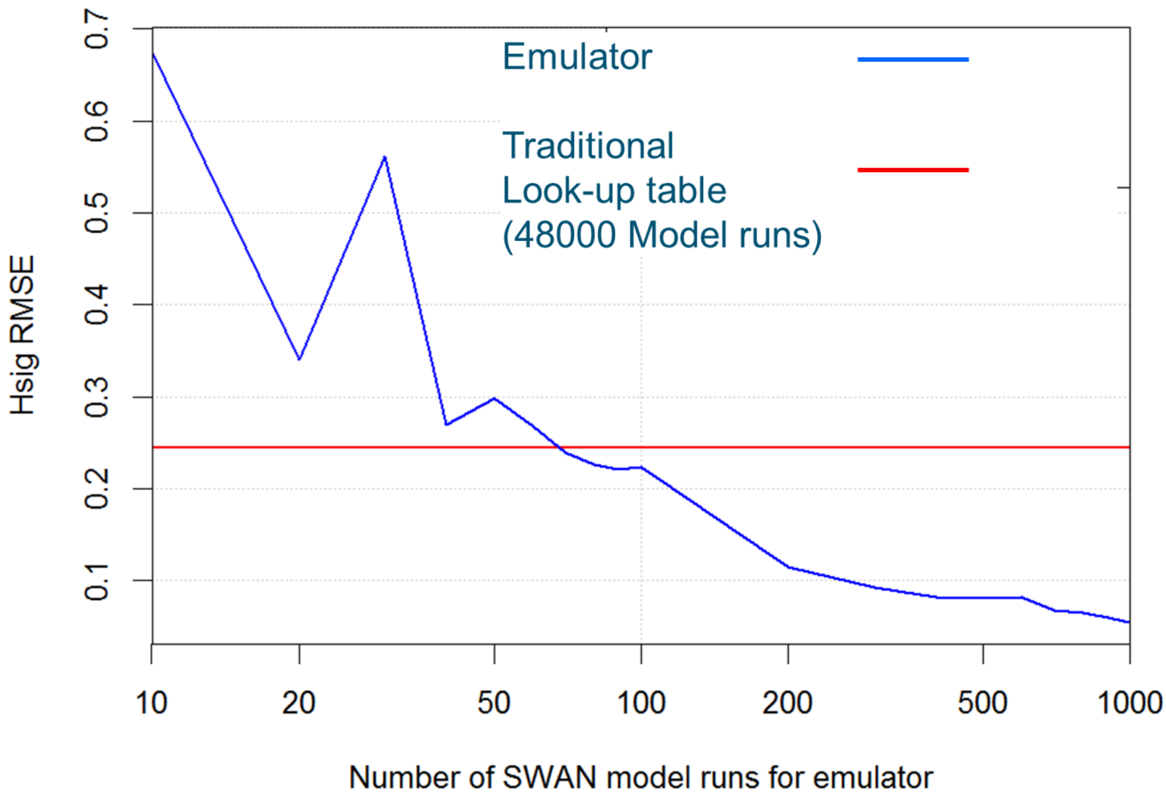


Figure 5-6: Route Mean Square Error as a function of design point simulations – comparison between a GPE and a traditional "look up" table approach

An emulator was therefore used to translate the large sample of Monte-Carlo events through to each of the defence structure toes, with a separate emulator created for each toe location. The extreme water levels outlined in Section 4.4 were then imposed for each structure toe based on linear interpolation. The emulators are trained on the sample data from the 500 SWAN model runs. Once created the emulator was used to translate all of the events within the large sample, through to the structure toe. Example data for defence 7 is shown in Figure 5-7. A more detailed description of the emulator approach adopted in this study is given in Appendix E.

To select the design points used to define the boundary conditions for the SWAN2D model, the Maximum Dissimilarity Algorithm (MDA) was used. This algorithm is described in Appendix F.

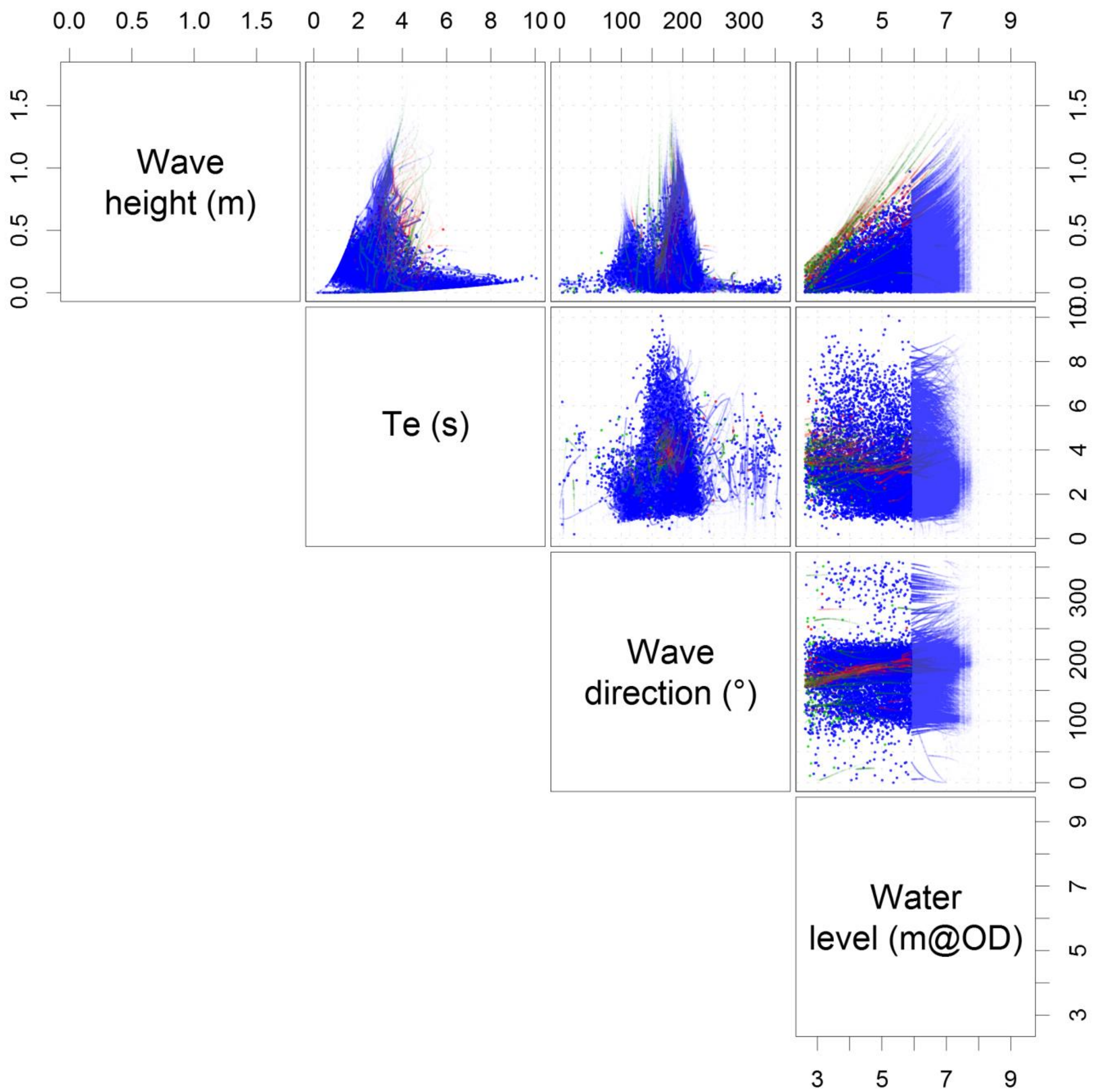


Figure 5-7: Example output from the emulator for section 7. Dots represent separate events obtained from the fitted multivariate statistical model

The red, green and blue points are based on the simulated points from the fitted and extrapolated statistical model. Grey points are the underlying time-series data (these are hidden within the coloured plots). The blurring in the water level column represents the water level threshold used for event selection.

5.5 Wave Overtopping Modelling

5.5.1 Schematisation of defences

Wave overtopping was calculated at a number of defence sections where the defence characteristics (such as type, material, geometry) were consistent. Initial overtopping assessments were then carried out at regular small spacing's for different profiles along each section to estimate the likely overtopping rate under extreme conditions. The profile which was deemed to give the approximate average overtopping rate for each section was then chosen to represent the entire section in the determination of overtopping rates and volumes.

This process took account of all information available for the Wentlooge to Caldicot frontage including:

- Survey data
- LiDAR data

- Site visits
- Photographs
- Local knowledge
- Design drawings (for sections completed after the surveys were carried out).

This process identified 32 discrete sections along the Wentlooge to Caldicot frontage which are outlined in Table 5-2.

Table 5-2: Wave overtopping profiles

Section	Toe level (mAOD)	Crest level (mAOD)	Section identification
1	2.93	9.45	Rock armoured revetment fronting a wave return wall
2	4.43	10.52	Grass embankment with berm
3	6.08	9.41	Grass embankment with berm
4	6.80	9.75	Grass embankment
5	3.81	10.06	Grass embankment
6	3.74	8.84	Grass embankment with berm
7	2.61	9.46	Rock armoured revetment fronting a wave return wall
8	7.00	9.56	Grass embankment
9	2.86	9.59	Grass embankment
10	-2.79	8.42	Grass embankment
11	6.03	11.34	Grass embankment
12	3.80	11.85	Grass embankment
13	5.90	11.10	Rock armoured revetment fronting a wave return wall
14	6.30	9.47	Blockstone fronting a wave return wall
15	3.12	9.90	Blockstone fronting a wave return wall
16	2.68	9.83	Blockstone fronting a wave return wall
17	3.30	9.72	Blockstone fronting a wave return wall
18	3.74	9.70	Blockstone fronting a wave return wall
19	4.97	9.77	Blockstone fronting a wave return wall
20	6.40	9.78	Blockstone fronting a wave return wall
21	6.71	8.94	Grass embankment
22	6.31	9.06	Grass embankment with berm
23	5.43	9.13	Grass embankment
24	6.91	8.99	Rock armoured revetment fronting a wave return wall with berm
25	6.85	9.13	Grass embankment
26	7.98	9.30	Rock armoured revetment fronting a wave return wall
27	7.47	9.48	Grass embankment
28	7.75	8.96	Grass embankment
29	7.28	8.97	Grass embankment with berm
30	7.38	9.29	Grass embankment
31	6.01	9.05	Grass embankment with berm
32	6.32	9.11	Grass embankment

An example of the process undertaken to identifying an appropriate profile to represent a particular defence section is outlined for defence section 7 below.

5.5.2 Example process of identifying sections

Section details from survey information identified the region identified as section 7 as a seawall, fronted by a sloping revetment formed of rock armour. Site visits and photographic evidence confirmed this assessment, as can be seen in Figure 5-8 below, as well as Google and Bing maps. Figure 5-8 also highlights the start and end of section 7, with the eastern end showing the start of section 8, a grassed embankment, and the western end, the end of section 6. Section 6 was a mixture of rock armour and grass embankments, however, available evidence, including sample model runs across the section and discussions with the client identified that it should be modelled as a grassed embankment.

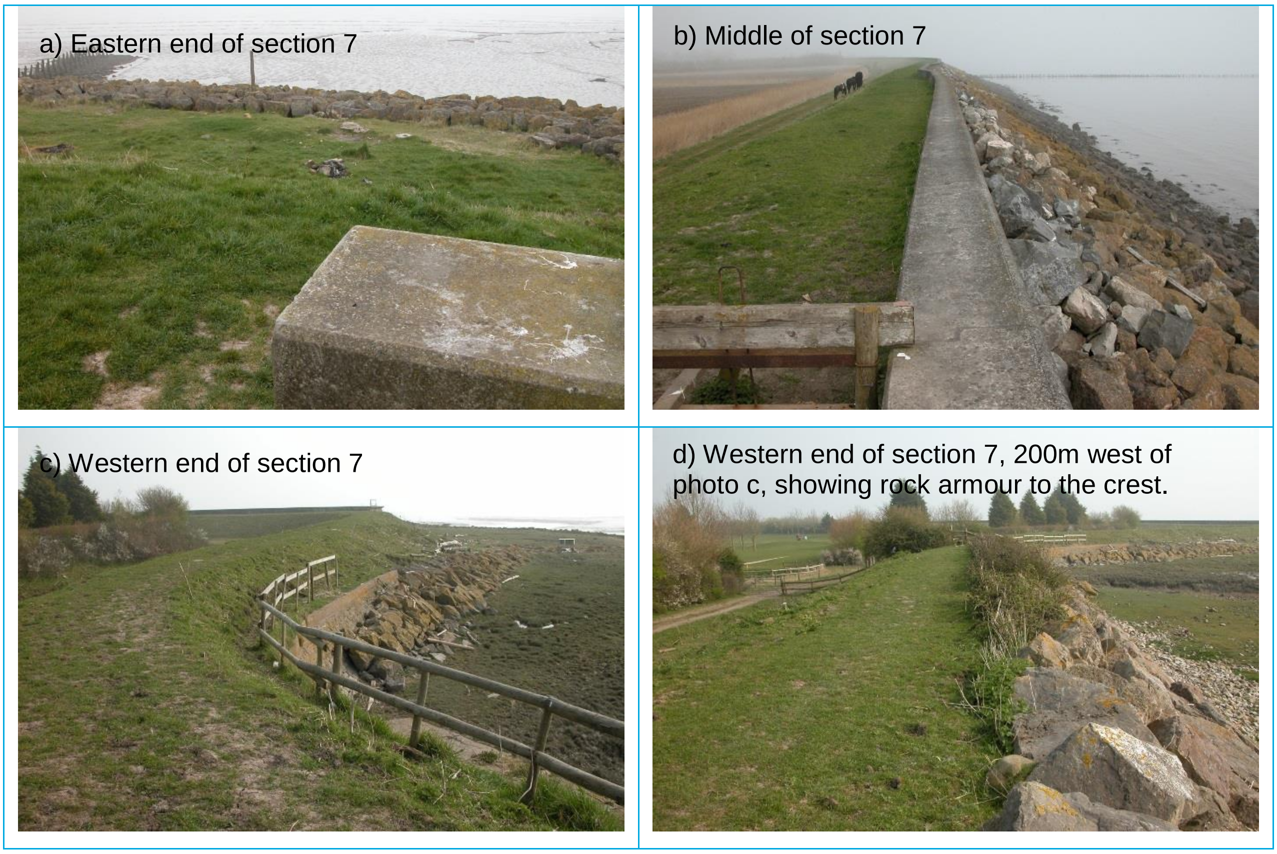


Figure 5-8: Photographs showing the seawall schematised as section 7.

Survey information combined with three-dimensional (3D) LIDAR data, as well as the other evidence identified above, was then used to provide schematised profiles at approximately 100m intervals along section 7. Figure 5-9 shows sample profiles considered for Section 7. Figure 5-10 shows the schematisation for profile 22, the one used as representative of average overtopping conditions across the whole of Section 7.

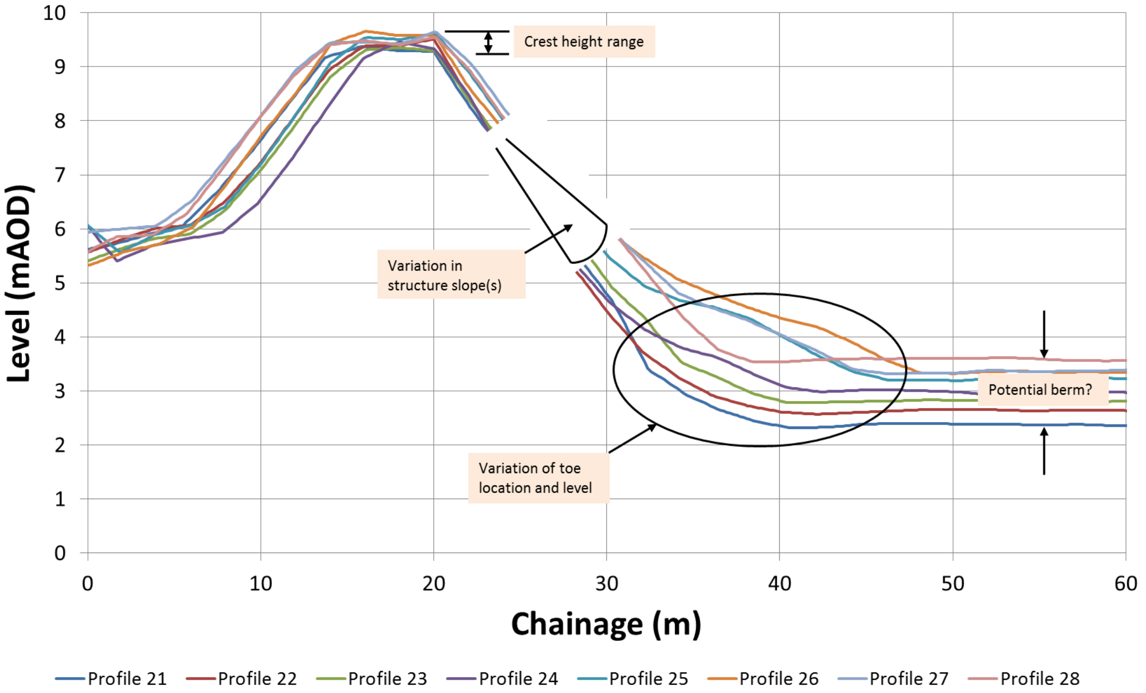


Figure 5-9: Selection of profiles considered to be representative of Section 7.

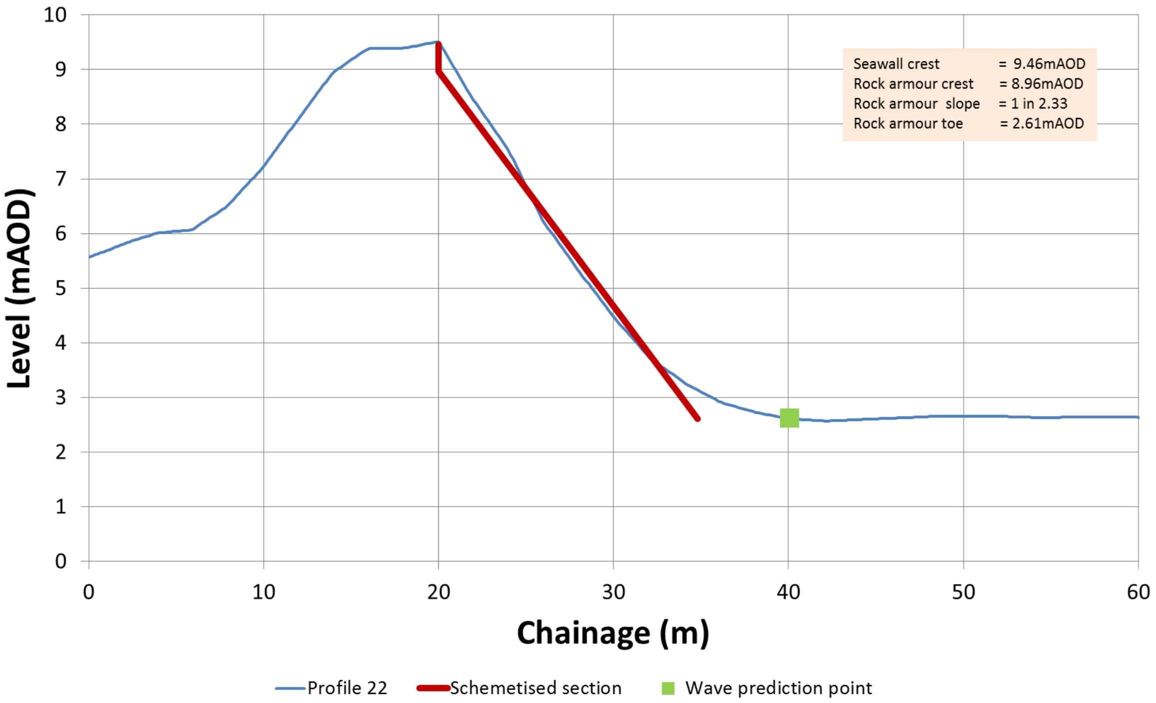


Figure 5-10: Schematised profile for Section 7.

5.5.3 Modelling wave overtopping

Wave overtopping was calculated based on the nearshore wave and defence characteristics using the BAYONET model. BAYONET (Kingston et al., 2008)[23] is a neural network overtopping tool. It is based on the widely used CLASH overtopping database and follows the general model of the

[23] Kingston G, Robinson D and Gouldby B (2008) "Reliable prediction of wave overtopping volumes using Bayesian neural networks", Proc. of FLOODrisk 2008, 30 Sep - 2 Oct, Oxford, UK In: Samuels et al. (eds). Flood Risk Management: Research and Practice. Taylor & Francis Group London.

CLASH neural network, (van Gent et al., 2007)[24] but incorporates additional information relating to uncertainty.

BAYONET has advantages over the empirical formulae given in the EurOtop overtopping manual (Pullen et al., 2007)[25] in that it is not restricted to specific structure types. BAYONET does not however, currently account for situations when the sea level is above the crest level (known as negative freeboard). In this situation, the method of Hughes and Nadal (2009)[26] was used to estimate overtopping rates. The input parameters for the BAYONET wave overtopping model are illustrated in Figure 5-11 and summarised in Table 5-3.

All of the Monte Carlo realisations at each structure toe (e.g. Figure 5-7) have been transformed through BAYONET into peak overtopping rates. The n overtopping rate samples are then ranked and each assigned a cumulative probability via the formula:

$$p_i = \frac{i-a}{n+1-2a}$$

where p_i is the cumulative probability assigned to the ith smallest overtopping rate. Empirical return periods are then assigned using

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

where n_y is the number of years of simulation (10,000). For the plotting position parameter we use $a = 0.5$ (Hazen, 1914) although alternative values only had a significant effect on the period assigned to the largest value which will not be used as we only make use of return period estimates up to 1,000 years. These results have then been analysed to determine a distribution (return periods) of overtopping rates.

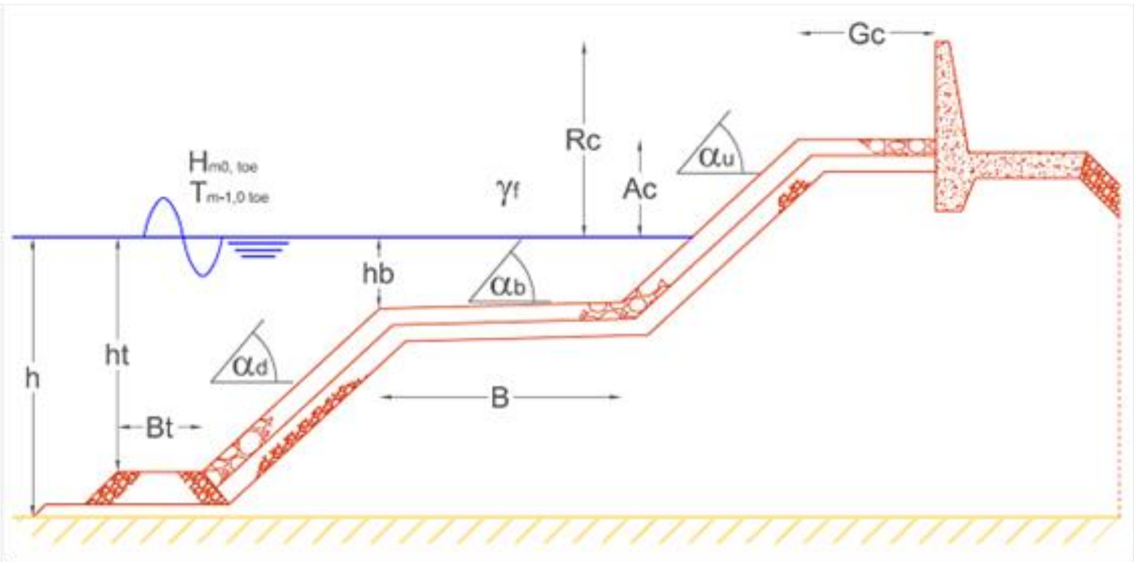


Figure 5-11: Parameter schematisation for use in BAYONET

Table 5-3: BAYONET parameter description

Parameter	Description
Hm0	Significant spectral wave height at the toe of the structure (m)
Tm−1,0	Mean spectral wave period at the toe of the structure (s)
β	Direction of wave attack w.r.t. the normal of the structure (º)
h	Water depth in front of the structure (m)

[24] Van Gent, M.R.A., Van Den Boogaard H.F.P., Pozueta B. and Medina J.R. (2007). Neural network modelling of wave overtopping at coastal structures. Coastal Engineering. 54(8): 586-593.

[25] Pullen, T., Allsop, N.W.H., Bruce, T., Kortenhaus, A., Schüttrumpf, H. and van der Meer J.W. (2007). EurOtop: Wave overtopping of sea defences and related manual, assessment manual.

[26] Hughes, S.A. and Nadal, N.C., (2009) Laboratory study of combined wave overtopping and storm surge overflow of a levee. J. of Coastal Engineering 56 (2009) 244–259.

Parameter	Description
ht	Water depth at the toe of the structure (m)
Bt	Width of the toe of the structure (m)
γf	Roughness / permeability of the structure (-)
cot αd	Slope of the structure downward of the berm (-)
cot αu	Slope of the structure upward of the berm (-)
B	Width of the berm (m)
hb	Water depth at the berm (m)
tan αb	Slope of the berm (-)
Rc	Crest freeboard of the structure (m)
Ac	Armour crest freeboard of the structure (m)
Gc	Crest width of the structure (m)
q	Overtopping discharge (m3/s/m)

An important part of the overtopping modelling was the screening-out of conditions that were outside the acceptable range for the modelling tools being used (so as not to produce spurious results). For BAYONET, a screening process was applied to identify any input conditions to BAYONET that were outside of the acceptable range, and for which an assumption of "negligible" overtopping rate was justified. These events – which are characterised by very oblique wave directions, or vanishingly small wave-heights – were not run through the BAYONET modelling chain and are, instead, assigned an overtopping rate of zero. These main rules applied for the screening are described below.

- Large oblique wave directions to the shore normal, i.e. greater than 80º
- The water depth at the toe is vanishingly small (where the waves described can be considered capillary waves, rather than gravity waves)
- The significant spectral wave height at the toe of the structure is vanishingly small

The ratio of the depth to spectral wave height is small or very large, indicating that wave height has limited effect on overtopping rates, and any overtopping is driven by potential weiring. The wave overtopping results are shown for section 7 in Figure 5-12.

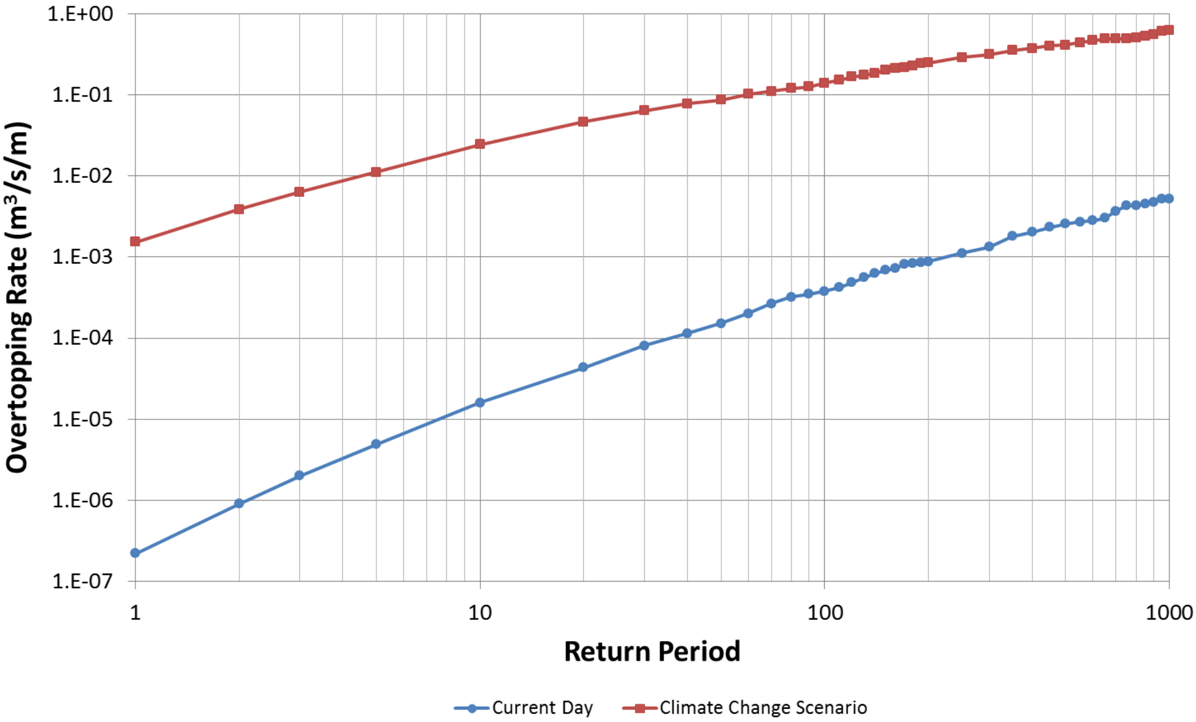


Figure 5-12: Distribution of wave overtopping rates for Section 7 showing the current day and climate change scenarios.

5.6 Flood inundation modelling

5.6.1 Configuration of flood inundation models

Flood inundation models for the Wentlooge and Caldicot levels were constructed using the TUFLOW modelling software. TUFLOW (Two-dimensional Unsteady Flow) simulates depth-averaged, two and one-dimensional free-surface flows. TUFLOW's fully two-dimensional solution algorithm solves the full two-dimensional, depth averaged, momentum and continuity equations for free-surface flow. These equations are known as the shallow water equations (SWE).

The aim of the model schematisation was to maximise the spatial resolution of the models whilst avoiding the limitations of TUFLOW. The number of grid cells that may be modelled in TUFLOW is limited by computer random access memory (RAM) and therefore, any model has to be a compromise between coverage and resolution. The entire study area covers a large area spanning from Chepstow on the border of Wales in the east to the eastern outskirts of Cardiff in the west with a combined area of approximately 200km^2. Given the large study area, two separate models were required to enable high resolution modelling of the floodplain, one model was created for the Wentlooge flood cell and the other for the Caldicot flood cell.

The Caldicot Levels is the larger of the two flood cells and the model was set-up with a 10m grid resolution. For the Wentlooge Levels, the model "With Defences" was set-up with a 5m resolution but given the large floodplain when the defences were removed, the "No Defences" model had a grid resolution of 10m.

At 10m resolution, many small features such as small embankments and channels are not fully represented. Indeed, the extensive network of artificial drainage ditches known as reens that characterise the study area are not fully represented within the model domains. The resolution chosen for each model does, however, give an overall representation of the underlying land surface and produces models that can easily be re-run within reasonable timescales. Topographic features of particular importance such as the tidal and fluvial defences and the infrastructure were added directly to the TUFLOW DEM. The two model domains are shown in Figure 5-13.

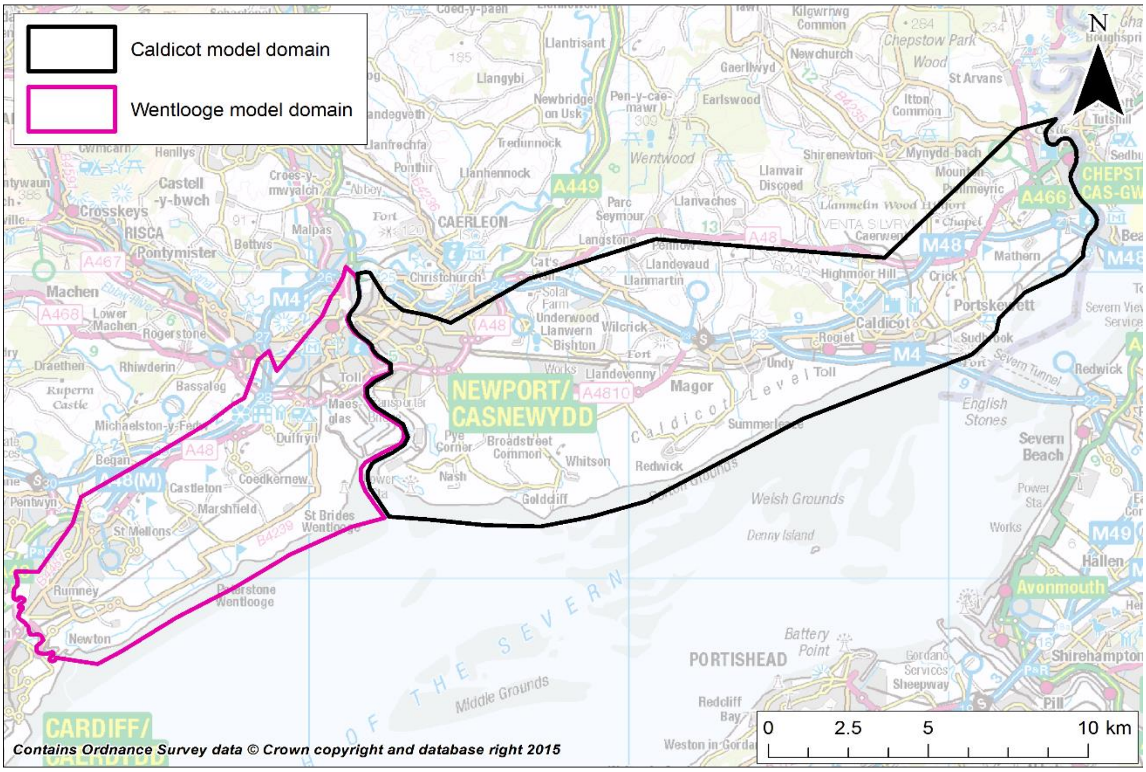


Figure 5-13: TUFLOW model domains

The area has been modelled in the past and several models were available to be re-used within this study. These models included the Cardiff Strategic Flood Consequence Assessment (SFCA) TUFLOW model, Newport Strategic Flood Risk Mapping (SFRM) ESTRY-TUFLOW model, Wentlooge M4 TUFLOW model and a Caldicot M4 TUFLOW model. Rather than re-use these models, elements of the models such as the defences, infrastructure, gully lines and culverts were extracted and updated for use in the new models.

5.6.2 Hydraulic boundaries

Along the coastal frontage the main tidal boundary used the updated CFB extreme sea-levels as described in Section 4.4.

As shown in Figure 5-13, the boundaries of both the Caldicot and Wentlooge models extend along rivers inland. These include the River Rhymney, River Usk and River Wye. The expected variation in water level along these rivers, as compared with sea-levels modelled at the open coast, was derived from previous modelling results, as detailed below:

- **Cardiff SFCA (Atkins, 2011)** A (1D) HEC-RAS model was used to derive an appropriate correction factor to represent the increase in water level expected further upstream along the River Rhymney boundary.
- **Newport SFRM modelling (JBA, 2011)** The 100, 200 and 1000-year water levels were extracted and used to represent the western boundary in the Caldicot model and eastern boundary in the Wentlooge model.
- **River Wye.** The Severn Estuary ISIS model was updated to include the lower reach of the River Wye to a point upstream of Chepstow. Water levels were extracted for the modelled tidal scenarios and used to represent the variation along the River Wye in the Caldicot model.

Along the coastal frontage, additional model boundaries were added to allow the input of wave overtopping discharges. As described in section 5.5, the defences were split into 32 sections, consistent with the Severn Estuary Flood Risk Management Strategy. Wave overtopping was calculated for each section.

Figure 5-14: Wentlooge wave overtopping profiles

Figure 5-15: Caldicot wave overtopping profiles

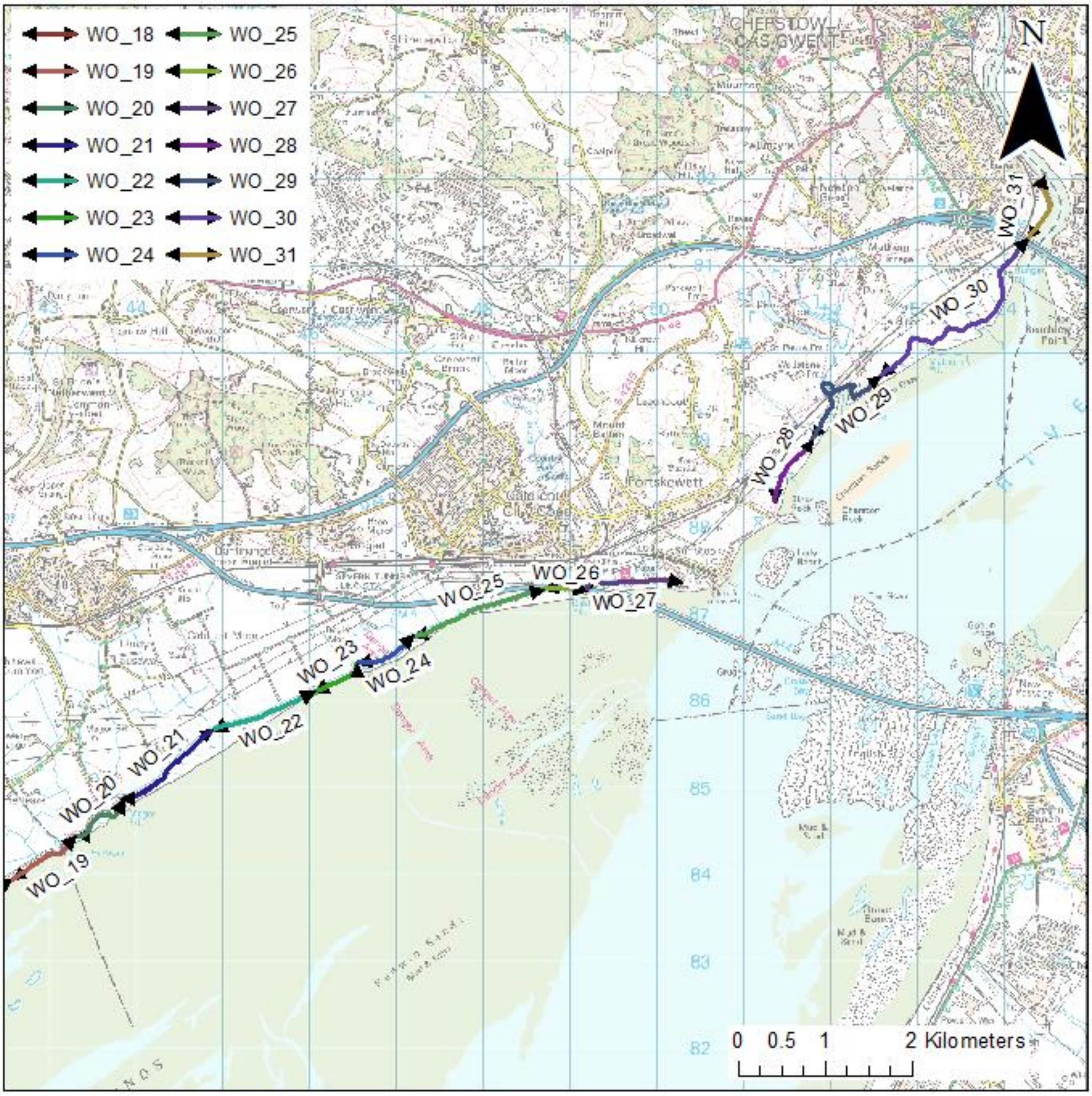


Figure 5-16: Caldicot wave overtopping profiles, Caldicot to Chepstow

5.6.3 Representation of buildings

Within high resolution TUFLOW models it is possible to use Ordnance Survey MasterMap data to represent the presence of buildings in the urban tidal floodplain. The representation of buildings within the 2D hydraulic models has important implications for determining the flood outlines and local flood depths, velocities and flood hazard ratings. There are a number of modelling options available within TUFLOW to cater for buildings, full details of which can be found in Syme (2008)[27]. These include blocking out the building footprint, applying a high surface roughness, adding form loss, representing external walls with an opening downstream, representing external walls with an opening upstream, representing a porous building, and representing a porous building with form losses.

For this modelling study, the individual buildings were represented using a high Manning's *n* relative to the surrounding roads and gardens. The grid resolution of modelling for this study is mostly 10m. At this resolution, approaches such as blocking out the footprint of the building is likely to create unrealistic continuous barriers to flooding since the narrow flow paths between

[27] Syme W. J. (2008). Flooding in Urban Areas – 2D Modelling Approaches for Buildings and Fences. Engineers Australia, 9th National Conference on Hydraulics in Water Engineering.

building and along small roads are unlikely to be fully represented at 10m resolution. This approach allows flow to pass across the building accounting for flood storage but floodwaters are slowed down by the increased surface roughness. Sensitivity testing was carried out on this parameter to ensure this method is appropriate.

5.6.4 Flood defence configuration, operation and data

The Caldicot and Wentlooge Levels consist of reclaimed land of which a substantial proportion lies below the MHWS level. For this reason, a continuous line of sea defences exists to prevent flooding from the sea.

The area is also relatively flat and the alluvium plains provide good quality agricultural land. This is a major type of land use in the region. These low lying, flat areas are drained by a network of artificial reen channels, which direct fluvial flow out to the sea.

5.6.5 Caldicot defences

Formal coastal defences in the study area are shown on Figure 5-17. The coastal frontage is defended by two stretches of earth embankment, the first running from Park Redding in Chepstow to Passage Gout Wharf and the second from Sudbrook to Uskmouth Power Station. Crest level survey data is available along the embankments and was provided by NRW. The survey was undertaken in 2013 and is the most recent data available.

Other formal fluvial and coastal defences not included in the 2013 NRW crest level survey are:

- a section of raised earth embankments at Portland Grounds and Chapel Farm. Crest levels at this location derived from 2015 design drawings
- earth embankment at Mireland Pill reen. No recent survey data at this location is available and so crest levels taken from LIDAR data
- series of embankments and raised concrete walls which make up the fluvial defences on the east bank of the River Usk from Uskmouth Power Station to the M4 Bridge. Extensive investigation of these defences was carried for a previous modelling study of Newport.

For the "No Defences" model simulations, the formal defences were removed from the model DTM manually, lowering the elevation down to the ground level. This was not undertaken for areas of high ground that forms a natural defence. The formal defences identified for removal in the "No Defences" scenario are shown in Figure 5-18.

Infrastructure

The infrastructure included in the model primarily consists of the railway and M4 motorway as shown in Figure 5-17. The railway line runs east to west across the model from Chepstow Railway Station in the east to Somerton in Newport in the west, then continues north to the M4 Bridge in Newport and south to Uskmouth Power Station. The section of the M4 in the model runs from Undy to Pill Farm Industrial Estate. Also included in the model infrastructure are a series of A and B roads in Newport and in the Caldicot Levels, including sections of the A48, the B4237, the B4596 and the B4591.

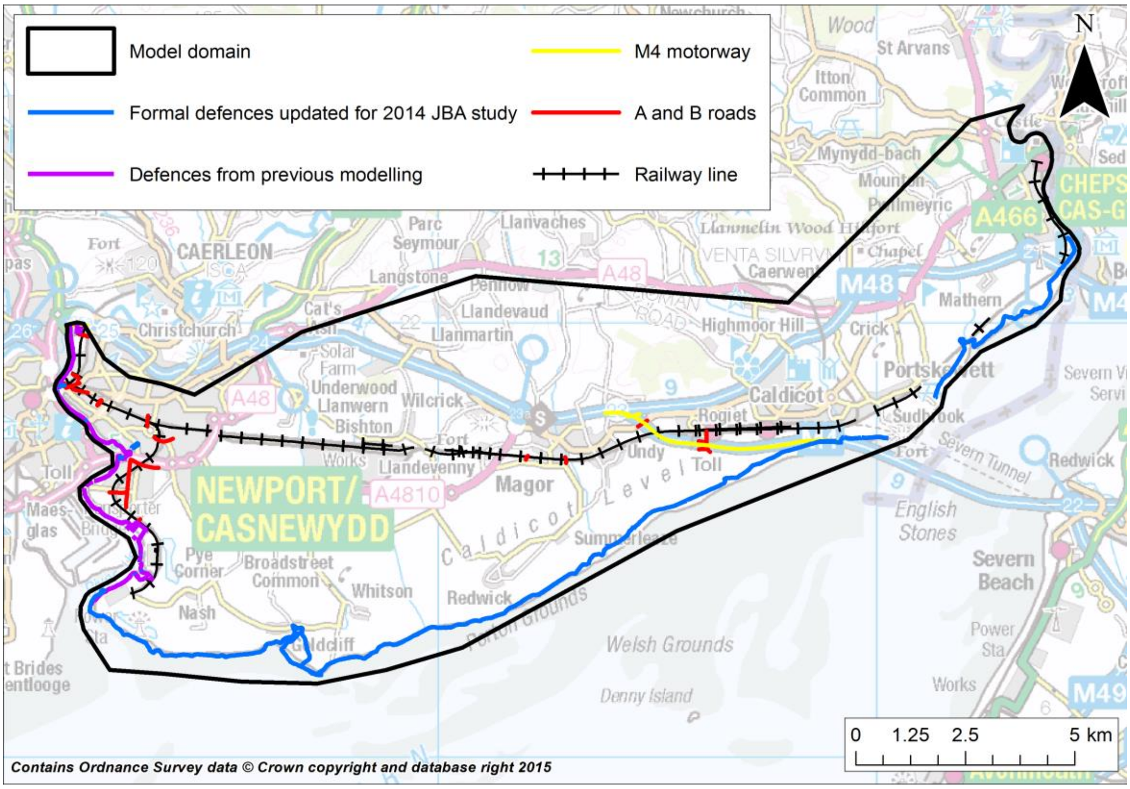


Figure 5-17: Formal flood defences and infrastructure included in the Caldicot TUFLOW model

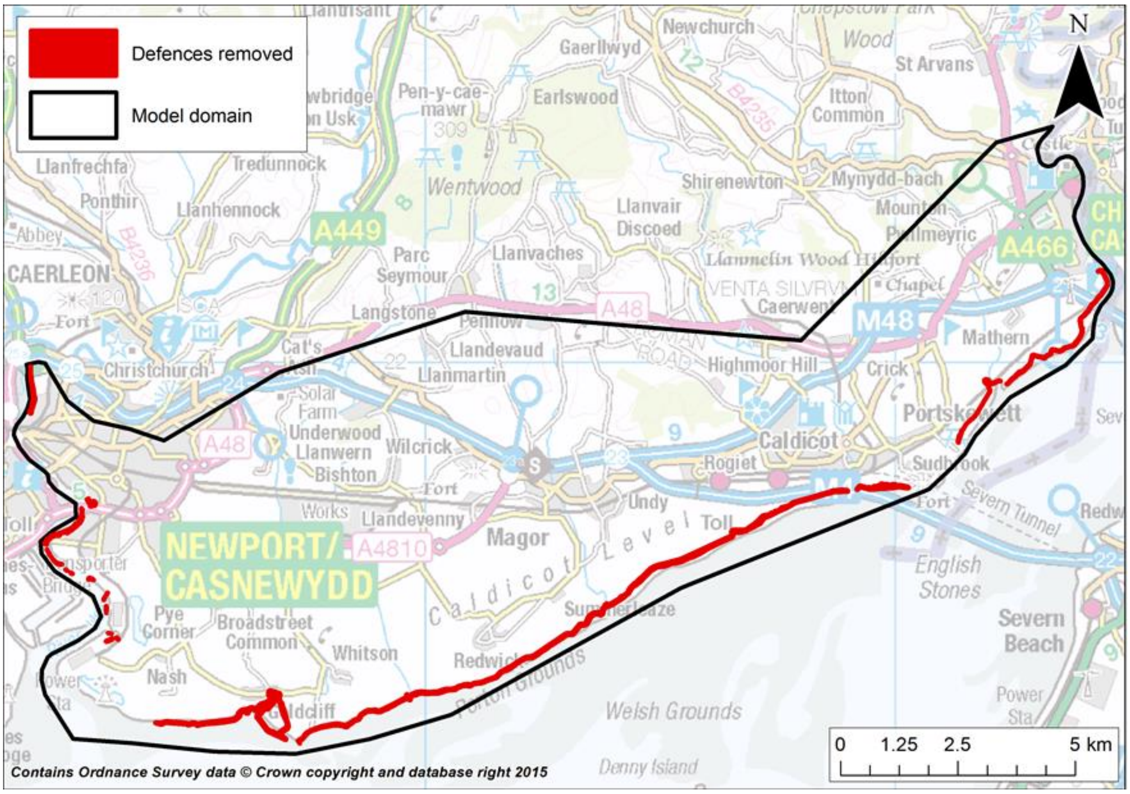


Figure 5-18: Formal flood defence footprints removed from the Caldicot TUFLOW model

Structures

Along the coastal frontage and up the River Usk 18 flood defence structures where identified in the Caldicot TUFLOW model domain, as shown on Figure 5-19. These structures are primarily flapped tidal outfalls which run along the coastal frontage from south of Nash to Chepstow. Details of the flood defence structures can be seen in Table 5-4. In the "With Defences" scenario, it was assumed that the structures with tidal flaps are closed and flow does not pass through them. In the "No Defences" scenario, the structures were assumed to be open or were removed as they form part of the defence system except from the rectangular culvert on Liswerry Pill. In addition to the 18 flood defence structures, inland culverts exist to allow flow under the railway and roads (Figure 5-19). These were included in all model scenarios.

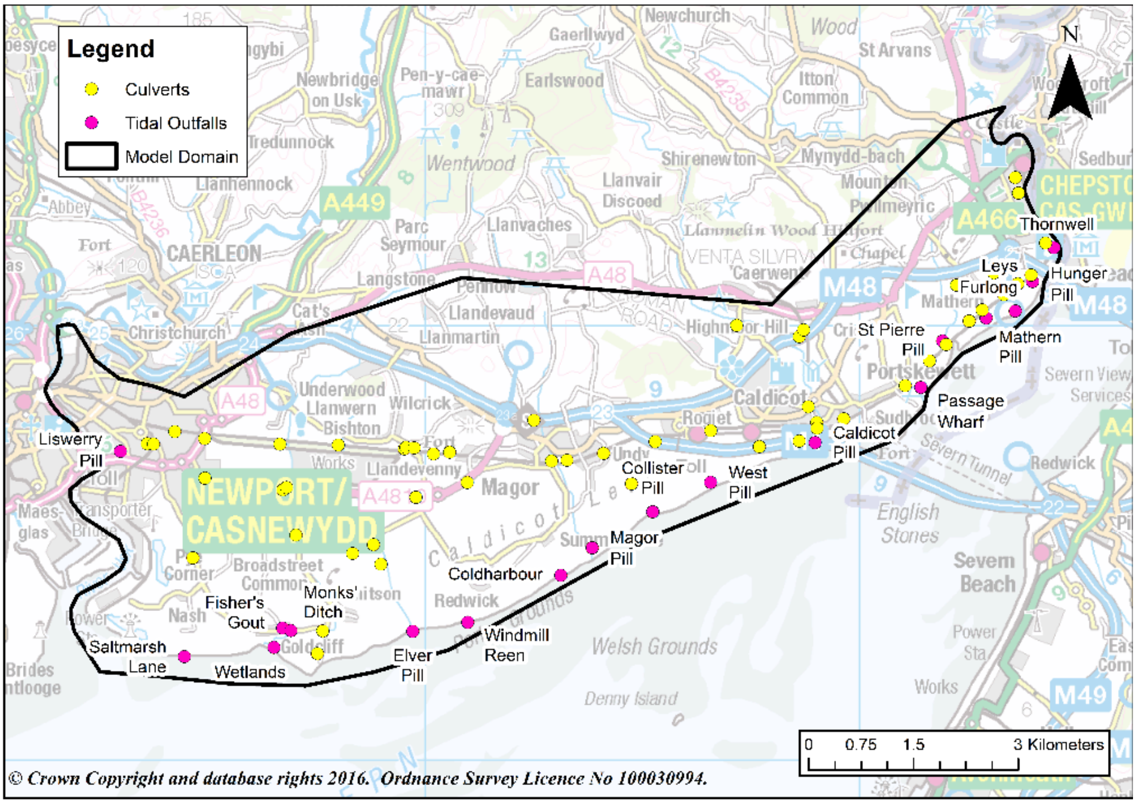


Figure 5-19: Flood defence structures and inland culverts included in the Caldicot TUFLOW model

Table 5-4: Caldicot flood defence structures

Asset Name	Asset Description	Easting	Northing
Fisher's Gout	Unidirectional flapped circular outfall	336707	183025
Monks' Ditch	Unidirectional flapped rectangular outfall	336901	182968
Elver Pill	Unidirectional flapped circular outfall	339710	182955
Windmill Reen	Unidirectional flapped rectangular outfall	340977	183163
Coldharbour	Unidirectional flapped circular outfall	343123	184243
Magor Pill	Unidirectional flapped circular outfall	343847	184875
Collister Pill	Unidirectional flapped circular outfall	345241	185704
West Pill	Unidirectional flapped circular outfall	346577	186384
Caldicot Pill	Unidirectional flapped rectangular outfall	348985	187302
Passage Wharf	Unidirectional flapped circular outfall	351423	188571
St Pierre Pill	Unidirectional flapped circular outfall	351925	189641
Mathern Pill	Unidirectional flapped circular outfall	352925	190170
Leys Furlong	Unidirectional flapped circular outfall	353602	190331
Hunger Pill	Unidirectional flapped circular outfall	353999	191008
Thornwell	Unidirectional flapped circular outfall	354491	191788
Saltmarsh Lane	Unidirectional flapped circular outfall	334446	182369
Wetlands	Circular culvert	336511	182581
Liswerry Pill	Rectangular culvert	332973	187099

Two pumping stations in the Caldicot Levels are used to aid in the drainage of the low-lying land and both were included in the model; the pumping station at Collister Pill and the pumping station and Archimedean screw at Greenmoor (Figure 5-20).

Details of the two pumping stations start and stop operating levels can be found in Table 5-5. The pumping station at Collister Pill has two pumps which operate together with a total pump capacity of 907l/s. If the capacity of the first pump is exceeded, the second pump switches on. At Greenmoor pumping station there is a large traditional pumping station with two pumps, a main pump and a standby pump in case of failure. The station is designed to pump at a capacity of 750l/s. The pumping station works in tandem with an Archimedean screw which has a capacity of 80l/s. The Archimedean screw comes on first, and during periods of low flow is the only pump to operate. The traditional pumps only work during times of high flow.

Table 5-5: Levels of operation for Greenmoor and Collister Pill pumping stations

Action	Greenmoor pump level (mAOD)	Greenmoor Archimedean screw level (mAOD)	Collister Pill pump level (mAOD)
Main start	3.60	3.40	2.40
Backup start	3.70	-	2.48
Main stop	1.97	3.10	1.97
Backup stop	-	-	1.97

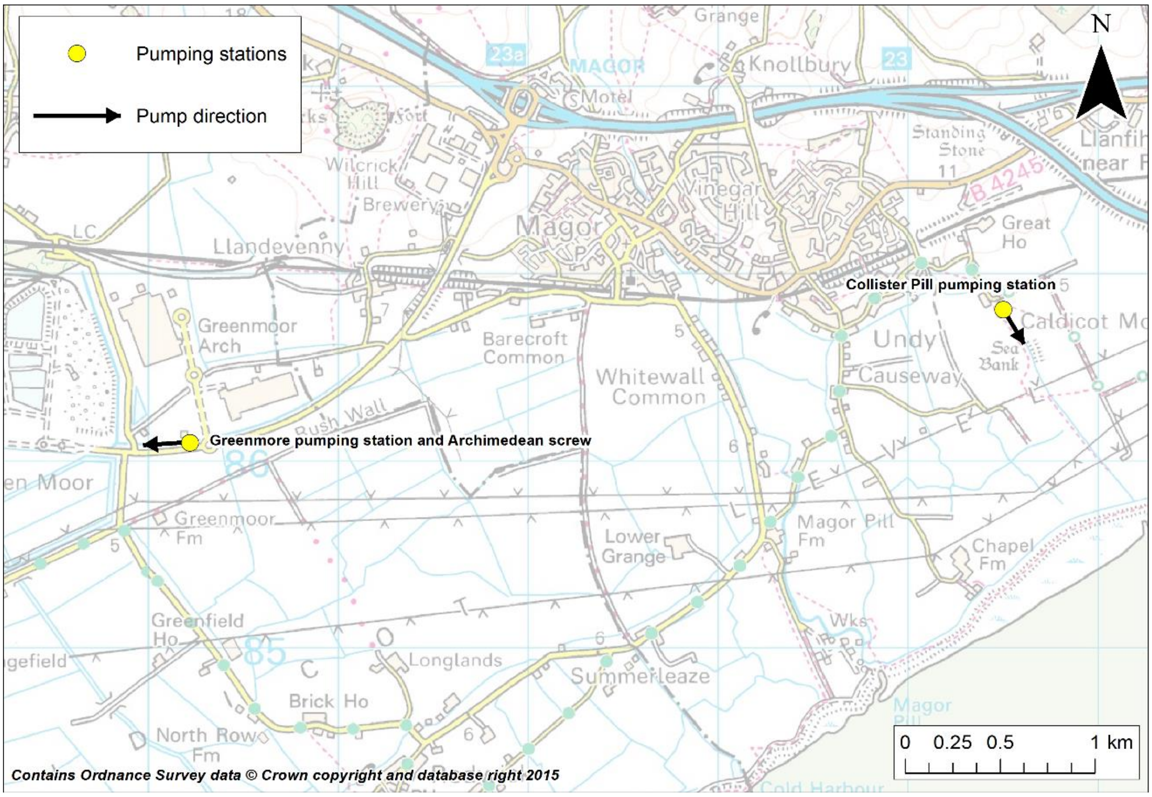


Figure 5-20: Pumping stations included in the Caldicot TUFLOW model

5.6.6 Wentlooge defences

The Wentlooge coastal frontage defence consists of an earth embankment which runs from Tredegar Pill in the east to Rumney Sea Wall in the west, as shown on Figure 5-21. As with the Caldicot model, crest levels along the embankments are available from 2013 survey. This is the most recent source of data.

Other formal coastal and fluvial defences in the area consist of:

- a grassed earth embankment running from the A48 to the railway line in Duffryn
- the raised concrete walls at Alexandra Docks in Newport

- a grassed earth embankment running along the Ebbw River from Pont Ebbw to the A48 in Maes-glas
- the series of embankments and raised concrete walls which make up the fluvial defences both in Newport and up the west bank of the River Usk from Powder House Point to Crindau Pill.

These were identified from previous modelling undertaken for the Newport area in 2011

Other identified defences that were included in the model are:

- raised earth embankments in Parc Tredelerch and at Rumney Playing Fields - included in the NRW's national defence dataset)
- section of raised earth embankment at Tabbs Gout - crest levels available from the 2015 design drawings.

In the "No Defences" scenario, the formal flood defence footprints are removed from the model. Defences identified to remove are shown on Figure 5-22.

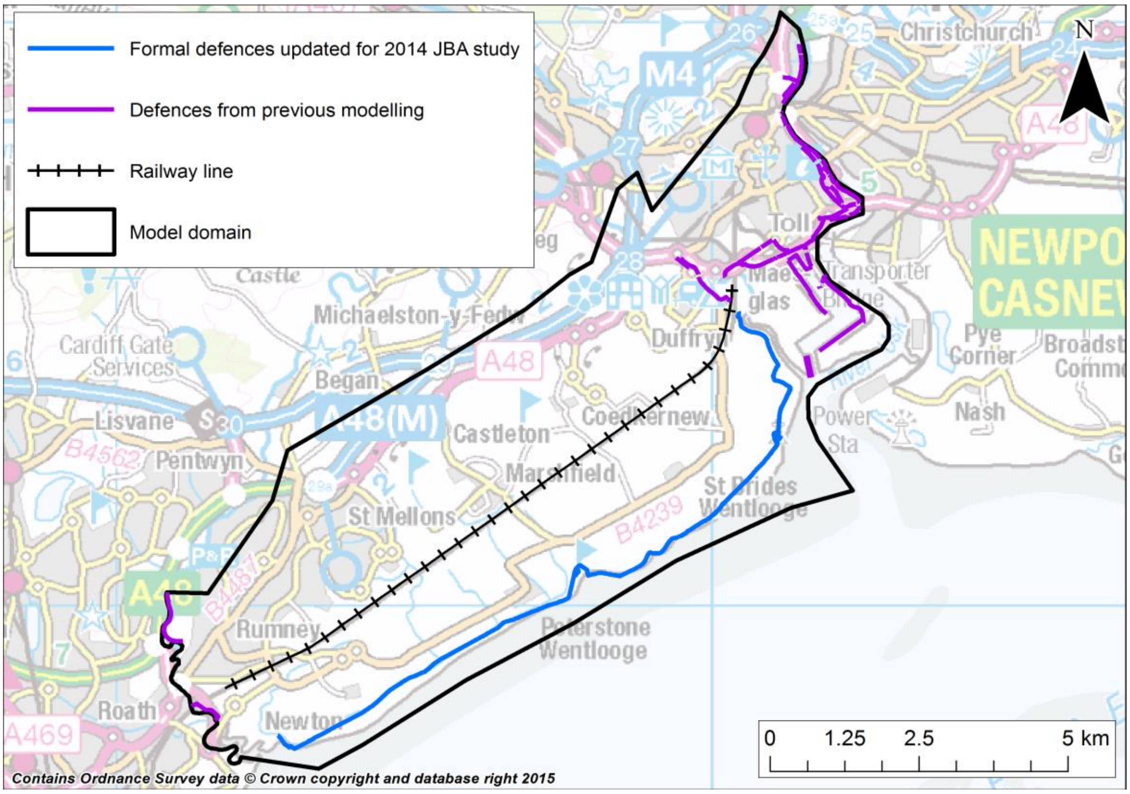


Figure 5-21: Formal flood defences and infrastructure included in the Wentlooge TUFLOW model

Infrastructure

The only infrastructure identified in the Wentlooge area was the railway line which runs west to east across the model domain, from Parc Tredelerch to the Wentlooge Level Playing Fields. Infrastructure was kept in the model for all the scenarios modelled according to the latest guidance for generating areas benefitting from defences.

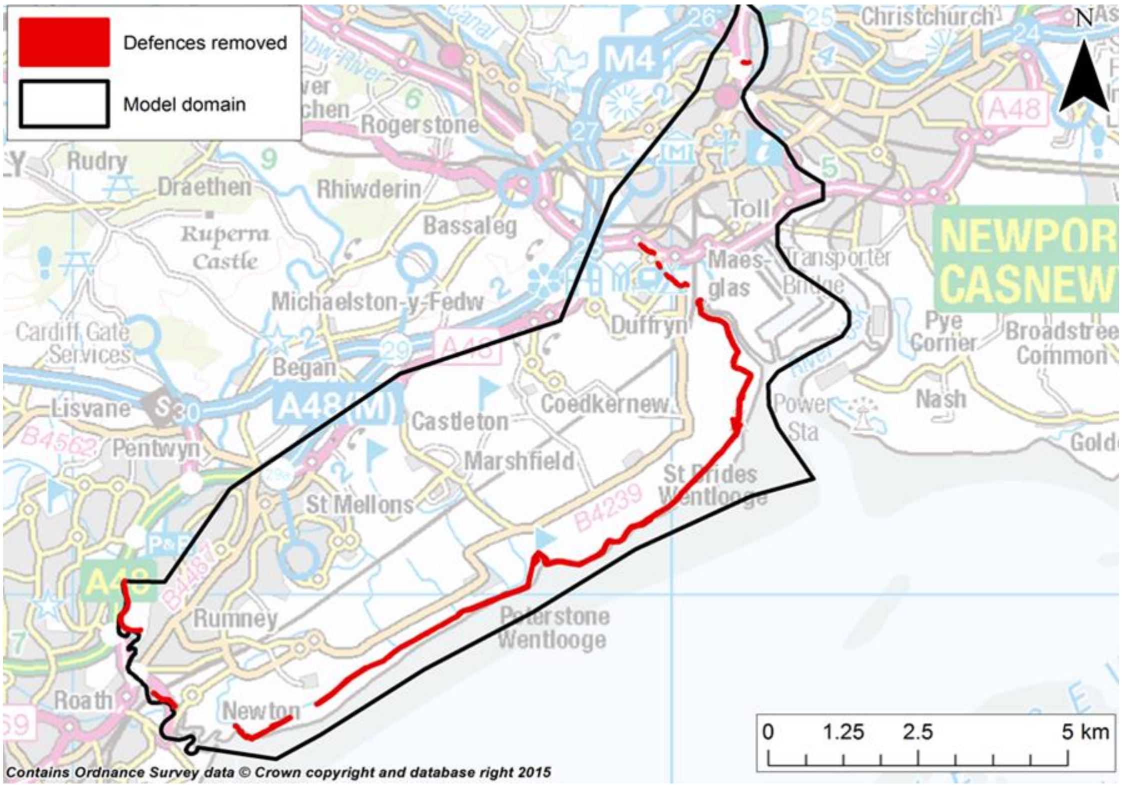


Figure 5-22: Formal flood defence footprints removed from the Wentlooge TUFLOW model

Structures

Six flood defence structures were identified in the Wentlooge area, as shown in Figure 5-23 and detailed in Table 5-6. All of the flood defence structures are tidal outfalls fitted with a flapped tidal gate. In the "With Defences" scenario, the flaps close automatically such that no tidal water passes through them. The structures were removed in the "No Defences" scenario else assumed open for the duration of the model simulation. Culverts were also identified in the study areas from previous modelling and from OS map data where water flows under roads and railways on the floodplain, as shown in Figure 5-23. These were included in the modelling for this study and included in all scenarios modelled.

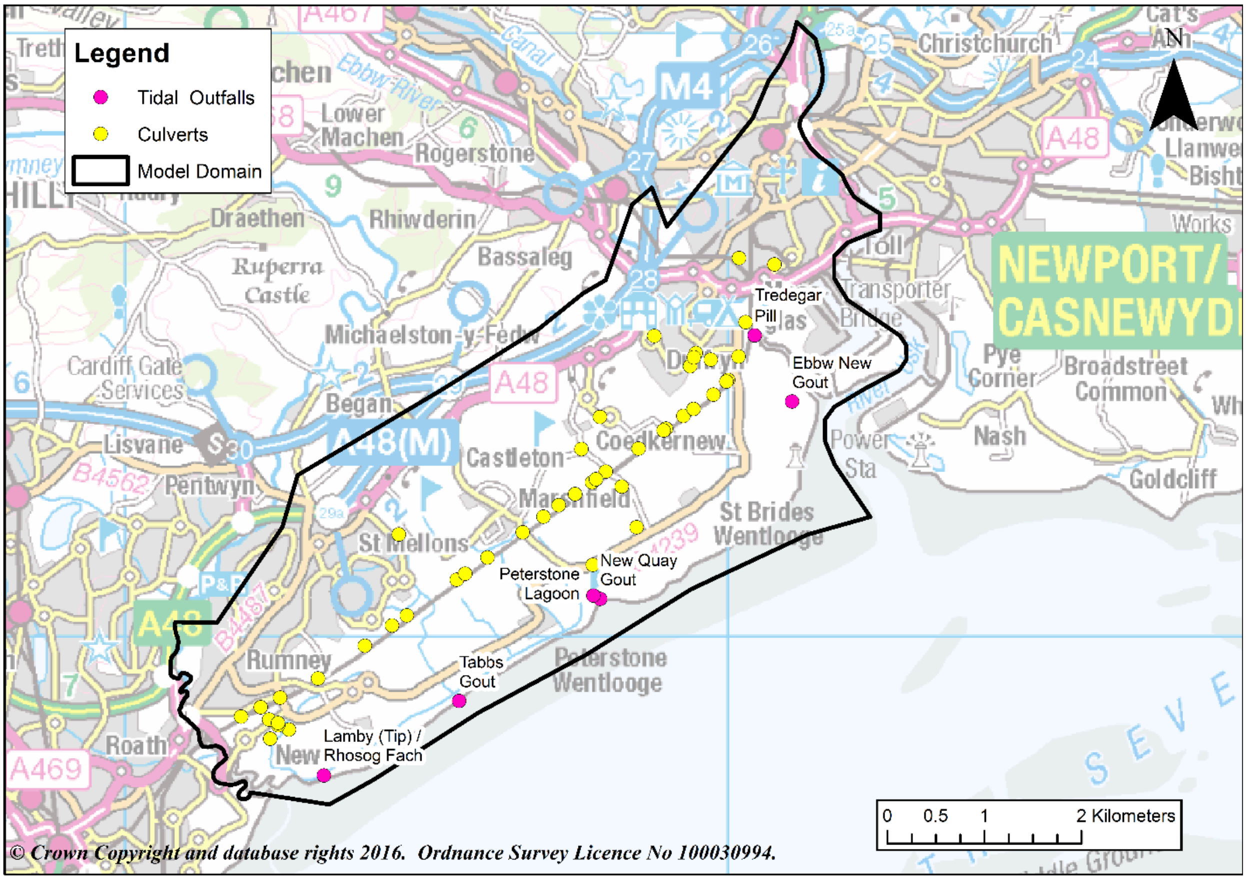


Figure 5-23: Flood defence structures and inland culverts included in the Wentlooge TUFLOW model

Table 5-6: Wentlooge flood defence structures

Asset Location	Asset Description	Easting	Northing
Tredegar Pill	Unidirectional flapped rectangular outfall	330435	184990
Ebbw New Gout	Unidirectional flapped rectangular outfall	331070	183885
New Quay Gout	Unidirectional flapped circular outfall	327898	180628
Peterstone Lagoon	Unidirectional flapped rectangular outfall	327756	180688
Tabbs Gout	Unidirectional flapped circular outfall	325450	179090
Lamby (Tip) / Rhosog Fach	Unidirectional flapped circular outfall	323284	177713

6 Breach modelling

The objective of the breach modelling work was to provide boundary conditions for the TUFLOW flood inundation model to estimate the flood extents, depths, velocities and subsequent risk. To achieve this, the HR Wallingford used the EMBREA (EMbankment BREAch) model to simulate the breaching process for five embankments at Wentlooge and 12 embankments at Caldicot.

6.1 An overview of the EMBREA model

In the development of the EMBREA model, HR Wallingford undertook extensive research to identify the best approach or tool to model the breaching of embankments and embankment dams. This included reviewing the existing methodologies used to model the failure (Mohamed 1998 and 2002)[28], developing an improved model (Mohamed et al. 2002a)[29] and testing the performance of the existing tools against field and laboratory physical modelling (IMPACT 2005)[30].

The research into breach modelling approaches and models showed that a number of deficiencies existed. HR Wallingford therefore developed the EMBREA model to meet industry needs for the prediction and management of dam breach formation due to overtopping or piping through flood defence embankments and embankment dams. Drawing on international and UK research EMBREA is considered a state-of-the-art tool for predicting breach growth, which provides an estimate of the rate at which the embankment might fail under different hydraulic conditions.

Research within the EC Funded IMPACT Project (www.impact-project.net) has shown that the performance of the EMBREA[31] model is, on average, the best of all models compared. This was based on the data of five prototype scale field tests and 22 laboratory tests, an extensive number of validation tests compared to existing tools. HR Wallingford have also continued to refine and extend the capabilities of this modelling tool through internal Company Research and the EC FLOODsite Project (www.floodsite.net)[32].

Given the above, the EMBREA model has been used in this study to model the overtopping and piping failure of the Wentlooge and Caldicot embankments.

Modelling overtopping in EMBREA is carried out assuming that an initial breach channel through the crest and downstream face is initiated. The dimensions of this channel are defined by the user. This 'initiation' channel constrains the initial breach flow and provides the focal point for breach simulation. In practical terms, this simulates a hole or dip in the embankment crest that might arise for a number of reasons, resulting in the focus of overtopping flow and scour. If the water level is below the initiation channel invert level, no further scour takes place. If the water level exceeds the initiation channel invert and the embankment crest level, further overtopping will occur, with the model simulating the following processes:

- Flow from the breach channel and over the crest
- Continuous erosion of material through the breach channel
- Slope instability of the breach channel sides.

Modelling piping in EMBREA is carried out assuming that a circular pipe has already been established along the embankment between the water source and the downstream face. The model then simulates the following processes:

- Erosion of the material in the pipe
- The collapse of the upper dammed section above the pipe, either under its own weight or by the water pressure forces
- Erosion of the dam body in a similar way to an overtopping failure.

---

[28] Mohamed, M. A. A., 1998, "Informatic Tools for the Hazard Assessment of dam Failure", MSc. Thesis, IHE, Delft, the Netherlands.

Mohamed, M. A. A., 2002, "Embankment Breach Formation and Modelling Methods". PhD. Thesis. The Open University.

[29] Mohamed, M. A. A., Samuels, P. G., Morris, M. W., and Ghataora, G. S., 2002a, "Improving the Accuracy of Prediction of Breach Formation through Embankment Dams and Flood Embankments", River Flow Conference, Louvain-la-Neuve, Belgium.

[30] IMPACT, 2005, "The IMPACT Project: Final Technical Report". Project technical reporting.

[31] Previously known as "HR Breach"

[32] FLOODsite (2007), Failure mechanisms for flood defence structures, FLOODsite Report T04-06-01, Task 4. www.FLOODsite.net

It has to be noted that the initiation of the overtopping and piping processes is not modelled in EMBREA.

6.2 Breach modelling

A number of modelling runs were undertaken for five representative embankments in Wentlooge and 12 representative embankments in Caldicot to establish the outflow hydrograph from the failure of each embankment. Table 6.1 shows the embankments that were modelled.

Table 6.1: Modelled embankments

Location	Defences
Wentlooge	W3, W4, W5, W8 and W9
Caldicot	C16, C17, C19, C20, C21, C22, C25, C27, C29, C30, C31 and C32

6.3 Model setup

This section provides a description of the model set up, including modelling boundary conditions, initial conditions and embankments' geometry and soil parameters.

Upstream boundary condition: A number of different water level conditions with different wave heights have been considered for the 5, 20, 200, 1000, 200+CC (Climate Change) and 1000+CC year events. These scenarios recognised the potential for breaching during very high water level and small wave conditions, even if they do not result in the worst-case overtopping rate. Generally, three following conditions were considered[33]:

1. The peak overtopping event calculated
2. A modelled event with a similar peak overtopping return period, but a higher water level and lower wave height. These were then factored to approximately match the required return period.
3. The extreme water level with no wave height.

Four tidal cycles were modelled for the condition that produces the highest breach peak outflow (i.e. worst case). For the downstream boundary condition it was assumed that no breach drowning takes place. The embankment geometry is summarised in Table 6-2.

Table 6.2: Embankment geometry

Defence / Parameter	Crest level (mAOD)	Base level (mAOD)	Crest width (m)	Downstream Slope (1:x)	Upstream slope (1:x)
Wentlooge					
W3	9.41	6.95	3.00	2.40	3.75
W4	9.75	6.95	3.00	2.20	2.00
W5	10.06	6.00	3.50	2.25	3.00
W8	9.56	7.00	3.65	3.25	2.00
W9	9.59	7.50	5.15	7.25	5.00
Caldicot					
C16	9.83	6.00	2.10	2.30	1.50
C17	9.72	6.02	2.00	2.75	1.50
C19	9.77	5.90	2.30	2.00	1.50
C20	9.78	6.55	2.15	1.80	1.50
C21	9.41	6.70	3.35	2.85	2.90
C22	9.06	7.50	1.50	3.00	2.90
C25	9.13	7.53	2.15	3.15	2.75
C27	9.30	7.85	2.50	2.70	3.00
C29	8.97	7.78	2.40	4.00	3.85

[33] It should be noted that there is an infinite number of combinations of wave heights and water levels that correspond to an overtopping event. It is not possible or appropriate to model these, therefore these three events have been chosen to represent the likely range covering the worst case scenario for breach modelling, including the case covering the extreme sea level for each return period.

Defence / Parameter	Crest level (mAOD)	Base level (mAOD)	Crest width (m)	Downstream Slope (1:x)	Upstream slope (1:x)
C30	9.29	7.25	5.10	3.10	3.60
C31	9.05	7.85	2.30	2.70	2.70
C32	9.11	6.50	3.75	2.85	3.50

Failure modes: Based on the upstream conditions and the embankment geometry (see above), the failure mode for each defence was defined. Overtopping was considered as the likely failure mode if the water levels significantly exceed the crest levels. In other cases the piping failure mode was considered. Table 6.3 shows the failure modes per defence and return period.

Table 6.3: Failure modes per defence and return period

Defence	Return Period					
	5	20	200	1000	200+CC	1000+CC
Wentlooge						
W3				Piping	Piping	Overtopping
W4					Piping	Piping
W5				Piping	Piping	Piping
W8					Piping	Overtopping
W9					Piping	Overtopping
Caldicot						
C16					Piping	Overtopping
C17					Piping	Overtopping
C19					Piping	Overtopping
C20					Piping	Overtopping
C21			Piping	Piping	Overtopping	Overtopping
C22		Piping	Piping	Overtopping	Overtopping	Overtopping
C25	Piping	Piping	Overtopping	Overtopping	Overtopping	Overtopping
C27			Piping	Overtopping	Overtopping	Overtopping
C29			Overtopping	Overtopping	Overtopping	Overtopping
C30		Piping	Piping	Overtopping	Overtopping	Overtopping
C31		Piping	Overtopping	Overtopping	Overtopping	Overtopping
C32					Overtopping	Overtopping
Notes: +CC = Plus Climate Change Empty cell = No run due to unlikelihood of breaching						

Initial conditions: As described above, the EMBREA model assumes that an initial pipe for piping failure mode or an initial notch for overtopping failure mode has been already established to model the defence failure. Therefore, for a piping failure mode, a pipe with a 0.10 m diameter was assumed to be formed along the defence to initiate the piping failure. The level of this pipe was varied to produce the highest breach outflow.

For the overtopping failures a notch with a 0.15m depth was assumed to be formed. This depth was based on the RELIABLE tool results which show that the probability of defence failure at this overtopping depth is higher than 50% for the base run (see section **Error! Reference source not found.** for details). In order to determine the notch width, a number of initial runs were undertaken with various widths. These runs show that the final breach width is dependent on the choice of the initial breach width. Therefore, it was decided with the Client to base the selection of the initial breach width on low spots along the defences that are overtopped by a depth of 0.15m or more. For example, for defence C25, the crest levels were extracted from the LIDAR data that was provided by the Client. The crest levels data were then analysed and a number of consecutive low spots of an equivalent width of 30m were found. Therefore, the initial notch that was used for defence C25 was 0.15m by 30m. The same exercise was repeated for each defence that can fail by overtopping to select the initial notch width. The outcome of this exercise is shown in

Table 6.4. As can be seen for a number of defences more than one breach location (i.e. low spot) was found. In that case, a breach run was carried out for each breach location. For defences W3 and C21, an initial width of 1m was assumed as these defences are going to be raised to a level

of 9.41mAOD hence crest levels are likely to be consistent and the existence of major low spots is unlikely.

Table 6.4: Initial breach width for each defence failing by overtopping

Defence	Number of potential breach locations	Initial Breach width (m)
Wentlooge		
W3	1	1
W8	3	68
		87
		177
W9	1	66
Caldicot		
C16	1	50
C17	1	65
C19	1	200
C20	1	12
C21	1	1
C22	1	34
C25	1	30
C27	2	15
		32.5
C29	2	30
		56
C30	1	20
C31	1	17
C32	2	4
		10

Soil properties: Based on the soil investigation reports that were provided by the Client, the following soil properties were used for each defence (See Table 6.5).

Table 6.5: Soil properties

Parameter	Defences W3,W4 and W5	Defences W8 and W9	All Caldicot Defences
Porosity	0.32	0.30	0.37
Dry unit weight (t/m3)	1.58	1.60	1.42
Friction Angle	28	23	25
Cohesion (kN/m2)	15	1**	21.5
Plasticity index	Non plastic	32.5	37
Erodibility coefficient (cm/N.s)	1.41*	1.50*	1.21*
Critical shear stress (N/m2)	0.1**	0.1**	0.1**
Notes: * Estimated based upon the % of clay and dry density. ** assumed			

6.4 Breach modelling results

The following sections provide details of the results of modelling work undertaken for each defence.

6.4.1 Results by defence

Table 6.6 shows the results summary by defence. Results show that the breach peak outflow increases as return period increases (i.e. the event becomes severer) which is expected. The highest breach peak outflow is estimated at defence C19 failing by overtopping under the 1000 +CC year return period. Two factors contribute to this high value the relatively large initial breach width (200m) and height of the embankment (3.78m). The second highest breach peak outflow was at defence W8 with an initial width of 180m and height of 2.56m.

Table 6.6: Results summary by defence

Defence	Return Period	Peak Breach Outflow (m3/s)	Final Breach Depth (m)	Final Breach Width (m)
Wentlooge				
W3	1000	8.50	1.86	2.60
	200 +CC	25.84	2.34	6.49
	1000 +CC	41.37	2.46	7.66
W4	200 +CC	22.26	2.79	6.09
	1000 +CC	32.83	2.79	6.70
W5	1000	35.29	4.05	6.94
	200 +CC	71.35	4.06	9.94
	1000 +CC	99.88	4.06	10.96
W8	200 +CC	22.67	2.55	5.01
	1000 +CC	668.00	2.56	89.78
	1000 +CC	524.11	2.56	70.79
	1000 +CC	1392.25	2.56	179.90
W9	200 +CC	10.12	2.09	3.15
	1000 +CC	358.04	2.09	68.99
Caldicot				
C16	200 +CC	61.86	3.64	9.37
	1000 +CC	540.80	3.65	53.92
C17	200 +CC	68.55	3.70	8.98
	1000 +CC	1000.40	3.52	71.52
C19	200 +CC	108.09	3.87	10.30
	1000 +CC	3424.85	3.78	210.35
C20	200 +CC	50.85	3.23	8.45
	1000 +CC	297.63	3.23	27.68
C21	200	10.76	1.84	3.40
	1000	16.28	1.91	3.97
	200 +CC	72.80	2.71	8.92
	1000 +CC	83.60	2.71	9.53
C22	20	2.68	1.37	1.61
	200	5.24	1.56	2.12
	1000	161.21	1.56	38.28
	200 +CC	380.38	1.56	42.69
	1000 +CC	513.01	1.56	42.69
C25	5	1.29	0.59	1.47
	20	3.66	1.60	1.80
	200	119.24	1.60	34.08
	1000	189.56	1.60	35.10
	200 +CC	356.53	1.60	36.86
	1000 +CC	476.66	1.60	36.80
C27	200	3.70	1.43	2.03
	1000	125.94	1.38	34.68
	1000	61.27	1.38	17.20
	200 +CC	300.22	1.38	36.10
	200 +CC	153.21	1.45	22.09
	1000 +CC	414.13	1.38	36.92
	1000 +CC	209.58	1.45	22.16
C29	200	69.57	1.13	31.78
	200	127.27	1.13	57.78
	1000	163.75	1.13	32.56
	1000	298.91	1.13	58.55
	200 +CC	371.44	1.13	34.75
	200 +CC	669.47	1.13	60.67
	1000 +CC	468.93	1.13	35.47
	1000 +CC	856.37	1.13	62.33
C30	20	5.85	2.04	2.69
	200	12.77	2.04	3.71
	1000	108.04	2.04	21.68
	200 +CC	293.44	2.04	26.20
	1000 +CC	396.11	2.04	27.56
C31	20	1.97	1.14	1.44
	200	44.31	1.20	17.98

Defence	Return Period	Peak Breach Outflow (m3/s)	Final Breach Depth (m)	Final Breach Width (m)
	1000	99.81	1.20	18.59
	200 +CC	213.75	1.20	21.07
	1000 +CC	295.64	1.20	22.55
C32	200 +CC	210.84	2.61	22.30
	200 +CC	131.03	2.61	14.92
	1000 +CC	280.00	2.61	22.77
	1000 +CC	173.32	2.61	15.46

6.4.2 Results by return period

Table 6.7 shows the results summary by return period. Results show that for low return periods (i.e. 5 and 20 years), the breach peak outflow is relatively low and does not exceed 6m3/s. As the return periods increases the breach peak outflow increase and becomes associated with significant overflowing along the full crest of the defence particularly in the 200+CC and 1000+CC year return periods. This overflowing needs to be taken into account when modelling the inundation in the TUFLOW model.

Table 6.7: Results summary by return period

Return Period	Defence	Peak Breach Outflow (m3/s)	Final Breach Depth (m)	Final Breach Width (m)
5	C25	1.29	0.59	1.47
20	C22	2.68	1.37	1.61
	C25	3.66	1.60	1.80
	C30	5.85	2.04	2.69
	C31	1.97	1.14	1.44
200	C21	10.76	1.84	3.40
	C22	5.24	1.56	2.12
	C25	119.24	1.60	34.08
	C27	3.70	1.43	2.03
	C29	69.57	1.13	31.69
		127.27	1.13	57.69
	C30	12.77	2.04	3.71
	C31	44.31	1.20	17.98
1000	W3	8.50	1.86	2.60
	W5	35.29	4.05	6.94
	C21	16.28	1.91	3.97
	C22	161.21	1.56	38.28
	C25	189.56	1.60	35.10
	C27	61.27	1.38	17.20
		125.94	1.38	34.68
	C29	163.75	1.13	32.45
		298.91	1.13	58.45
	C30	108.04	2.04	21.68
	C31	99.81	1.20	18.59
200+CC	W3	25.84	2.34	6.49
	W4	22.26	2.79	6.09
	W5	71.35	4.06	9.94
	W8	22.67	2.55	5.01
	W9	10.12	2.09	3.15
	C16	61.86	3.64	9.37
	C17	68.55	3.70	8.98
	C19	108.09	3.87	10.30
	C20	50.85	3.23	8.45
	C21	72.80	2.71	8.92
	C22	380.38	1.56	42.69
	C25	356.53	1.60	36.86
	C27	153.21	1.45	22.09
		300.22	1.38	36.10
	C29	371.44	1.13	34.75
		669.47	1.13	60.67
	C30	293.44	2.04	26.20

Return Period	Defence	Peak Breach Outflow (m3/s)	Final Breach Depth (m)	Final Breach Width (m)
	C31	213.75	1.20	21.07
	C32	131.03	2.61	14.92
		210.84	2.61	22.30
1000+CC	W3	41.37	2.46	7.66
	W4	32.83	2.79	6.70
	W5	99.88	4.06	10.96
	W8	668.00	2.56	89.78
		524.11	2.56	70.79
		1392.25	2.56	179.90
	W9	358.04	2.09	68.99
	C16	540.80	3.65	53.92
	C17	1000.40	3.52	71.52
	C19	3424.85	3.78	210.35
	C20	297.63	3.23	27.68
	C21	83.60	2.71	9.53
	C22	513.01	1.56	42.69
	C25	476.66	1.60	36.80
	C27	209.58	1.45	22.16
		414.13	1.38	36.92
	C29	468.93	1.13	35.47
		856.37	1.13	62.33
	C30	396.11	2.04	27.56
	C31	295.64	1.20	22.55
	C32	280.00	2.61	22.77
		173.32	2.61	15.46

6.5 Observations and Conclusions

The following points may be concluded from the results of modelling work undertaken:

- Piping and overtopping are likely failure modes for the defences in Wentlooge and Caldicot.
- Breach modelling was successfully undertaken using the EMBREA model for a number of return periods with and without climate change impacts.
- Breach modelling indicates that during a 5 to 20-year return period defences fail by piping, with a relatively low peak outflow (under 6m^3/s). As the return period increases, defences start to fail by overtopping, increasing the peak outflow to over 3,000m^3/s and 1,000m^3/s at defences C19 and W8 respectively.
- In addition to breach outflow, overflowing along the defence crest occurs during extreme events including climate change (e.g. 200+CC and 1000+CC events).

7 Flood defence reliability assessment

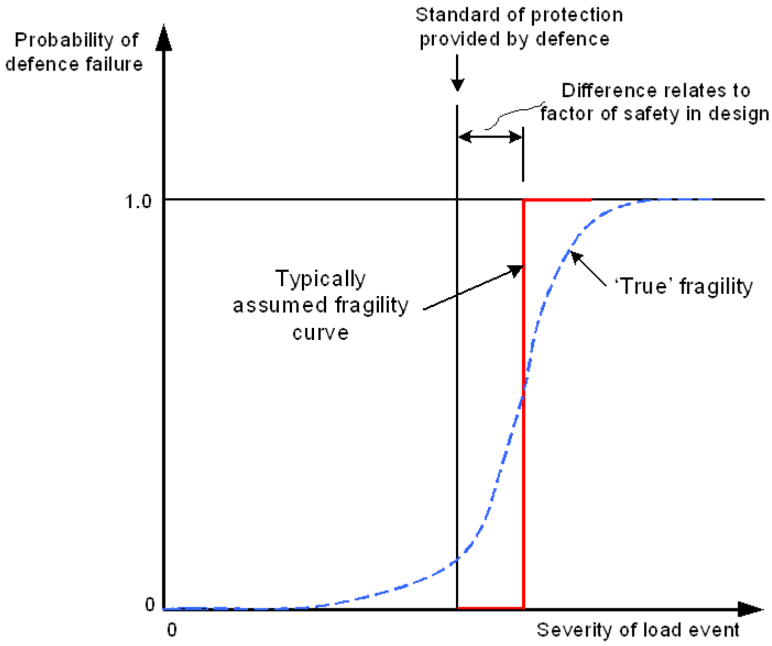
7.1 Objective

The objective of this part of the study was to undertake an assessment of the likelihood of failure of the flood defence infrastructure. HR Wallingford's RELIABLE tool has been used to undertake this assessment for five embankments in Wentlooge and Caldicot. The following sections give an overview of fragility curves and their uses, the RELIABLE tool, potential failure modes of the Wentlooge and Caldicot defences, input data and the reliability modelling results.

7.2 Overview of fragility curves

The fragility of a structure is defined as the probability of failure, which is conditional on a specific loading, e.g. wind, water levels etc.[34] The concept of fragility has been widely used in other industries to characterise structural performance across a range of imposed loads. The concept of fragility was first postulated for use in flood risk management in the United States of America (USA)[35].

The concept of fragility curves is shown in conceptual form in **Error! Reference source not found.**. In classical deterministic design, embankments are deemed to follow the red line and either no fail (probability of failure = 0) or fail (probability of failure = 1.0), switching between the two at the nominal failure load. In reality, embankment failure may follow the blue line. Even during small loads there is the potential for an embankment to fail, and conversely the defence may stand even when loads exceed the nominal failure load.



Source: HR Wallingford

Figure 7.1: Generic form of a fragility curve

Four generic approaches for deriving fragility curves have been identified[36], comprising of:

1. Judgemental – based on expert judgement.
2. Empirical – based on data collected on similar failures
3. Analytical – based on a wide range of numerical modelling tools
4. Hybrid methods – based on a combination of two or more of these approaches.

[34] Casciati F. and Faravelli L., 1991, Fragility Analysis of Complex Structural Systems, Research Studies Press

[35] USACE. 1993. Reliability assessment of existing levees for benefit determination, Engineering and Design, Engineer Technical Letter 1110-2-328.

[36] Schultz, M., Gouldby, B., Simm, J. and Wibowo, J. (2010) Beyond the Factor of Safety: developing fragility curves to characterize system reliability, Vicksburg, MS: Engineer Research and Development Center: Geotechnical and Structures Lab

The early UK approaches typically employed expert judgement based approaches to form curves[37]. Analytical approaches were introduced within NaFRA in 2004, which have been updated and used within this study. These allow the use of conventional physical process-based models in the absence of failure data, which is preferred due to the relatively low rates of embankment failures and subsequent data.

The following steps are required to generate a fragility curve for an embankment flood defences.

1. Define the overall function of the flood defence.
2. Systematically identify and analyse all relevant failure modes likely to lead to flooding, and the interaction between these failure modes. In this first stage analysis, conventional deterministic approaches can be helpful to eliminate unrealistic failure modes.
3. Identify an appropriate "model" to represent each failure mode. This could range from some kind of Limit State Equation (LSE) to a finite element numerical model. Having identified the Limit State Equation or model, the failure can be considered in terms of its reliability (Z):

   $Z = R - S_{NH} - S_H$

   a. where:
      - R = strength, which represents the gathering together of all terms or parameters which relate to the strength of the structure.
      - S = loading (S_{NH} is non-hydraulic loading and S_H is hydraulic loading), which represents the gathering together of all terms or parameters which relate to the magnitude of the loading.
4. Produce a schedule of the engineering parameters feeding into the LSEs, such as the width and form of the uncertainty bands.
5. Prepare fault trees that specify the logical sequence of all possible failure mechanisms leading to the failure of the defence.
6. Perform a series of reliability assessments under a series of different hydraulic loading conditions. Each analysis for a given loading condition comprises of a series of Monte Carlo simulations which represent the uncertainty bands for each input parameter. Failure arises when the combinations of parameter values in the limit state function gives a value for Z which is less than or equal to zero. The probability of failure for that loading is the number of times when the simulation gives Z as less than or equal to zero divided by the total number of simulations.
7. Step 6 is repeated for an appropriate series of different hydraulic loadings and the results interpolated to define a continuous fragility curve.

7.3 Uses of fragility curves

7.3.1 Use for embankment assessment

Design approaches, whether deterministic or semi-probabilistic, generally address potential failure modes independently. They create design solutions based on inclusion of margins of safety generally considered to be good practice. The fact that different approaches and safety factors are adopted for different mechanisms can therefore be accommodated without difficulty. When it comes to assessment of existing embankments, however, an approach is required which explicitly identifies the probability of failure for each mechanism and also permits the assessment and combination of the effect of the various relevant failure mechanisms on a consistent basis.

Fragility curves provide just such an approach, although they raise the question of the allowable probability of failure that might be considered reasonable. Dutch guidance[38] typically assumes that for the nominal design event, a maximum of 10% probability of failure should be allowed for all mechanisms combined.

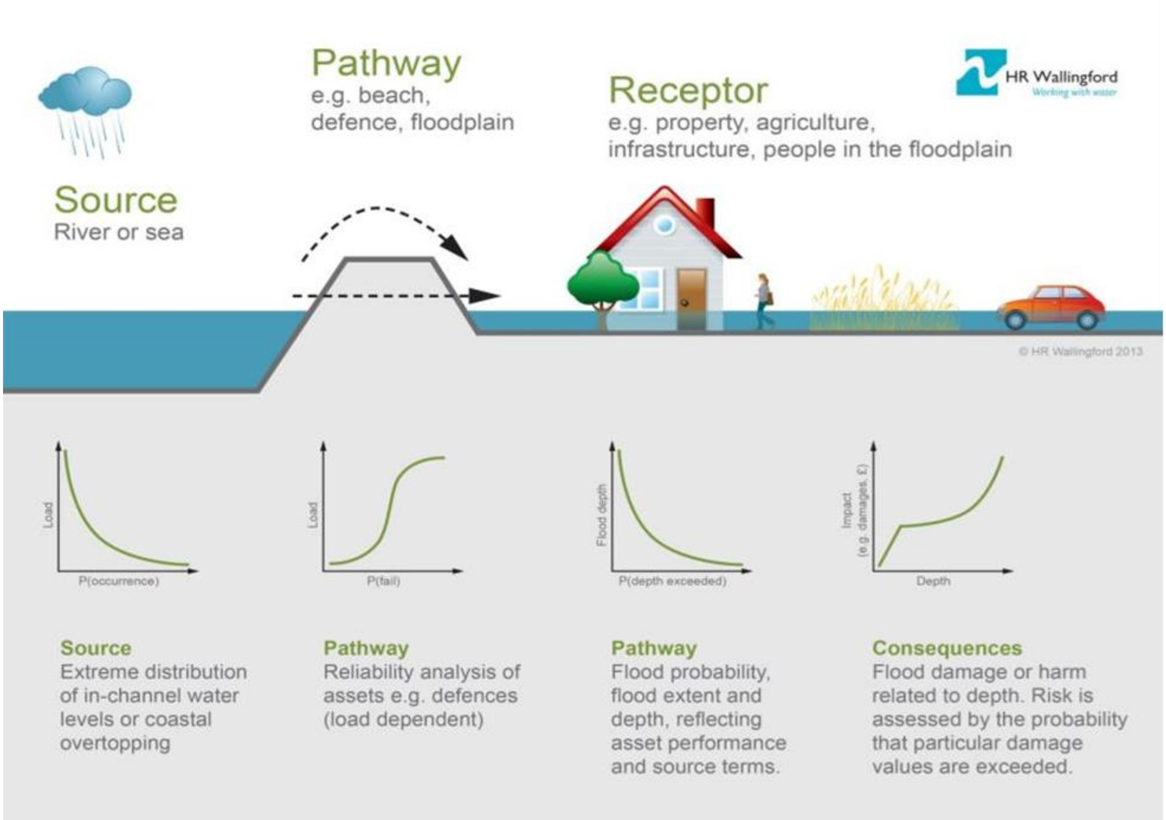
[37] HR Wallingford., (2002), Risk assessment for Flood and Coastal Defence for Strategic Planning, High Level Methodology: A Review Report SR603

[38] Simm, J., Gouldby, B., Sayers, P., Flikweert, J., Wersching, S. and Bramley, M. (2008) 'Representing fragility of flood and coastal defences: getting into the detail', in Proc. Eur. Conf. on Flood Risk Management: Research into Practice (FLOODrisk 2008), Rotterdam, Taylor & Francis, 621-631.

7.3.2 Use for flood risk analysis

Fragility curves have a significant role within overall risk analysis. They allow an improvement of traditional flood modelling and mapping methods that assume flood defences are either (i) not present or (ii) are present and cannot fail. Whilst modelling undertaken using these simple assumptions does have a purpose, it is important to recognise their limitations. If flood defences are present, assumption (i) can lead to significant overestimates of flood outlines and risk. If flood defences are robustly constructed and well maintained, then assumption (ii) is reasonable. However, any modelling that rely on assumption (ii) offer no means to provide an economic justification for undertaking defence maintenance or refurbishment activities. It is therefore not possible to explore the impact on risk of reducing maintenance expenditure. Moreover, where defences have deteriorated in time and there is the possibility of failure, assumption (ii) is likely to lead to an underestimation of risk and flood mapping extents.

The United State Army Corps of Engineers (USACE) and the Environment Agency in the UK has recognised these limitations for many years and thus adopted a flood risk analysis approach that explicitly represents the performance of flood defences through the use of fragility curves. This approach is conceptualised in **Error! Reference source not found.**.



Source: HR Wallingford

Figure 7.2: Source-Pathway Receptor conceptualisation of flood risk systems

Because the embankment segment associated with each component of residual flood risk can be identified, it is therefore possible to evaluate the relative residual flood risk associated with a range of embankment segments. This functionality was not used at the Caldicot and Wentlooge defences, although the fragility curves did permit comparison of the performance of the various embankment segments.

7.4 The RELIABLE tool

To make the above process easier, a flexible tool 'RELIABLE' was developed to analyse the reliability of flood defences[39,40]. This prototype software tool facilitates the construction of fault

[39] The tool being developed under the European project FLOODsite Task 7 and UK project FRMRC (WP4.4)

[40] FLOODsite (2007), Failure mechanisms for flood defence structures, FLOODsite Report T04-06-01, Task 4. www.FLOODsite.net

trees, selected from a range of over 50 different LSE's and enables reliability calculations (i.e. fragility curve generation) to be undertaken on a site specific basis, if data is available. The tool includes a total of 72 failure modes represented as simple LSEs, a flexible fault tree component, and a probabilistic failure analysis component based on Monte Carlo simulation. It is applicable to foreshores, dunes and banks; embankments and revetments; walls; and point structures, and accounts for hydraulic loading due to water level difference across a structure; wave loading; and lateral flow velocities.

The user interface of RELIABLE is provided via a MS Excel spreadsheet. For a given flood defence structure, values must be supplied for each of the parameters required by the relevant LSEs. A value may be fixed or specified as a statistical distribution with associated parameters. Using a Monte Carlo technique, random sample values are generated according to the specified distributions. For each sample, the fault tree is evaluated calling associated LSEs with the sample values. To generate fragility curves using RELIABLE, the hydraulic loading conditions are specified as fixed variables. These are then varied systematically, with a failure probability calculated for each value of loading considered, leading to the generation of fragility curves. Further information about RELIABLE is given in Appendix G.

7.4.1 Potential failure modes

The potential failure modes for Wentlooge and Caldicot defences are shown in Figure 7-1 and described below. Further information on the equations and assumptions associated with seepage and erosion failure is included in Appendix G.

External erosion

External erosion of the rear face of the embankment may be caused by flow of water over the crest and down the rear (dry) slope. If the flow velocities are high enough, grass cover may be damaged, leading to direct erosion of embankment materials. Damage is assumed to occur when the actual head difference or overflow discharge exceeds the critical limits for each failure mode

Seepage through levee

Seepage through levee is caused by the flow through levee material or through holes. This failure mode can take place in predominantly sandy or silty fluvial / estuarial levees, or pre-damaged clay surface layer. Flow can lead to piping (internal erosion) and bursting.

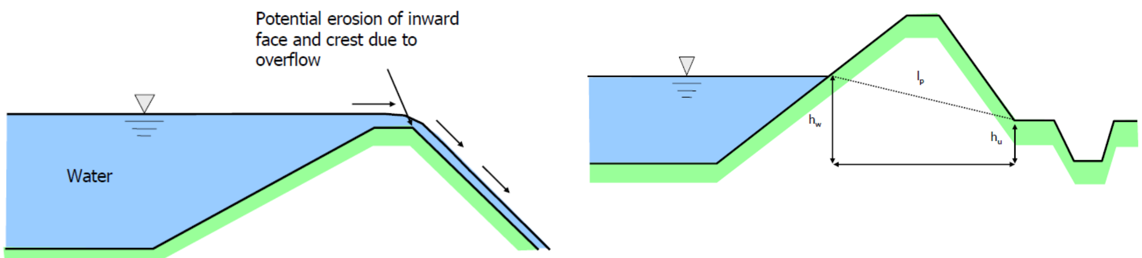


Figure 7-1: Potential failure modes of the Wentlooge and Caldicot defences, showing external erosion (left) and seepage through levee (right)[41]

7.5 Input data

A range of hydraulic, geotechnical and geometric data was needed to input into the RELIABLE model. This has been collated from the following sources:

- Hydraulic data: The external water level conditions for the 200+CC year return period accounting for sea level rise was used
- Geotechnical data: Typical soil parameters were included in the analysis and were obtained from appropriate literature.
- Geometric data: This was obtained from the data provided by Natural Resources Wales.

[41] FLOODsite (2007), Failure mechanisms for flood defence structures, FLOODsite Report T04-06-01, Task 4. www.FLOODsite.net

A schedule of input parameters is included in Appendix G.

7.6 Resulting fragility curves

In this section the fragility curves for the Wentlooge and Caldicot embankments are presented. The curves shown here are the individual curves that were obtained for each failure model described in Section **Error! Reference source not found.Error! Reference source not found.**.

7.6.1 Wentlooge defences

The Wentlooge defences (i.e. W3, W4, W5, W8 and W9) all fail due to piping under the 200+CC year return period. Defence W5 produced the highest breach peak outflow and defence 9 the lowest. The two were selected to represent the whole group.

7.6.2 Defence W5

The fragility curves obtained for defence W5 are presented in Figure 7-2. Run 1 is the base run with a base permeability value and full seepage length. Runs 2 and 3 are same as the base run but with low and high permeability values respectively. Runs 4 and 5 are same as the base but with a seepage length reduced by 25% and 50% respectively.

It can be seen for this defence that probability of failure is almost zero for the base run and Runs 2 and 4. For Runs 3 and 5 failure probabilities remain insignificant up to a water level of 8.5mAOD for Run 3 and 8.0mAOD for Run 5. After that probabilities reach 22% and 28% at the extreme water levels.

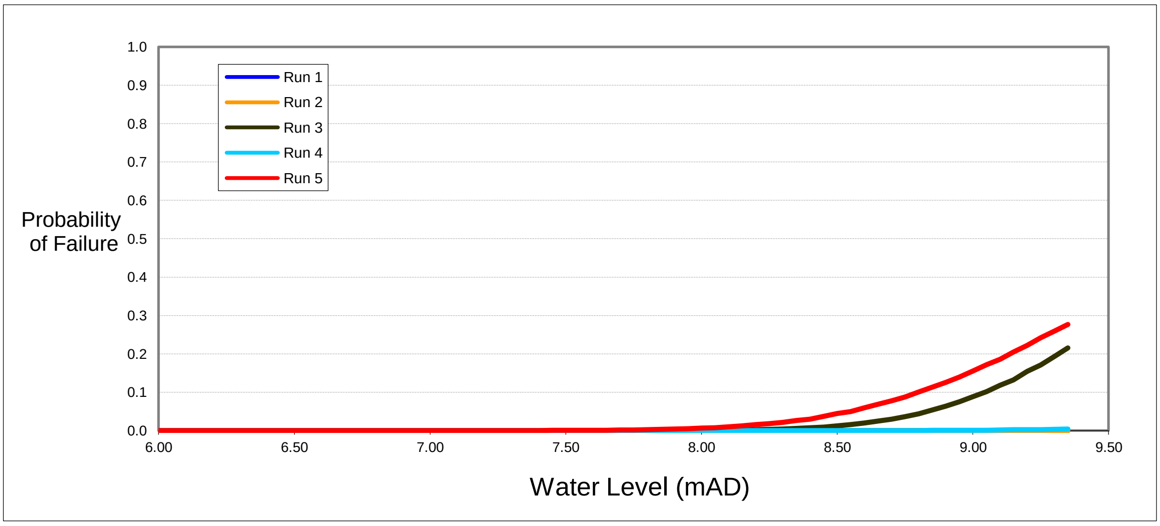


Figure 7-2: Defence W5 fragility curve.

7.6.3 Defence W9

The fragility curves obtained for defence W9 are presented in Figure 7-3. The run conditions are as described for Defence W5 in section 7.6.2. The probability of failure is almost zero for the base run and Runs 2, 3 and 4. For Run 5 failure probabilities remain insignificant up to a water level of 8.75mAOD. After that probabilities reach 13% at the extreme water levels.

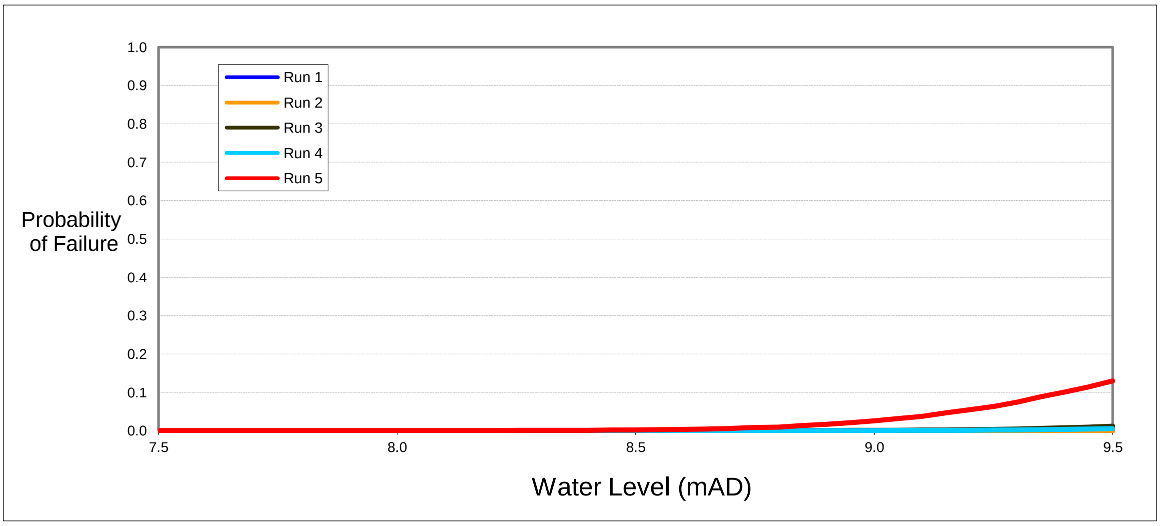


Figure 7-3: Defence W9 fragility curve.

7.6.4 Caldicot defences

In order to determine the reliability of the Caldicot defences which fail due to piping or overtopping, defence C19 was selected to represent those that fail by piping under the 200+CC year return period (i.e. C16, C17 and C20) as it gives the highest breach peak outflow. Defences C25 and C29 were selected to represent the defences that fail by overtopping under same return period.

7.6.5 Defence C19

The fragility curves obtained for defence C19 are presented in Figure 7-4. The run conditions are as described for defence W5 in section 7.6.2. It can be seen for this defence that probability of failure is almost zero for the base run and Runs 2 and 4. For Runs 3 and 5 failure probabilities remain insignificant up to a water level of 7.2mAOD for Run 3 and 8.0mAOD for Run 5. After that probabilities reach 88% and 22% at the extreme water levels. The higher probability for this defence is likely to be due to the short seepage length that it has compared to defences W5 and W9.

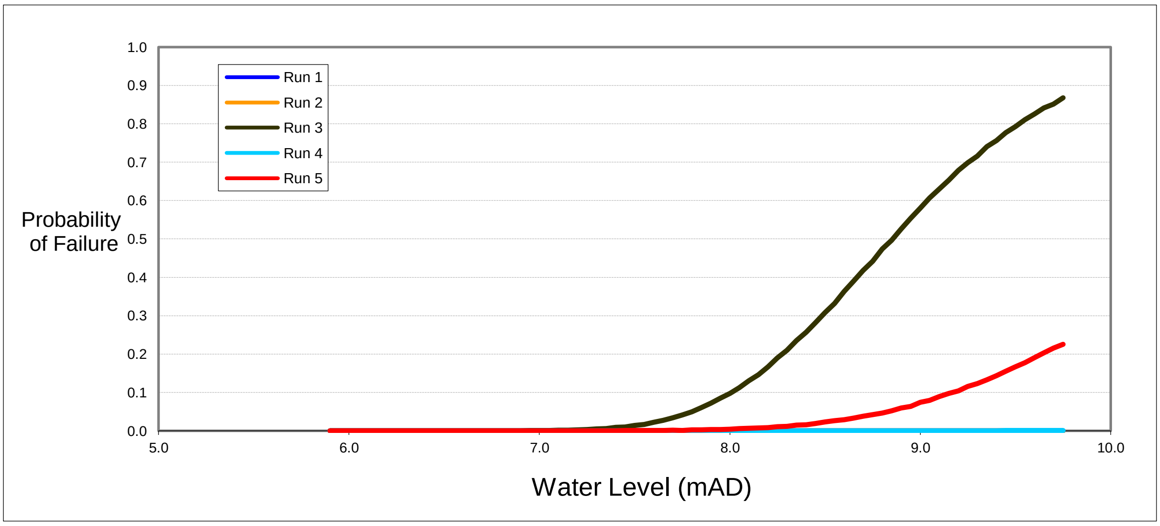


Figure 7-4: Defence C19 fragility curve.

7.6.6 Defence C25

The fragility curves obtained for defence D25 is presented in Figure 7-5. Run 1 is the base run with a medium quality downstream face protection. Runs 2 and 3 are same as the base run but with low and high quality downstream face protection respectively. It can be seen for this reach that probability of failure reaches 1.0 at water depth of 0.30m, 0.15m and 0.55m for Runs 1, 2 and 3 respectively. This shows the importance of keeping the protection on the downstream face at the highest possible quality as this would reduce the failure probability of the embankment.

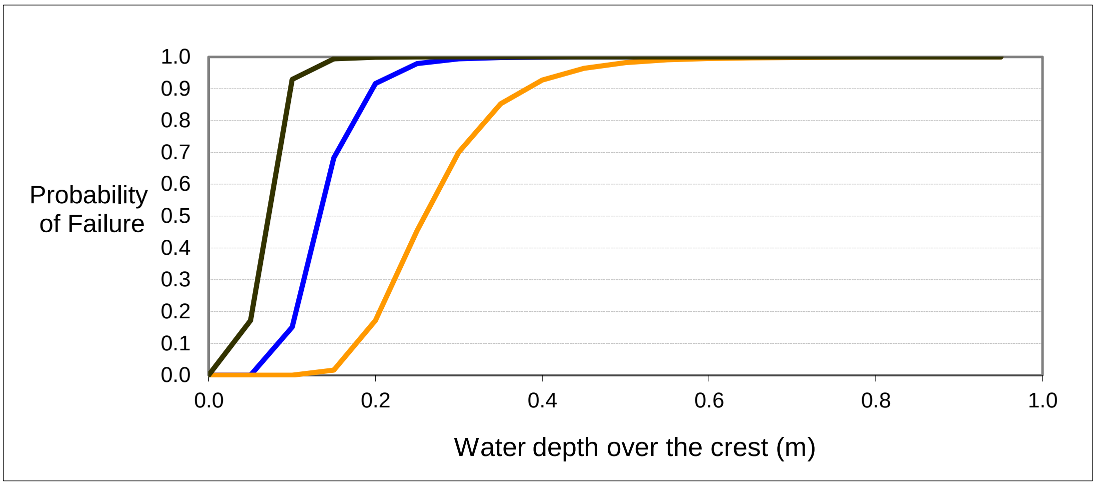


Figure 7-5: Defence D25 fragility curve.

7.6.7 Defence C29

The fragility curves obtained for defence C29 is presented in Figure 7-6. The run conditions are as described for defence C25 in section 7.6.6. Similar to defence 25, it can be seen for this reach that probability of failure reaches 1.0 at water depth of 0.30m, 0.15m and 0.65m for Runs 1, 2 and 3 respectively. This also shows the importance of keeping the protection on the downstream face at the highest possible quality as this would reduce the failure probability of the embankment.

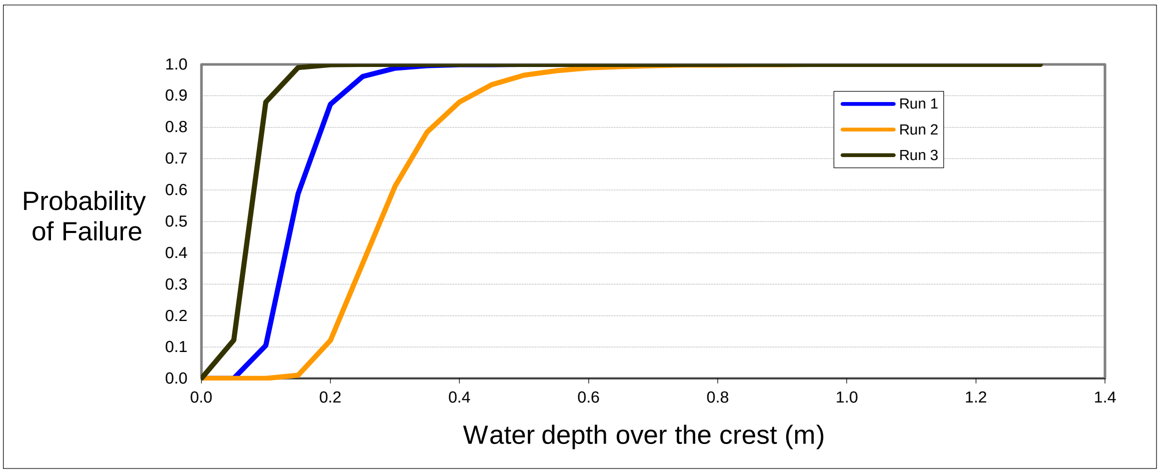


Figure 7-6: Defence C29 fragility curve.

7.7 Observations and conclusions

The following points may be concluded from the results of modelling work undertaken:

- Wentlooge and Caldicot defences are most at risk due to seepage through defence leading to piping, or external erosion of the rear face of the embankment.
- Failure probabilities for defences experiencing piping are insignificant unless the permeability inside the embankment reaches high values (i.e. in the range of $1e^{-3}$ m/s) or seepage length is reduced by 50% or more.
- Failure probabilities for defences being overtopped are more significant. An overtopping depth of 0.15m is likely to cause failure if the protection layer condition becomes at or below average.
- The risk of overtopping failure can be mitigated in Caldicot by ensuring downstream protection is maintained at the highest possible quality. This would reduce the failure probability even if the embankment is subjected to an overflow event.

8 Model proving

8.1 Introduction

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

8.2 Wave transformation model calibration

The SWAN wave model described in section **Error! Reference source not found.** was previously calibrated by Deltares[42]. This study verified the performance of the updated model using the new computational grid. This verification was undertaken by comparing the performance of the model against known events which occurred during the winter of 2013/2014. The model was run in stationary mode and offshore boundary conditions were varied to account for the time taken for the waves to travel to the site, calculated as approximately three hours. The results of the validation are presented in **Error! Reference source not found.** to **Error! Reference source not found.**[43]. Sensitivity of the model outputs to variation of the wave spreading parameter was observed as being minimal.

The Root Mean Square Error (RMSE) of the wave height across the four events is 8% at the Second Severn Crossing and 14% (approx. 8% excluding the result of verification 1, which may be attributed to erroneous input data) at the Scarweather Buoy.

The RMSE of the wave period ranged between 11% and 41%. This poor performance was attributed to the frequency 'bin' discretisation of the wave model. As a check, the mean period predicted by the model can be approximately related to the peak period predicted by the model (given a Jonswap spectrum with gamma of 3.3) by assuming the peak period is 1.29 times (Reeve, Chadwick and Flemming, Spon Press 2004) the mean period (T_{m-01}). In the case of the verification results shown, generally this yields a peak period (unconstrained to the central value of a particular frequency bin) closer to the observed value.

Table 8-1: Verification 1 (27 Dec 2014 0600hrs) model inputs, outputs and percentage difference

Parameter	Hs	Tp	Dirn
Boundary Condition:			
Input	5.70 m	11.24 s	250.00°
Second Severn Crossing:			
Observed	0.75 m	5.8 s	-
Predicted	0.73 m	3.42 s	219.00°
Diff	3 %	41 %	-
Scarweather:			
Observed	5.06	13.4 s	258.80
Modelled	3.92	11.75 s	244.90
Diff	29 %	14 %	13.90°

Table 8-2: Verification 2 (03 January 2014 2100hrs) model inputs, outputs and percentage difference

Parameter	Hs	Tp	Dirn
Boundary Condition:			
Input	8.26 m	15.83 s	256.90°
Second Severn Crossing:			
Observed	0.75 m	4.60 s	-
Predicted	0.84 m	4.14 s	229.64°
Diff	11 %	11%	-
Scarweather:			
Observed	4.72 m	10.50 s	253.10°
Modelled	4.14 m	15.61 s	245.96°
Diff	14 %	33 %	7.14°

[42] NRW (2011). Wales Coastal Flood Forecasting Model. Deltares.

[43] For consistency, the various physical constants were not altered and hence a less than perfect agreement was accepted for this verification exercise. The value of spreading was not provided in the original offshore data, and a representative value of 30 degrees was used in all cases. Sensitivity of the model outputs to variation of this parameter was observed as being minimal.

Table 8-3: Verification 3 (08 Feb 2014 1900hrs) model inputs, outputs and percentage difference

Parameter	Hs	Tp	Dirn
Boundary Condition:			
Input	8.39 m	17.54 s	255.20°
Second Severn Crossing:			
Observed	1.10 m	5.30 s	-
Predicted	1.00 m	4.14 s	214.99°
Diff	10 %	28 %	-
Scarweather:			
Observed	5.06 m	10.10 s	258.80°
Modelled	5.51 m	17.16 s	243.14°
Diff	8 %	41 %	15.66°

Table 8-4: Verification 4 (15 February 2014 0200hrs) model inputs, outputs and percentage difference. In this case, the tide level is that of the next hours' time step.

Parameter	Hs	Tp	Dirn
Boundary Condition:			
Input	6.73 m	14.71 s	243.40°
Second Severn Crossing:			
Observed	1.31 m	5.80 s	-
Predicted	1.22 m	4.56 s	221.85°
Diff	8 %	27 %	-
Scarweather:			
Observed	4.89 m	11.20 s	241.90°
Modelled	4.97 m	14.20 s	243.57°
Diff	1 %	21 %	1.67°

8.3 Wave overtopping model validation

There were no available data to quantitatively validate the wave overtopping calculations. Instead the validation was performed through an analysis of the frequency of the wave overtopping estimates when calculated using available hindcast model data.

8.3.1 Frequency analysis of wave overtopping

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

For this study, the performance of the wave overtopping models was evaluated using a long-term performance method to calculate a continuous record of overtopping events. The frequency of modelled overtopping was then compared to available records between 1980-2014 to ensure the models do not over- or under-predict the magnitude or frequency of observed wave overtopping.

Observed frequency of overtopping

Based on the historical review presented in Section **Error! Reference source not found.**, there are relatively few records of flooding along the coast. Information on the key events since 1980 are documented in Table 8-5. The records indicate past events to be predominantly tidal flooding rather than wave overtopping. Whilst an online search failed to return any additional results, wave overtopping is not unknown in the area with a video in 1998 recording spray. It is more likely that due to the sparsly populated coastal frontage, there may not be many people around to record the occurrence of wave overtopping during stormy conditions.

Based on this analysis it is expected that large overtopping rates (e.g. over 50 l/s/m) will have a very low annual frequency, moderate events (e.g. 10 l/s/m) may occur several years apart, and only low events (0.1 to 1 l/s/m) would occur annually.

Table 8-5: Key flood events between 1980 and 2014

Date	Mechanism	Notes
13/12/1981	Tidal flooding	Estimated 8.4mAOD at Newport Docks, flooding on the west bank of the Usk with 480 properties affected. East bank of the Usk upstream of the railway bridge and Bell Ferries downstream of the transporter bridge affected
1997	Tidal flooding	Tidal flooding in Chepstow
01/01/1998	Wave overtopping	Video of spray overtopping at Goldcliffe
30/10/2000	Flooding from reens	Flooding from reens occurred in Duffryn and Liswerry
03/01/2014	Large tide and surge	8.03mAOD at Newport, flooding in Tintern, Goldcliffe, Crindau and Caerleon on the Usk. Also evidence of debris over the defences along the defence at Mathern

Long-term overtopping model assessment

The long-term test was undertaken by calculating daily peak overtopping rates for a 34-year period spanning from 1980-2014. The input data for this analysis included:

- Hindcast model data from the Met Office WWIII model were used to provide time-series of wave heights, wave periods, wave directions and wind speed and direction.
- Hindcast surge model data from the National Oceanography Centre (NOC) CS3X surge model was stitched together with surge data from the European 2G wave model (1990-2000) and the UK Waters Wave model (2000-2008) to provide a continuous time-series of surge predictions from 1990-2014.
- Astronomical tidal predictions and observed sea-levels from the Newport Class A tide gauge were extracted for the same period as the surge data and then combined with the surge model data to produced continuous time-series of total sea-levels from 1993 to 2014.
- Using the time-series of sea-level, wind and wave data and the wave model emulators generated for the Monte Carlo modelling, nearshore wave data was calculated to provide the inputs for the wave overtopping models.

Using the nearshore data, a continuous time-series of wave overtopping discharges was calculated at each of the 32 schematised defences. The resulting overtopping frequency was analysed in relation to the targets described in section **Error! Reference source not found.**. Small changes were then made to the schematisations to 'calibrate' their long-term performance where needed.

Table 8-6 summarises the final overtopping frequency for defences modelled to overtop at least once during the 34-year timeframe. For all other profiles, no overtopping was predicted. Each defence was analysed against a number of overtopping thresholds based on published tolerable discharges[44] for the safety of unaware pedestrians (0.03L/s/m - considered very minor), aware pedestrians (0.1L/s/m - considered minor), and a range of discharges suitable for trained staff (between 1 and 10L/s/m - considered moderate).

The results show 17 of the profiles did not overtop during the hindcast period, 13 experienced very minor overtopping less than once per year, one experienced minor overtopping almost annually, and only one profile (Ref 23) experienced overtopping rates of over 10L/s/m.

[44] EurOtop Manual: Pullen, T., Allsop, N.W.H., Bruce, T., Kortenhaus, A., Schüttrumpf, H., van der Meer, J.W. (2007): EurOtop - Wave Overtopping of Sea Defences and Related Structures: Assessment Manual. http://www.overtopping-manual.com/manual.html

Table 8-6: Number of overtopping discharges that exceed published EurOtop thresholds for pedestrians and trained staff

Profile number	Number of overtopping events (and annual frequency)				
	0.03 l/s/m	0.1 l/s/m	1 l/s/m	2 l/s/m	10 l/s/m
1	1 (0.05)	0 (0)	0 (0)	0 (0)	0 (0)
5	7 (0.33)	5 (0.24)	0 (0)	0 (0)	0 (0)
6	9 (0.43)	6 (0.28)	1 (0.05)	0 (0)	0 (0)
7	2 (0.10)	1 (0.05)	0 (0)	0 (0)	0 (0)
9	3 (0.14)	2 (0.10)	1 (0.05)	0 (0)	0 (0)
12	16 (0.76)	3 (0.14)	0 (0)	0 (0)	0 (0)
14	13 (0.62)	1 (0.05)	0 (0)	0 (0)	0 (0)
15	**78 (3.69)**	**24 (1.14)**	1 (0.05)	0 (0)	0 (0)
16	9 (0.43)	3 (0.14)	0 (0)	0 (0)	0 (0)
17	13 (0.62)	4 (0.19)	2 (0.10)	1 (0.05)	0 (0)
18	3 (0.14)	0 (0)	0 (0)	0 (0)	0 (0)
19	3 (0.14)	0 (0)	0 (0)	0 (0)	0 (0)
22	16 (0.76)	14 (0.66)	8 (0.38)	4 (0.19)	0 (0)
23	**80 (3.78)**	**51 (2.41)**	16 (0.76)	10 (0.47)	6 (0.28)
31	10 (0.47)	9 (0.43)	6 (0.28)	5 (0.24)	0 (0)

Notes:
Number relates to the number of exceedances over the 21-year period and the annual frequency is the average number of exceedances per year over the same period. Bold values indicate annual frequency over 1 per year.

0.03 l/s/m = EurOtop threshold for unaware pedestrians
0.1 l/s/m = EurOtop threshold for aware pedestrians
1 l/s/m = EurOtop lower threshold for trained staff
2 l/s/m = arbitrary mid threshold for trained staff
10 l/s/m = EurOtop upper threshold for trained staff

Discussion of final overtopping model

Due to the difficulty in collecting accurate wave overtopping information (e.g. the volume overtopped during a storm), the testing of overtopping models through long-term performance testing offers the most pragmatic way to validate their performance. When simulated over many years the performance of overtopping models can be compared to years of anecdotal information.

The hindcast results indicate many defences where wave overtopping is not expected to occur. There were many occasions when very minor amounts of wave overtopping would be expected - likely to be observed as spray. There were very few defences where moderate wave overtopping was predicted, e.g. between 1-10L/s/m. The overtopping modelling hindcast results suggest that for most locations wave overtopping is predicted to occur less than once per year. Profile 15 near Goldcliffe and Profile 23 near West Pill predicted greater rates of wave overtopping, with almost four events per year exceeding the safe limit for unaware pedestrians. One of these events corresponded to the recorded wave overtopping video from Goldcliffe in 1998. These results are considered to adequately reflect the observed conditions, which suggested only low rates of overtopping would occur annually.

8.4 Flood inundation model calibration

In addition to the very few records of flooding, there are no recorded historic flood outlines to compare wave overtopping inundation extents. Due to the flood inundation model set-up, which establish lateral boundaries along the line of the tidal rivers, the 1D ISIS and HEC-RAS models used to provide the boundaries were calibrated but the 2D boundaries along the tidal river were fixed, and calibration of this model was not possible. Instead, sensibility and sensitivity testing was completed.

8.5 Sensitivity testing

The flood inundation models were run through a series of sensitivity tests to help understand the uncertainty in the model set-up and boundary conditions. The sensitivity tests performed were a test on the model roughness by increasing and decreasing the design modelled roughness values by ±20% and a test on the representation of buildings by modelling as stubby buildings raised 0.3m above ground level to compare against the results of the buildings represented through increased roughness. All sensitivity tests were completed using the 1 in 200-year event.

8.5.1 Sensitivity to model roughness

The resulting flood inundation for the three model simulations (base case, plus and minus 20% roughness) have been colour coded and are shown in Figure 8-1 and Figure 8-2 for Wentlooge and Caldicott respectively.

Overall there was very little change to the flood extents when an increase or decrease to the model roughness values were applied in the Wentlooge TUFLOW model. The area with the most noticeable change in extent in the Wentlooge model was in Newport where depths of flooding changed by ±0.01m with an increase in roughness resulting in a slightly smaller flooded area and a decrease in roughness increasing the flood extents. For the Caldicot model the most noticeable differences were around Newport and Liswerry with increased flood depths of 0.08m and more extensive flooding for the reduced roughness simulation. Along the open coast there were no noticeable differences in the flood extents.

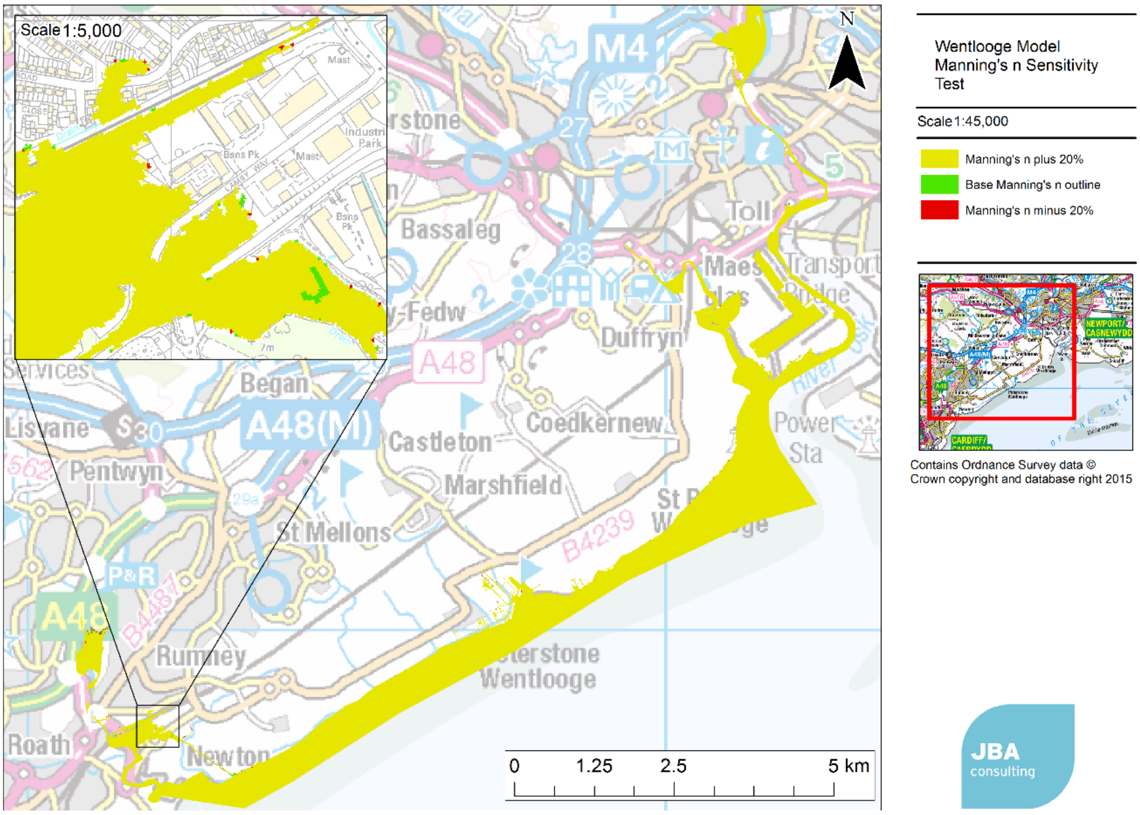


Figure 8-1: Wentlooge modelled water levels resulting from three sensitivity tests: A base case, plus and minus 20% roughness values (represented as Mannings n vales).

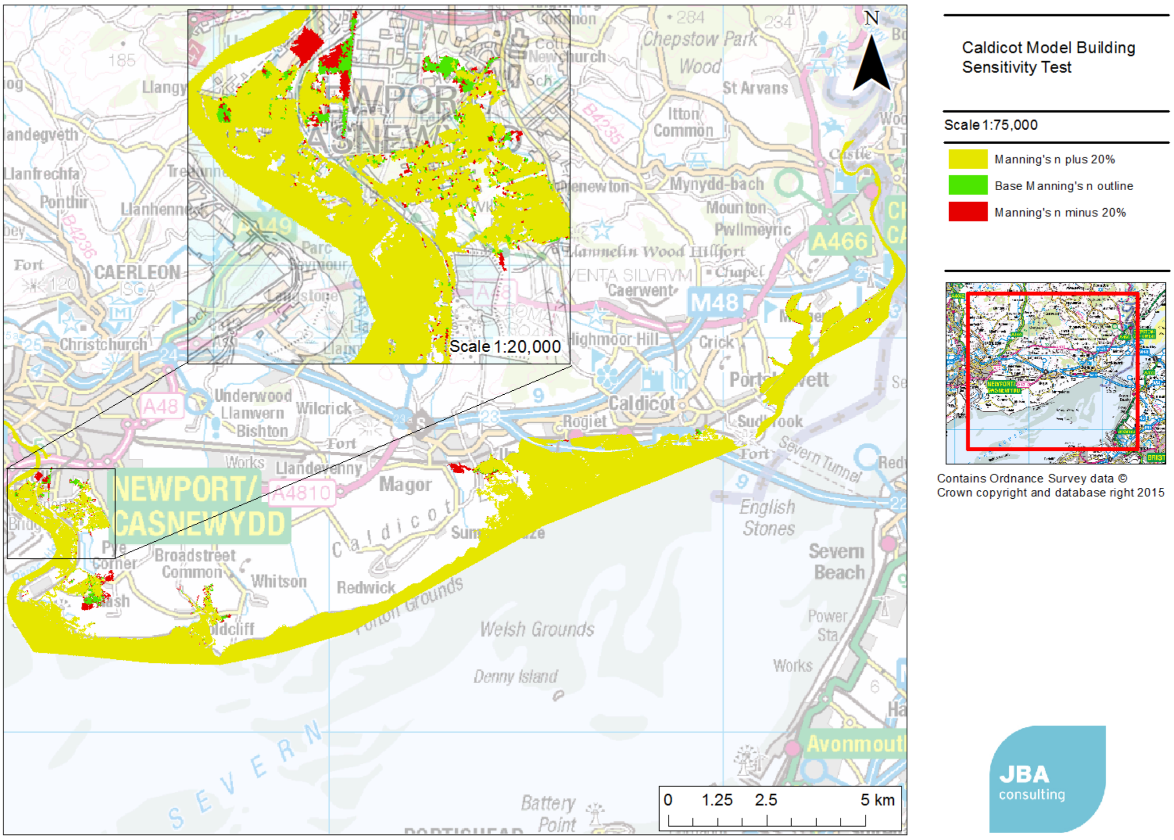


Figure 8-2: Caldicot modelled water levels resulting from three sensitivity tests: A base case, plus and minus 20% roughness values (represented as Mannings n vales).

8.5.2 Sensitivity to representation of buildings

In the design flood model, the restricted flow through buildings was represented by increasing the roughness within a building footprint. An alternative simulation using 'stubby buildings' was tested to understand the sensitivity of this approach. Building footprints were raised by 0.3m to represent building foundations, with the raised land then assigned the increased roughness value to represent the restricted flow through the building, if flooded.

The resulting flood inundation for the two model simulations (base case and 'stubby buildings') have been colour coded and are shown in Figure 8-3 and Figure 8-4 for Wentlooge and Caldicot respectively. The results for the Wentlooge tests show an almost identical flood extent, with minor differences only observed to the south of Rumney. For the Caldicot model the sensitivity to building representation resulted in changes to the flood depths of ±0.05m and a smaller flood outline, noticeable to the south of Newport. The base model flood extents are more conservative as the stubby building approach blocks some of the flow paths where the flood depths are shallower than the 0.3m increased building footprint. In the lower resolution Caldicot model, the 10 grid resolution results in some of the buildings being merged and unrealistically blocking flow paths between buildings. The final design modelling approach use the increased roughness over the building footprint, not the stubby buildings.

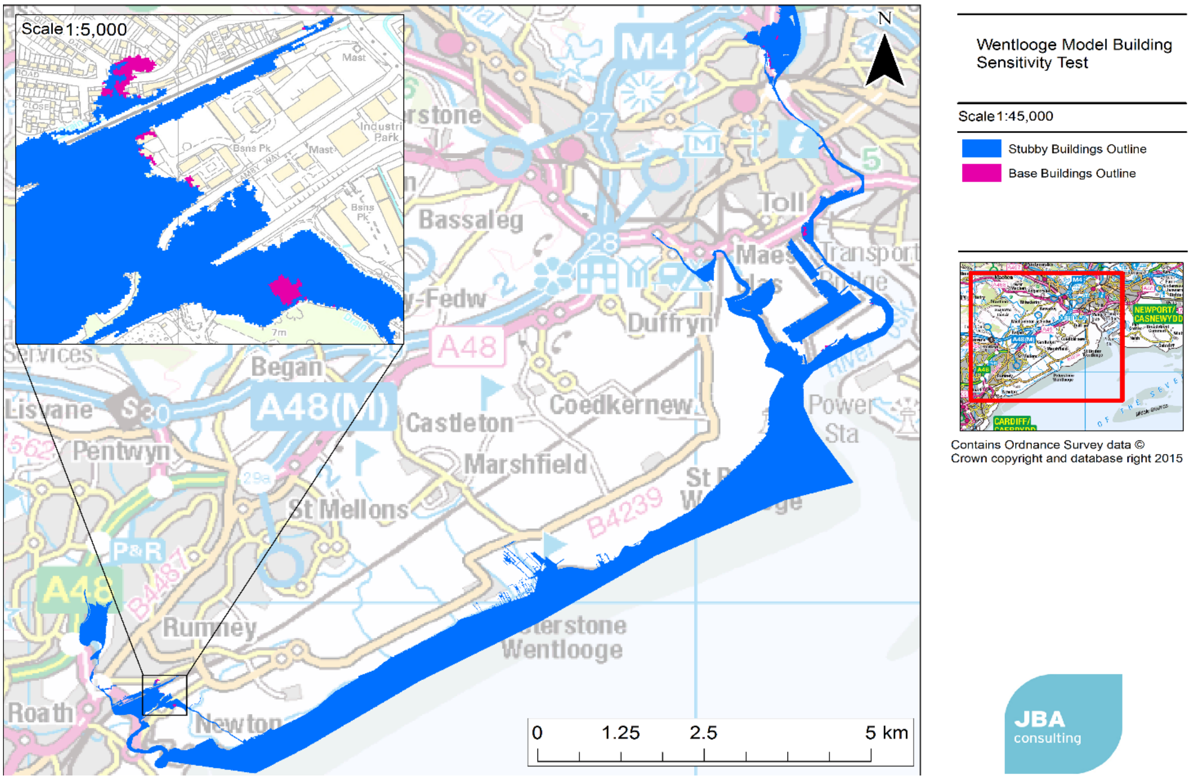


Figure 8-3: Wentlooge modelled water levels resulting from two sensitivity tests (roughness patches and stubby buildings)

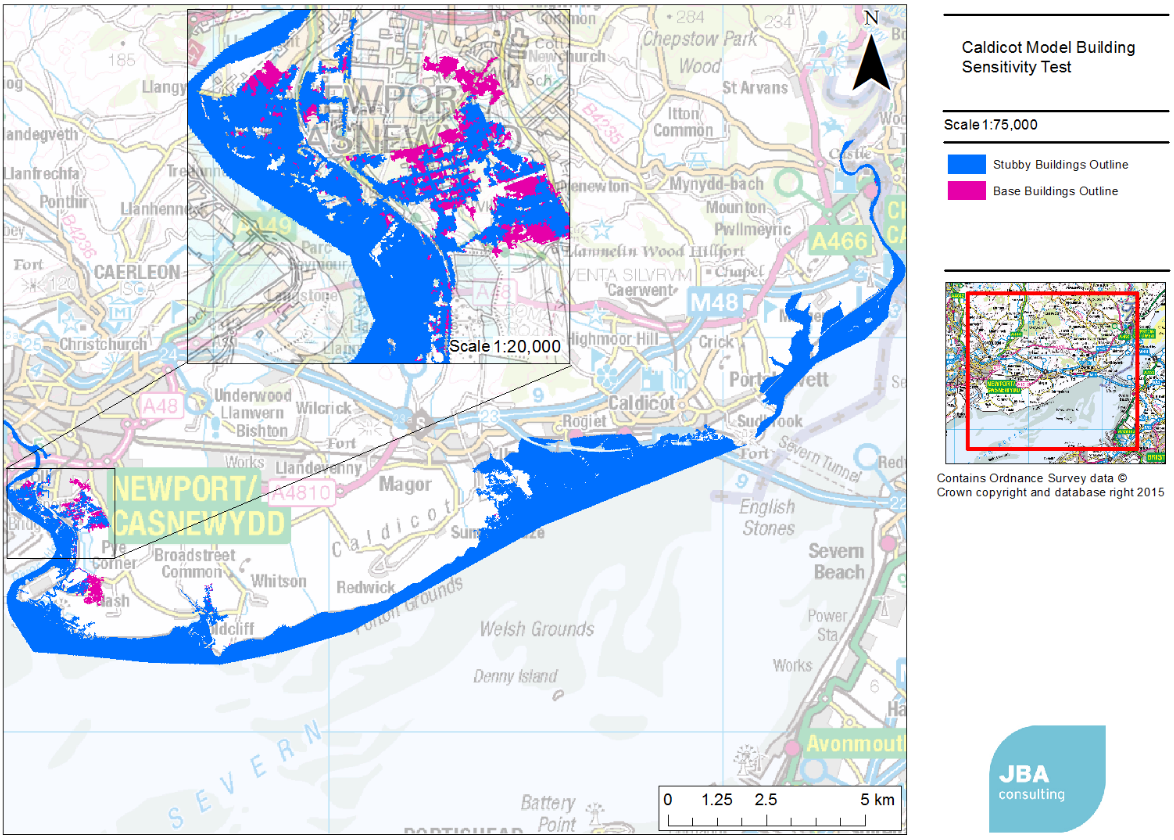


Figure 8-4: Caldicot and Wentlooge modelled water levels resulting from two sensitivity tests (roughness patches and stubby buildings)

9 Results

9.1 Introduction

This chapter discusses the model results and summarises the flood risk in the main urban areas. The study was separated into two models and as such this chapter is divided into two main sections, Wentlooge and Caldicot.

9.2 Modelled scenarios

A large number of model simulations were completed to quantify the flood risk for a range of scenarios. These included still water scenarios "With Defences" but no wave overtopping, with and without defence scenarios, breach modelling and tidal outfall failure modelling. A full list of the modelled scenarios is included in Table 9-1.

Table 9-1: Scenarios and events required for the study

Description of scenario	Event	Total number of TUFLOW model runs
Tidal with defences, no wave overtopping	5	2 (1 per model domain - Caldicot and Wentlooge)
	20	2
	50	2
	200	2
	1,000	2
Tidal with defences and wave overtopping	5	2 (1 per model domain - Caldicot and Wentlooge)
	20	2
	50	2
	200	2
	200 CI	2
	200 CC	2
	200 CC CI	2
	1,000	2
	1,000 CI	2
	1,000 CC	2
	1,000 CC CI	2
	5,500	2
Tidal no defences (outfalls assumed open, no pumping stations)	200	2 (1 per model domain - Caldicot and Wentlooge)
	200 CC CI	2
	1,000	2
	1,000 CC CI	2
Breach (5 Wentlooge, 12 Caldicot)	5 - 1,000 CC	13 (some runs contain multiple breaches)
Tidal gate failure - open scenario - no fluvial flow applied behind outfall, worst case for tidal	100 (tidal), 100 (fluvial)	15 (1 per outfall)
Tidal gate failure - closed scenario - fluvial flow applied behind outfall but no drainage back to sea, worst case for fluvial	100 (tidal), 100 (fluvial)	15 (1 per outfall)
Notes: *CC = Climate Change, CI = Confidence Interval		

9.3 Wentlooge model

The Wentlooge Levels model covers an area extending from Cardiff to Newport, bounded by the River Rhymney to the west and River Usk to the east.

9.3.1 With defences no wave overtopping

With the defences in place the flood risk on the Wentlooge Levels is relatively limited. Figure 9-1 summarises the flood risk from still water flooding for the 200 and 1,000-year return period events. The flooding in these scenarios is limited to the east bank of the River Rhymney at Rumney, affecting the Eastgate Business Park and the west bank of the River Usk in Newport affecting the Power Station and a significant number of properties to the north of the transporter bridge. Along the coastal frontage there is flooding of the low lying land around Peterstone Gout.

Figure 9-1: With defences flood risk for the Wentlooge Levels, 200 and 1,000-year extents with no wave overtopping

9.3.2 With defences and wave overtopping

When wave overtopping inflows are added in addition to the still water boundaries, the flood risk remains the same on the east and west banks of the Rivers Usk and Rhymney, as these are sheltered from the effects of wave overtopping. The flood risk on the open coast increases, particularly around Peterstone Gout and to the west towards Lighthouse Park. The flood extents are shown in Figure 9-2.

Figure 9-2: With defences flood risk for the Wentlooge Levels, 200 and 1,000-year extents, with wave overtopping

9.3.3 No defences

When the defences were removed the flood risk is greatly increased. All of the low lying land within the Wentlooge Levels are inundated. The flood extents are topographically controlled and where the ground levels start to rise on the edge of the floodplain, there is a steep increase in elevation and the flood extents from the 200 and 1,000-year outlines are very similar. The flood extents encompass the whole of the Eastgate Business Park, the Wentloog Corporate Industrial Park, properties in Rumney, Trowbridge, St Mellons Business Park, Marshfield, Tredegar Park at Duffryn, Lighthouse Park and all other properties on the low lying Levels. The flood extents for the no defence scenarios are shown in Figure 9-3.

Figure 9-3: No defences flood risk for the Wentlooge Levels, 200 and 1,000-year extents

9.3.4 Modelling the uncertainty in the extreme sea-levels

The extreme sea-levels included in the CFB data (and revised CFB calculated for this study) contain uncertainty. To understand the impact of this uncertainty on the modelling, the upper confidence limits were added to the extreme sea-levels modelled for this study. The tidal-graph was then regenerated using the same process detailed in section 4.4.1. The flood inundation models were used to simulate the increased flood risk associated with the uncertainty in the sea-level estimates. The 200-year return period simulation was run with 0.2m added to the tidal boundaries and the 1,000-year return period simulations were completed with 0.5m added to the tidal boundaries. When the confidence intervals are added the flood risk increases, showing additional properties at risk in Marshfield, Duffryn and Pillgwenlly. The flood risk extents and properties at risk are summarised in Figure 9-4 and Table 9-2.

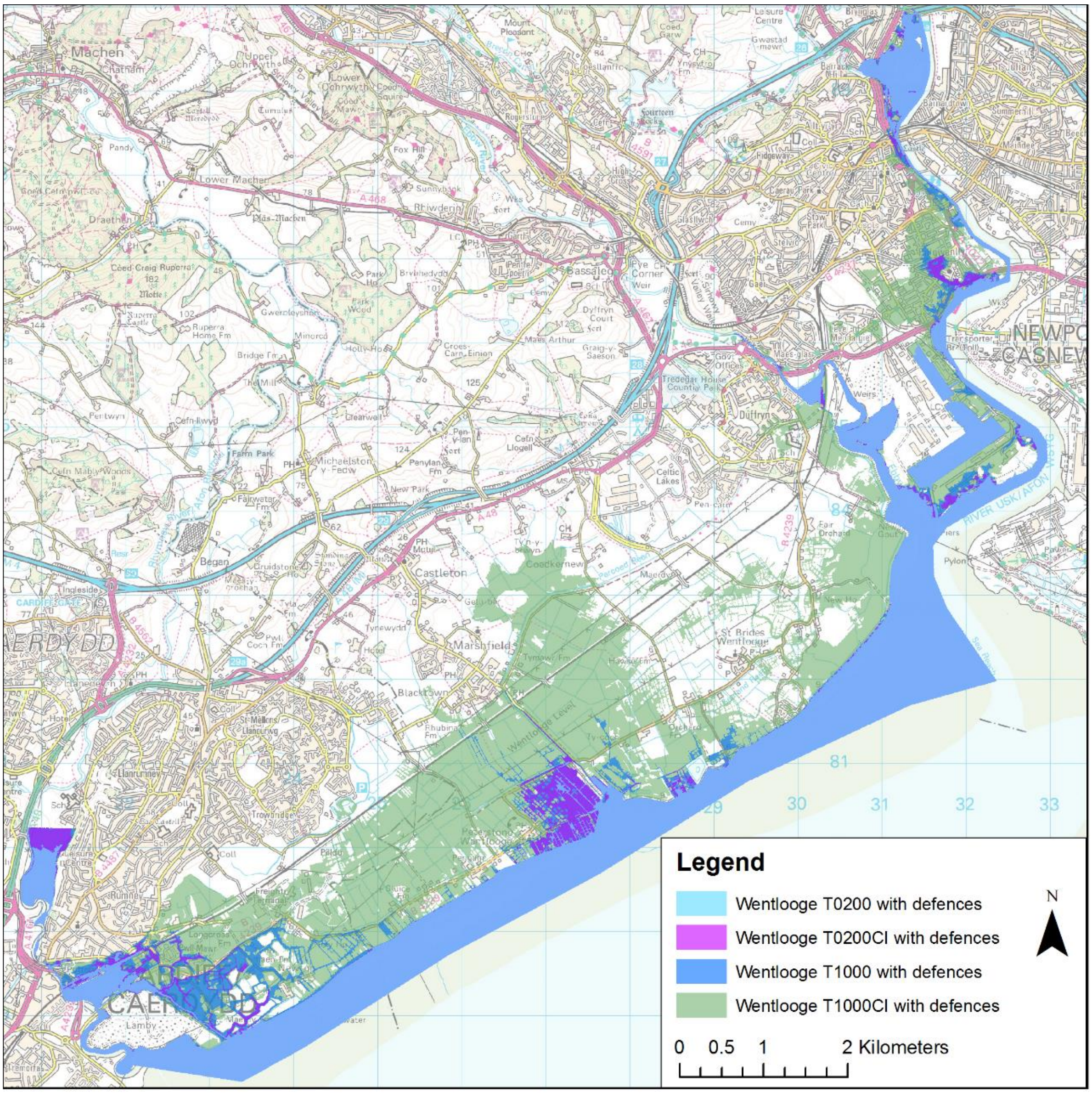


Figure 9-4: Wentlooge flood risk with confidence intervals

9.3.5 Property counts for the Wentlooge model scenarios

The 2014 National Receptor Data (NRD) was used to calculate the number of properties within each of the modelled flood extents. The NRD data was filtered to remove properties that were marked for exclusion from the NaFRA property counts, as detailed in Appendix D of the Geomatics Reconciliation Report. The results of the property counts for the Wentlooge "With Defences", "With Defences No Wave Overtopping" and "No Defence" scenarios are detailed in Table 9-2.

Table 9-2: Wentlooge property counts

Model scenario	Residential properties (number)	Commercial properties (number)	Unclassified (number)
With Defences model scenarios			
T0005 WD	267	81	0
T0020 WD	432	142	0
T0050 WD	543	150	0
T0200 WD	755	171	0
T1000 WD	1,360	410	0
T5500 WD	2,618	824	0
T0200 CI WD	955	261	0
T1000 CI WD	3,765	1,091	0

Model scenario	Residential properties (number)	Commercial properties (number)	Unclassified (number)
With Defences but not wave overtopping model scenario			
T0005 WD NWO	267	80	0
T0020 WD NWO	432	141	0
T0050 WD NWO	542	149	0
T0200 WD NWO	754	170	0
T1000 WD NWO	1,356	409	0
No Defences model scenarios			
T0200 ND	5,735	641	0
T1000 ND	7,127	895	0

9.3.6 Impact of climate change

The potential impacts of climate change were modelled within the wave transformation, wave overtopping and flood inundation models to account for the predicted rise in sea-levels, increasing wind speeds and increased wave heights. Increasing sea-levels not only increase the flood risk from still water levels exceeding defence crests, but the deeper water also allows larger waves to propagate further inshore and increase the risk of wave overtopping. To simulate these events, the tidal-graphs in models built for present day conditions were modified, with 1.06m added to the sea-levels and the winds increased 10% in the wave transformation model.

With Defences simulations

The flood inundation models were used to simulate the increased flood risk for the 200 and 1,000-year return period events. When the impact of climate change was applied the flood risk increases. The flood extent is similar to the No Defences scenario, and the flood extents for 200-year CC and 1,000-year CC outlines are similar. The flood extents encompass the Eastgate Business Park, properties in Rumney, Trowbridge, Marshfield and Duffryn, the Wentlooge Corporate Business Park and all of the properties located on the low-lying land of the Wentlooge Levels. The flood extents for the climate change scenarios are shown in Figure 9-5.

Further simulations were carried out to assess the impact of climate change with the uncertainty in the sea-level estimates applied to the calculated levels. When the confidence intervals are added to the climate change runs the flood risk increases, showing additional properties at risk in St Mellons, Marshfield and Duffryn (Figure 9-6).

No Defences simulations

Simulations were carried out to assess the impact of climate change with the uncertainty in the sea-level estimates for the No Defences scenario. When these values were applied to the No Defences simulations, the flood risk for the Wentlooge Levels increased. Additional properties are inundated in St Mellons, Marshfield and Newport, in particular the Maesglas Industrial Park, to the south of St Woolos and around the Alexandra Docks. The flood risk extents and properties at risk are summarised in Figure 9-4 and Table 9-3.

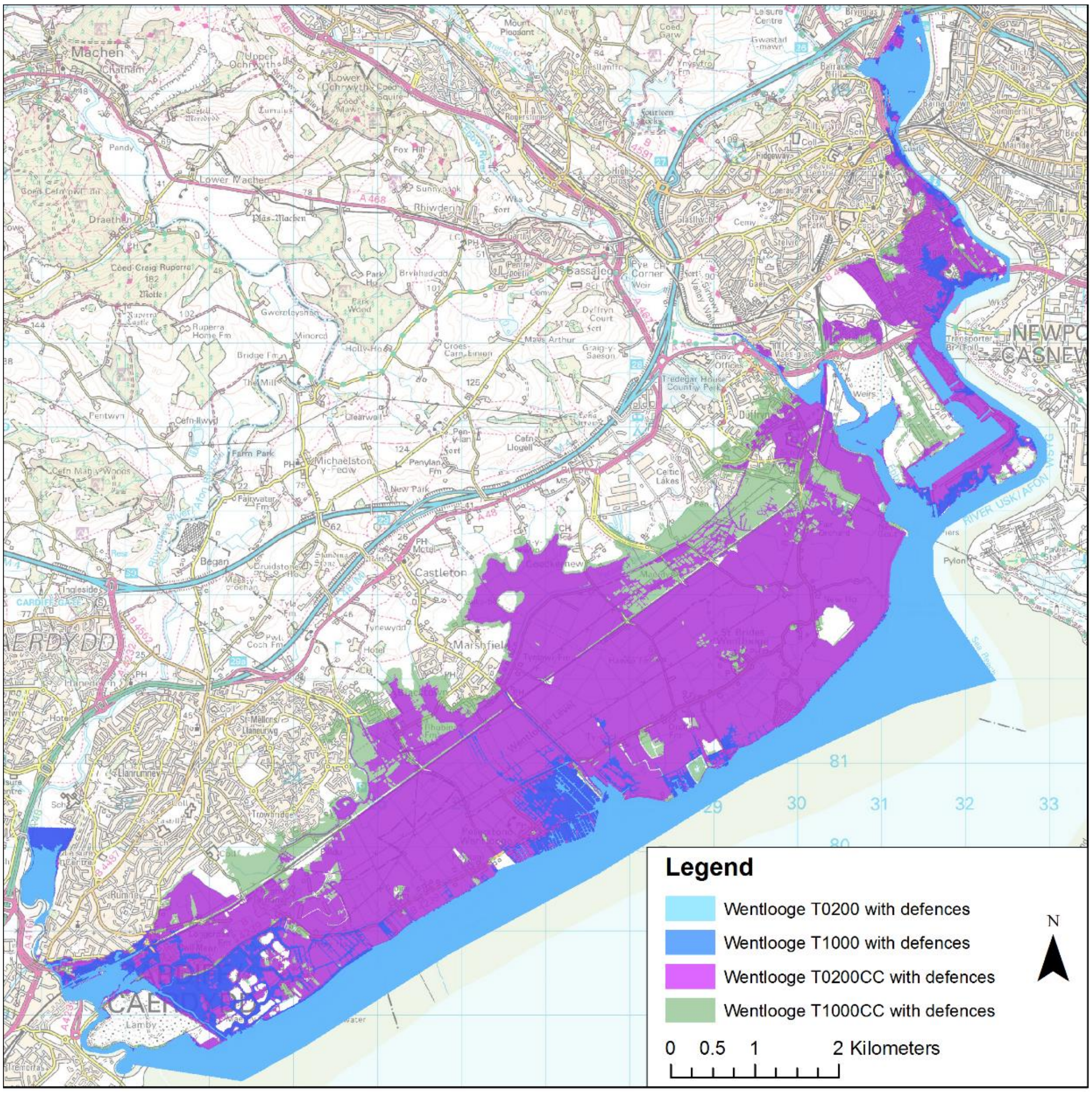


Figure 9-5: Wentlooge model climate change flood extents

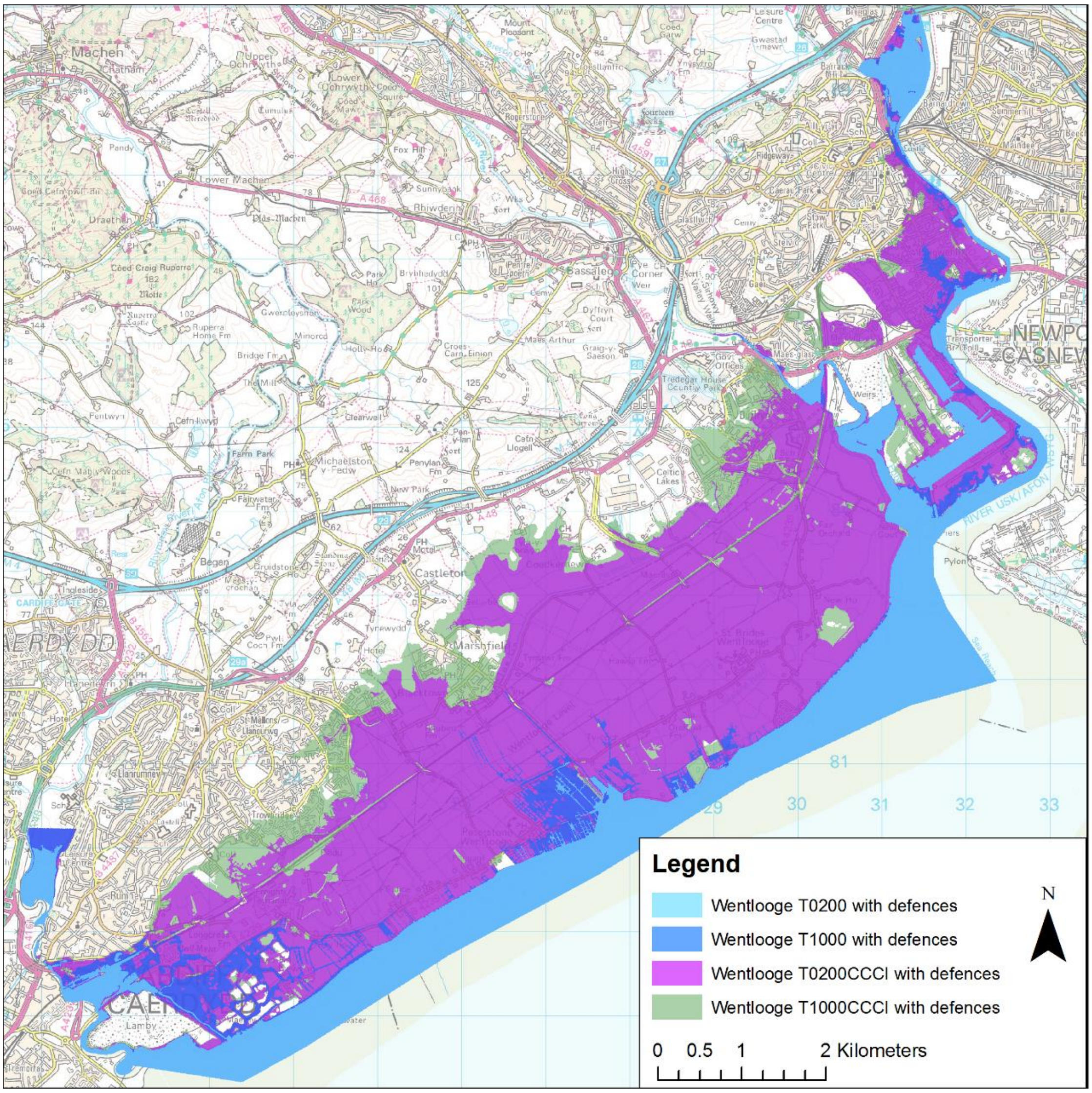


Figure 9-6: Wentlooge model climate change flood extents with confidence intervals

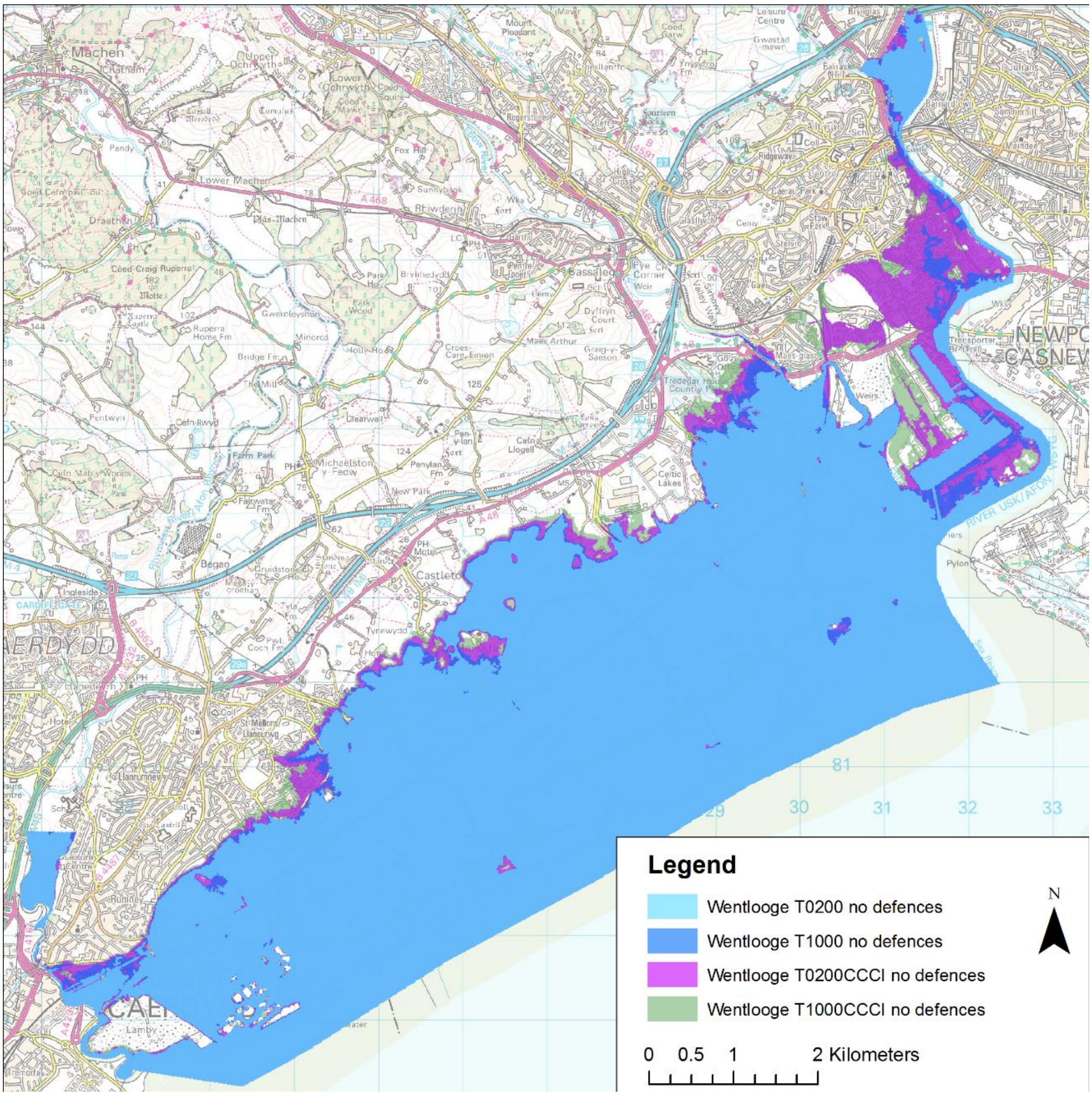


Figure 9-7: No defence flood risk extent for the Wentlooge model for climate change with confidence intervals

Table 9-3: Wentlooge climate change property counts

Model scenario	Residential properties (number)	Commercial properties (number)	Unclassified (number)
With Defences model scenarios			
T0200 CC WD	4,858	1,644	0
T1000 CC WD	6,135	1,798	0
T0200 CC CI WD	5,612	1,738	0
T1000 CC CI WD	9,917	2,082	0
No Defences model scenarios			
T0200 CC CI ND	10,973	1,926	0
T1000 CC CI ND	11,870	2,146	0

9.4 Caldicot model

The Caldicot Levels model covers an area extending from Newport to Chepstow, bounded by the River Usk to the west and River Wye to the east.

9.4.1 With defences no wave overtopping

The Caldicot Levels are not defended to the same standard of protection as the Wentlooge Levels, which increases the flood risk for all modelled events. With the defences in place there is a risk of flooding from the River Usk in Newport, and along the coastal frontage at defence low spots near Goldcliff Pill, West Pill, Caldicot Pill, Mathern Pill and St Pierre Pill. Figure 9-8 summarises the flood risk from still water flooding for the 200 and 1,000-year return period events. The flooding in these scenarios is limited to the east bank of the River Usk between the A48 road crossing and the Uskmouth Power Station, affecting the Power Station and many residential and commercial properties in Liswerry. Along the coastal frontage the flooding from the Usk joins the flooding from a low section of defence near the Goldcliff Pill, during the 1,000-year event. This affects properties in Goldcliff and Whitson. Close to Caldicot, there is significant inundation of the Levels affecting the M4 motorway, the Pill Farm Industrial Estate and scattered properties on the Bridewell and Undy Commons. Further east towards Chepstow, the railway and Newhouse Farm Industrial Estate are shown to be at flood risk.

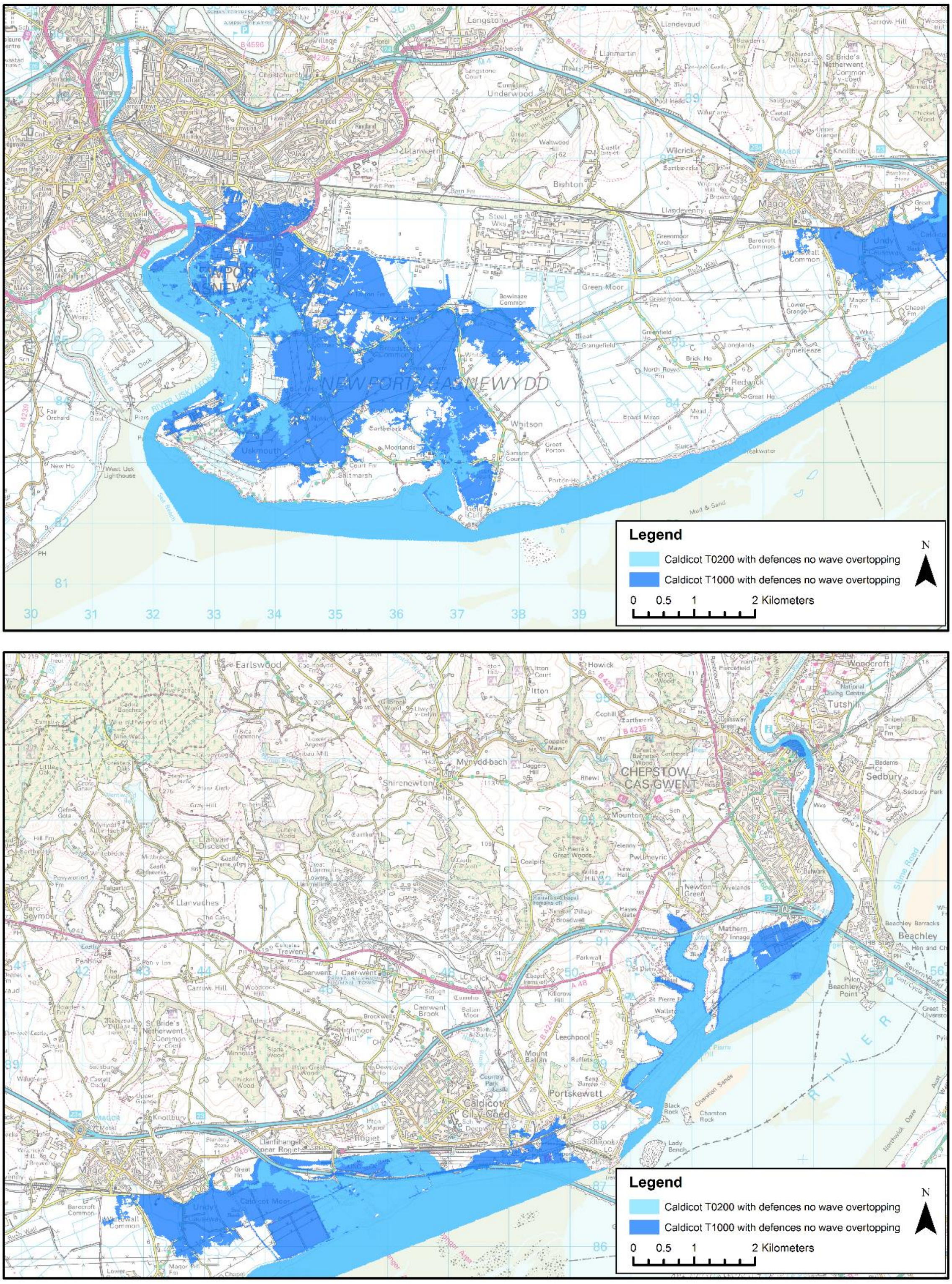


Figure 9-8: With defences flood risk for the Caldicot Levels, 200 and 1,000-year extents with no wave overtopping

9.4.2 With defences and wave overtopping

When wave overtopping inflows are added in addition to the still water boundaries, the flood risk marginally increases for the 200-year event, affecting an additional 20 properties. The flood risk extents significantly increase during the 1,000-year event, leading to the inundation of the Seven Bridge Industrial Estate at Caldicot and large areas of Whitewell Common. The flood extents are shown in Figure 9-9.

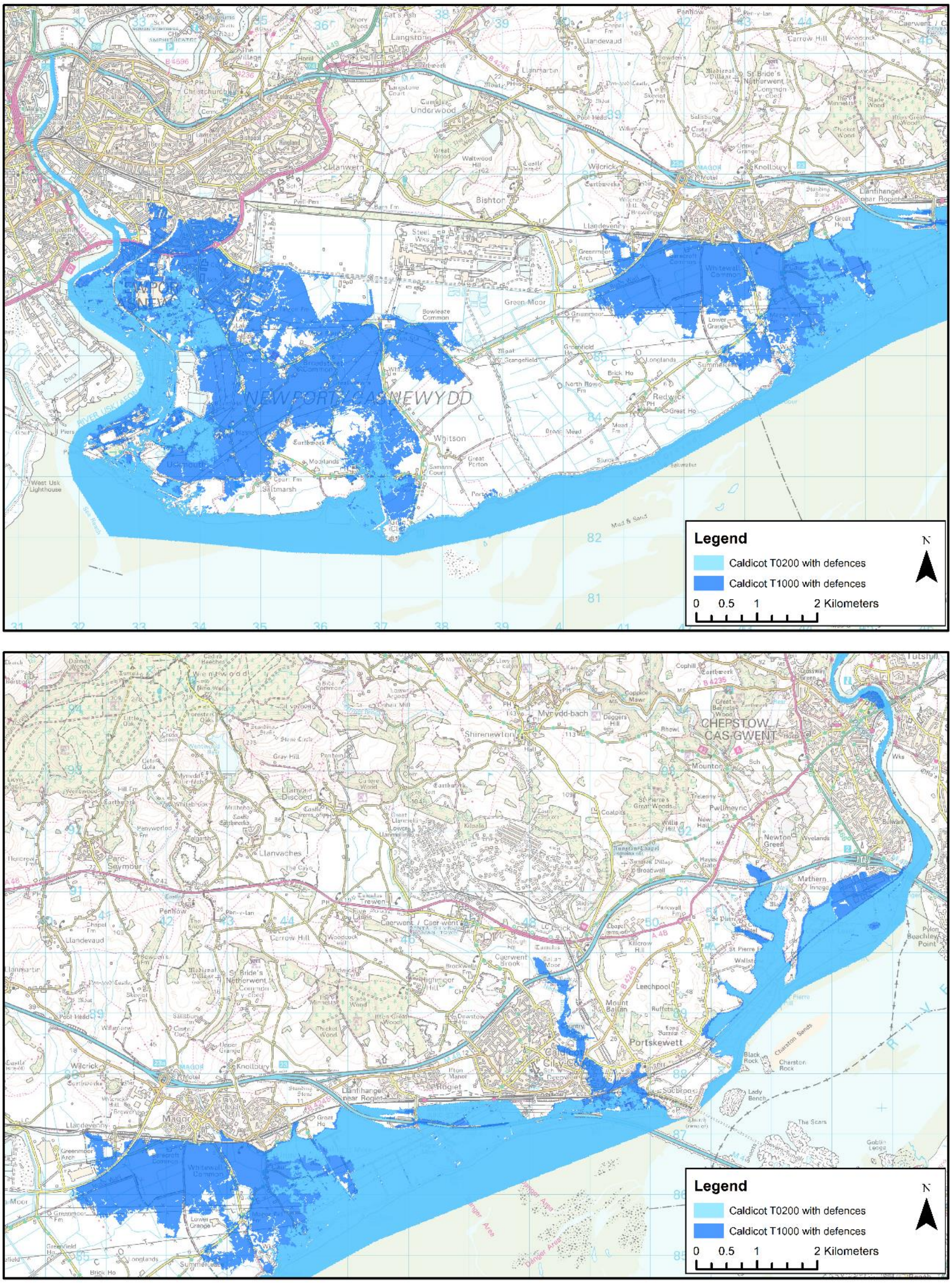


Figure 9-9: With defences flood risk for the Caldicot Levels, 200 and 1,000-year extents, with wave overtopping

9.4.3 No defences

Simulations with "No Defences" greatly increased the flood risk. All of the low lying land within the Caldicot Levels are showing to inundate. Similar to the Wentlooge Levels, the flood extents are topographically controlled producing similar extents for the 200 and 1,000-year events. These inundate all of the industrial areas on the east bank of the Usk along with the residential areas of Barnardstown, Maindee, Somerton and Liswerry. Between Newport and Chepstow, almost all areas to the south of the Great Western Railway are shown to be at flood risk, with the exception of Sudbrook and a few isolated areas of high ground. To the north of the railway the flooding follows the contours surrounding the drainage channels affecting the following locations:

- Monk's Ditch, affecting Llanwern
- Wilcrick Moor reen, affecting Bishton
- St Bride's Brook, affecting Magor
- Low lying land in Rogiet and west Caldicot
- Eastern Caldicot and the Severn Bridge Industrial Estate around Nedern Brook
- Low lying land around Mounton Brook near Mathern
- The Newhouse Farm Industrial Estate near Chepstow.

The flood extents for the no defence scenarios are shown in Figure 9-10.

Figure 9-10: No defences flood risk for the Caldicot Levels, 200 and 1,000-year extents

9.4.4 Modelling the uncertainty in the extreme sea-levels

The extreme sea-levels included in the CFB data (and revised CFB calculated for this study) contain uncertainty. To understand the impact of this uncertainty on the modelling, the upper confidence limits were added to the extreme sea-levels modelled for this study. The tidal-graph will then be regenerated using the same process detailed in section 4.4.1. The flood inundation models were used to simulate the increased flood risk associated with the uncertainty in the sea-level estimates. When the confidence intervals are added to the 200-year event the flood risk marginally increases, affecting the Newhouse Farm Industrial Estate and the low-lying lands of the Caldicot Levels. However, when the confidence intervals are added to the 1,000-year the flood extent increases greatly, inundating lands and properties including those at Spencer Steelworks, Redwick, Caldicot and in Newport. The flood risk extents are summarised in Figure 9-11.

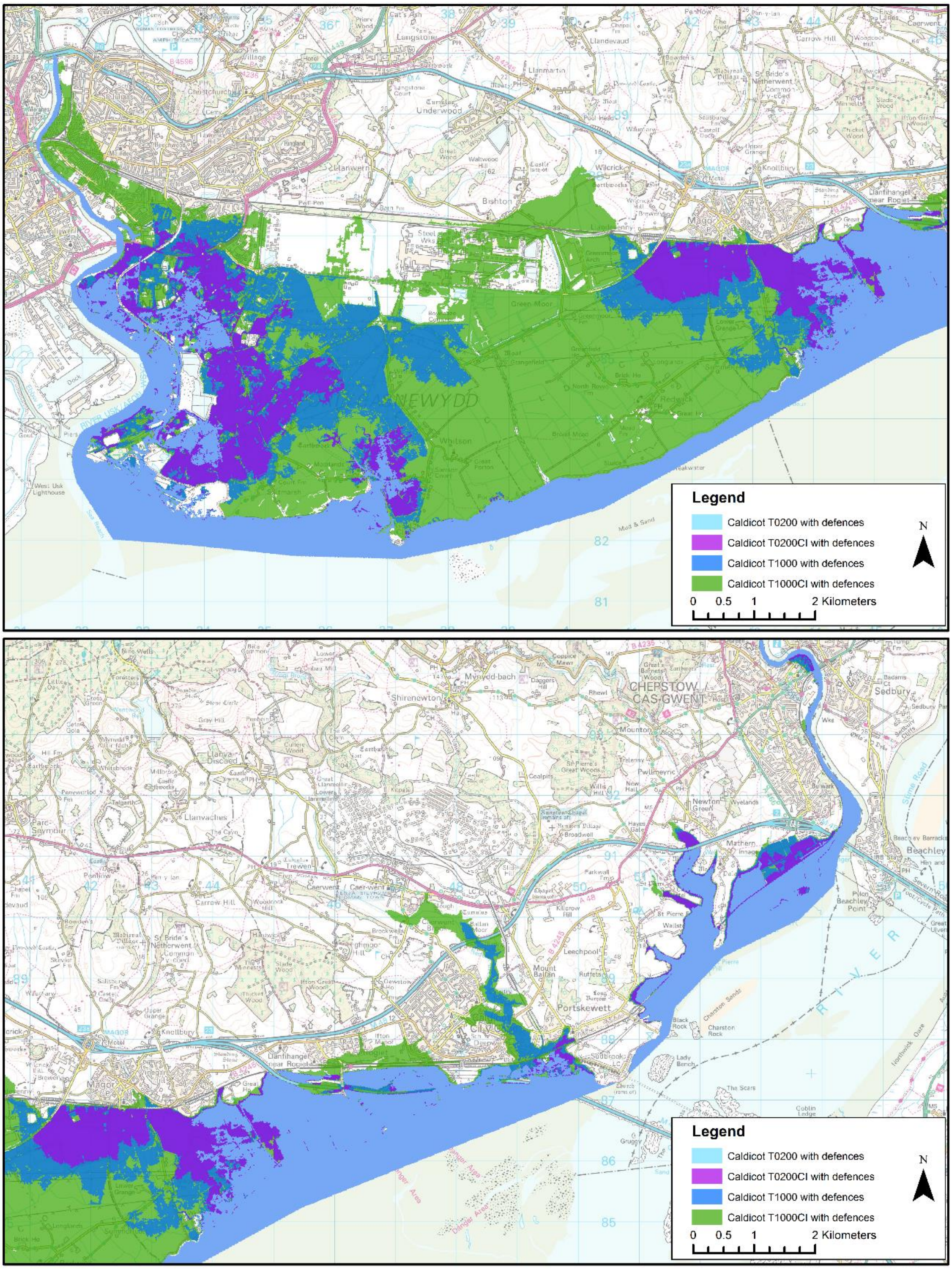


Figure 9-11: Caldicot flood risk with confidence intervals

9.4.5 Property counts for the Caldicot model scenarios

The results of the property counts for the Caldicot "With Defences", "With Defences No Wave Overtopping" and "No Defence" scenarios are detailed in Table 9-4.

Table 9-4: Caldicot property counts

Model scenario	Residential properties (number)	Commercial properties (number)	Unclassified (number)
With Defences model scenario			
T0005 WD	5	4	0
T0020 WD	7	9	0
T0050 WD	7	12	0
T0200 WD	26	92	0
T1000 WD	1,995	501	1
T5500 WD	4,644	709	2
T0200CI WD	556	226	0
T1000CI WD	8,411	1,171	7
With Defences but not wave overtopping model scenario			
T0005 WD NWO	1	2	0
T0020 WD NWO	3	6	0
T0050 WD NWO	3	8	0
T0200 WD NWO	18	80	0
T1000 WD NWO	1,936	395	1
No Defences model scenarios			
T0200 ND	9,657	1,306	3
T1000 ND	10,896	1,402	7

9.4.6 Impact of climate change

The potential impacts of climate change were modelled in the same way as the Wentlooge models with 1.06m added to the sea-levels and 10% added to the wind speeds in the wave transformation models. The resulting nearshore waves were used to calculate wave overtopping discharges.

With Defences simulations

The flood inundation models were used to simulate the increased flood risk for the 200 and 1,000-year return period events. When the impact of climate change was applied the flood risk greatly increases. The flood extents encompass Spencer Steelworks as well as properties in Caldicot, Rogiet, Redwick and all of the properties located on the low-lying land of the Caldicot Levels. The flood extents for the climate change scenarios are shown in Figure 9-12.

Further simulations were carried out to assess the impact of climate change with the uncertainty in the sea-level estimates applied to the calculated levels. When the confidence intervals are added to the climate change runs the flood risk increases, showing additional properties at risk in Caldicot, Rogiet, Magor, Llanwern and the suburbs in Newport (Figure 9-13).

No Defences simulations

Additional simulations were carried out to assess the impact of climate change with the uncertainty in the sea-level estimates for the No Defences scenario. When the values are added to the No Defences simulation the flood extent is increased, although marginally as the No defences simulations without climate change is already very extensive. Additional flooding occurs in Newport, Magor, Rogiet, Caldicot and in St Pierre Park. The flood risk extents and properties at risk are summarised in Figure 9-14 and Table 9-5.

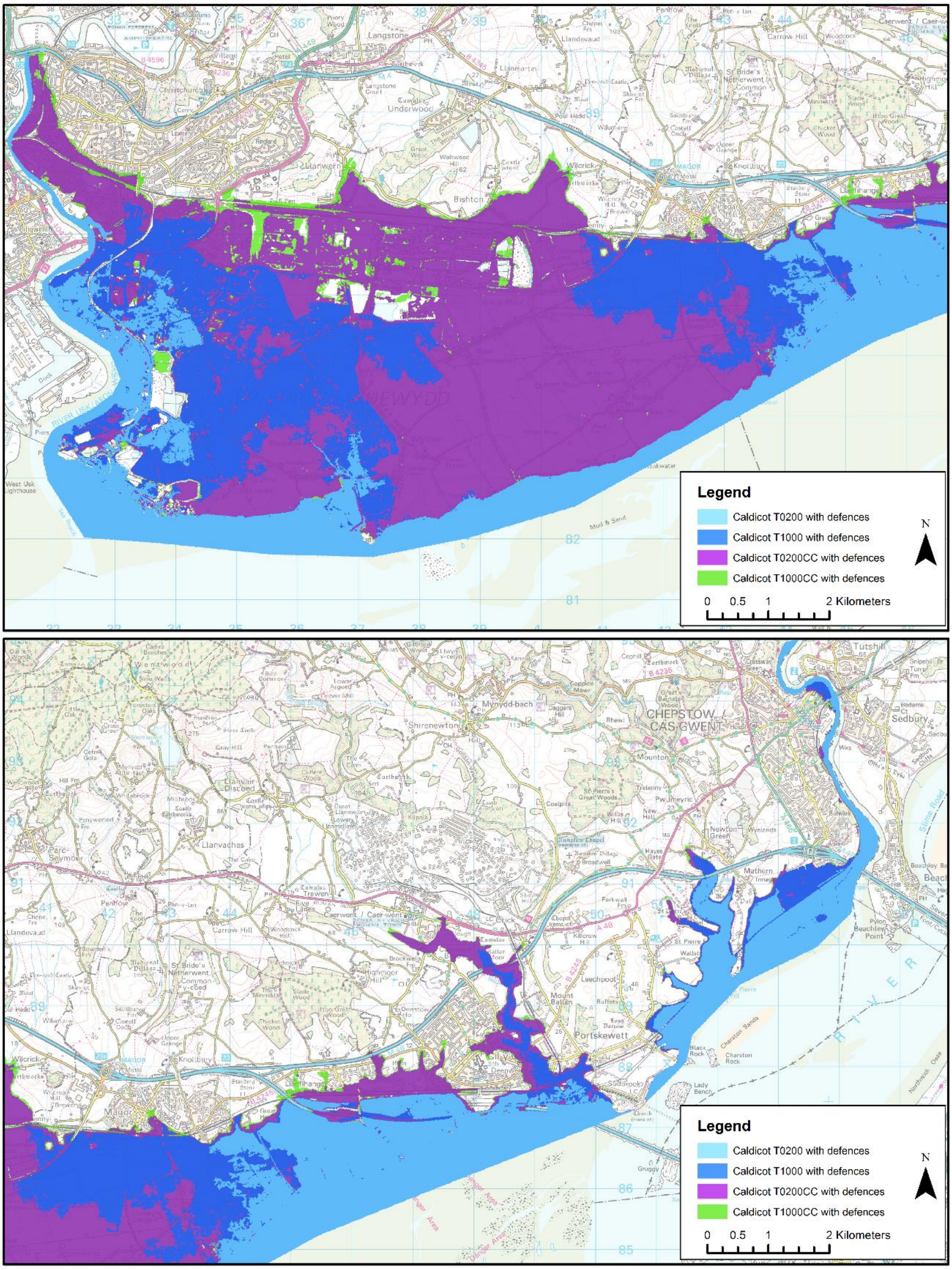


Figure 9-12: Caldicot model climate change flood extents

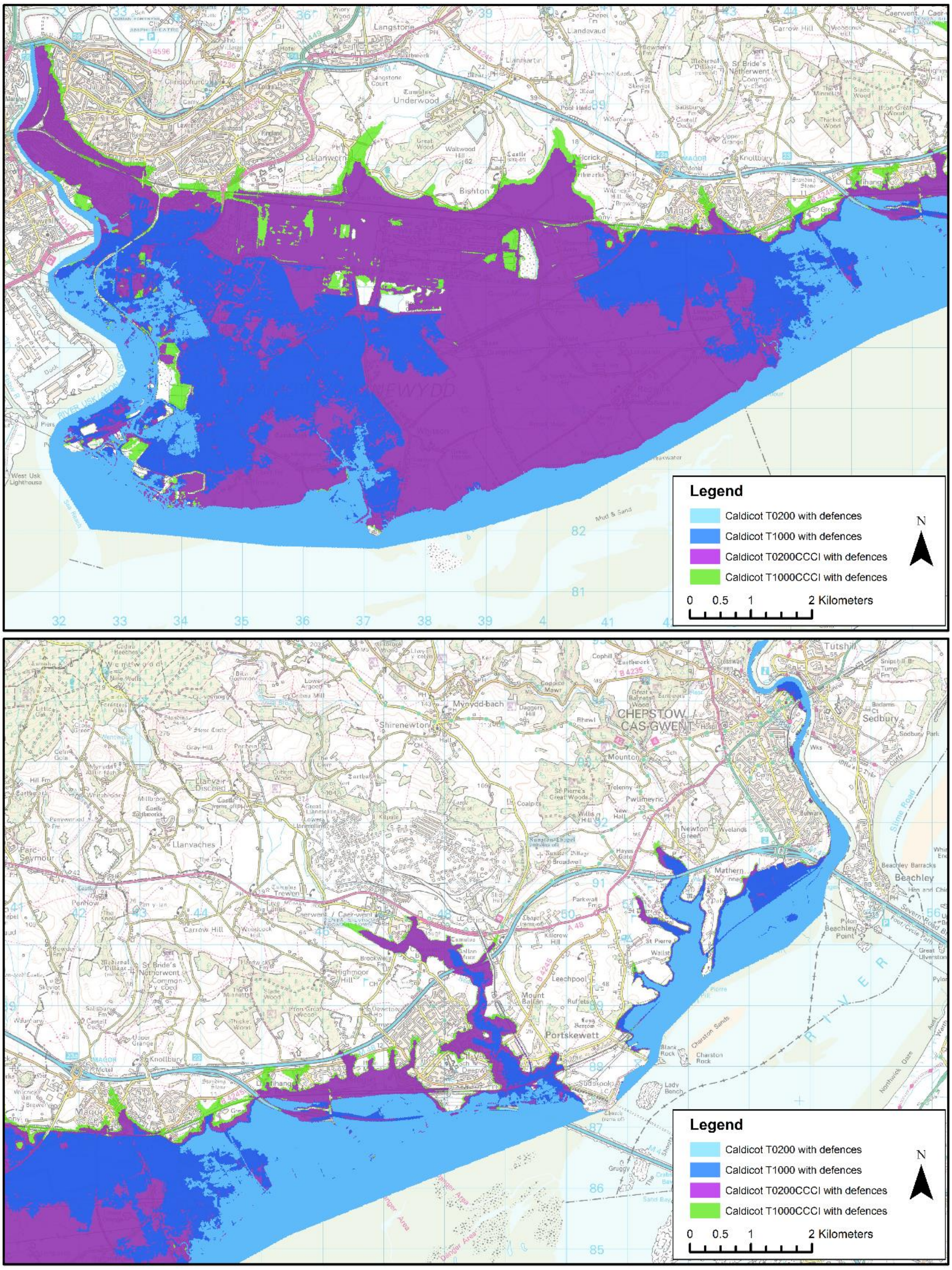


Figure 9-13: Caldicot model climate change flood extents with confidence intervals

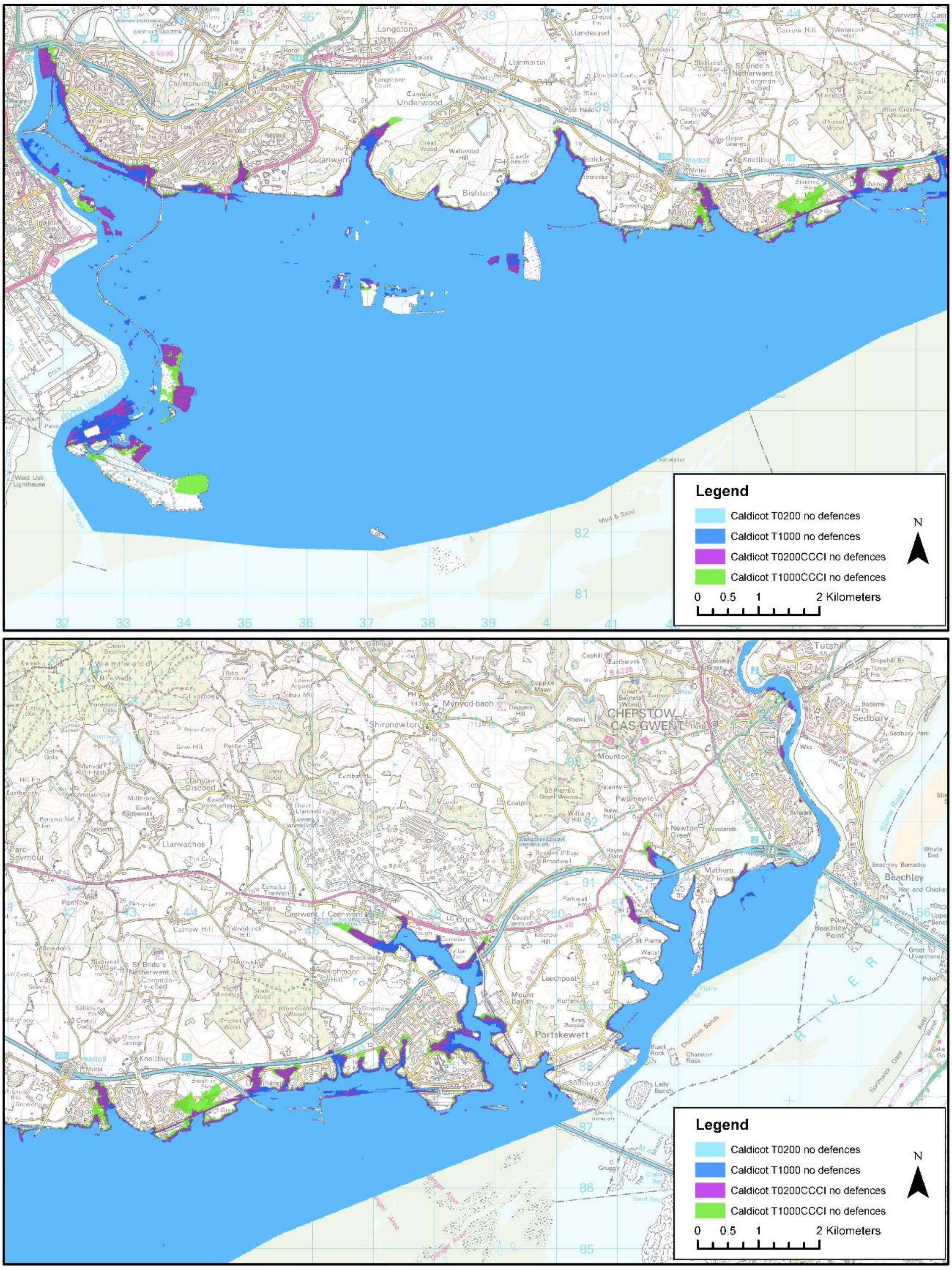


Figure 9-14: No defence flood risk extent for the Caldicot model for climate change with confidence intervals

Table 9-5: Caldicot climate change property counts

Model scenario	Residential properties (number)	Commercial properties (number)	Unclassified (number)
With Defences model scenarios			
T0200 CC WD	10,241	1,450	10
T1000 CC WD	11,916	1,541	11
T0200 CC CI WD	11,201	1,509	11
T1000 CC CI WD	13,403	1,610	12
No Defences model scenarios			
T0200 CC CI ND	12,815	1,570	11
T1000 CC CI ND	13,847	1,645	12

9.5 Additional modelling scenarios

In addition to those required to produce the Flood Zones and ABDs, a range of other scenarios and events were completed. A full list of scenarios and events are listed in Table 9-1. The additional scenarios include breach modelling and tidal gate failures. The adjustments made to the base models are described below.

9.5.1 Breach modelling

The breach modelling described in Chapter 6 was used to guide the breach widths and depths used for the flood inundation modelling. Rather than use the calculated flows from the breach models as inflows into the TUFLOW models, the maximum breach widths and depths were stamped into the defences to allow water to flow in and out of the defence breach.

The peak wave overtopping rates for each defence, at each return period, were analysed and compared to thresholds at which the risk of damage to embankments occur. The thresholds used in the analysis were:

- 0.003m³/s/m (3L/s/m)- Onset of damage to embankment if crest not protected.
- 0.020m³/s/m (20L/s/m) - Onset of damage to embankment if back slope not protected.
- 0.050m³/s/m (50L/s/m) - Onset of damage to embankment even if fully protected.

If the peak overtopping rate exceeded a threshold, for the relevant defence type, the defence was highlighted as being at risk of breaching. As the magnitude of the event increases the number of defences at risk of breaching increases. For the 5-year return period, only one defence was at risk of breaching and this increased to 17 defences for the 1,000-year climate change event, as summarised in Table 9-6.

Table 9-6: Number of breaches per modelled return period

Return period (years)	Number of breaches in the Wentlooge model	Number of breaches in the Caldicot model
5	0	1
20	0	4
200	0	8
1,000	2	9
200 CC	5	12
1,000 CC	5	12

9.5.1.1 Breach widths and depths

The EMBREA model simulated two types of defence failure; piping and overtopping. For defences at risk from failure due to piping, the initial width of the breach was set to 0.1m. For defences at risk of failure due to overtopping, the initial breach width was based on the length of defence with an elevation below the peak water level for the modelled event, therefore, if the peak water level was 8m and 70m of the defence had an elevation below 8m, the initial width would be set at 70m.

Some defences had more than one section of defence below the elevation of the peak water level; for these defences, multiple breaches were applied. Multiple breaches were modelled for defences 8, 27, 29 and 32. The EMBREA model was used to model the lengths and depths of failure for each defence.

The Wentlooge flood inundation model was developed with a 5m grid resolution. All breach widths were rounded to the nearest 5m to allow the defences to be lowered across an entire grid cell. The defence crest levels were lowered by the amount calculated by the breach modelling. The parameters used for each breach is shown in Table 9-7 and the locations of each breach can be seen in Figure 9-15**Error! Not a valid bookmark self-reference.**.

Table 9-7: Breach parameters used in the Wentlooge model

Defence	Event	Breach depth (m)	Breach width (m)
3	1,000	7.43	10
	200 CC	7.43	10
	1,000 CC	7.43	10
4	200 CC	6.77	10
	1,000 CC	6.77	10
5	1,000	5.91	10
	200 CC	5.91	10
	1,000 CC	5.91	10
8a	200 CC	6.80	90
	1,000 CC	6.80	90
8b	1,000 CC	6.80	70
8c	1,000 CC	6.70	60
9	200 CC	7.00	70
	1,000 CC	7.00	70

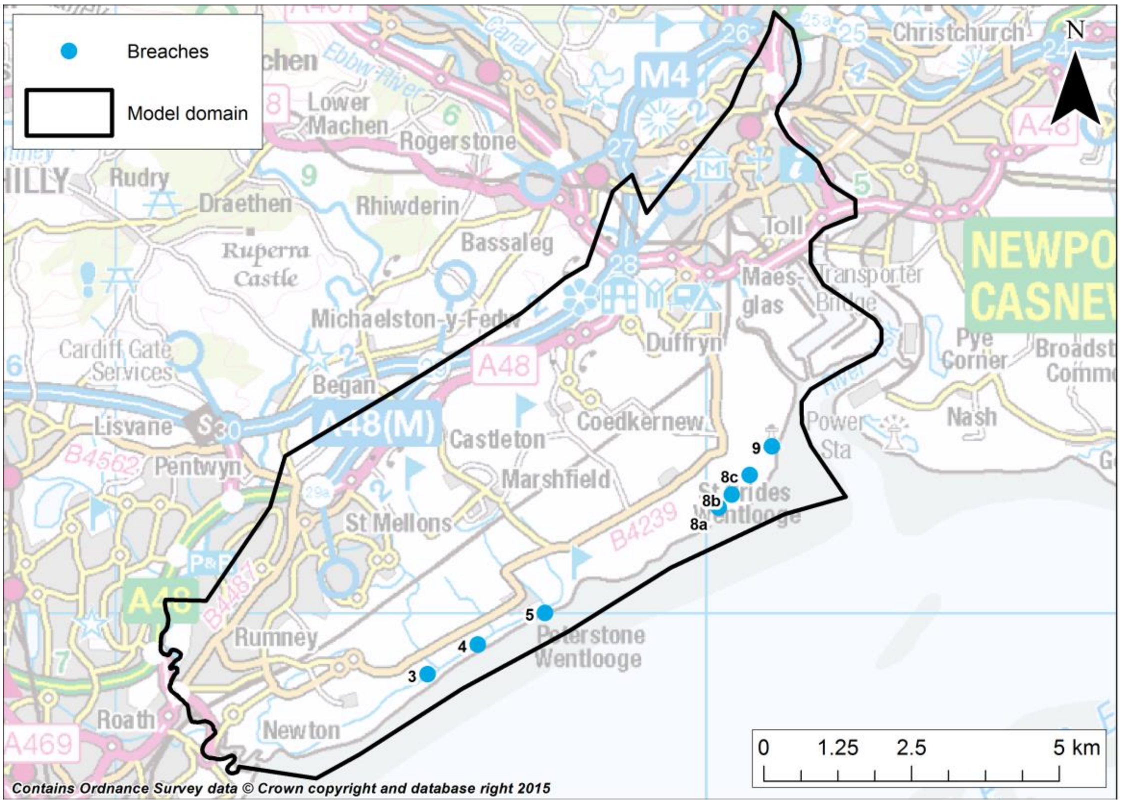


Figure 9-15: Breach locations for the Wentlooge TUFLOW model

The Caldicot flood inundation model was developed using a 10m grid resolution. All breach widths were rounded to the nearest 10m to allow the whole of the defence crest to be lowered across an entire grid cell. The defence crest levels were lowered by the amount calculated by the breach modelling. The parameters used in the modelling are shown in Table 9-8 and the locations of the breaches are shown on Figure 9-16.

Table 9-8: Breach parameters used in the Caldicot model

Defence	Event	Breach depth (m)	Breach width (m)
16	200 CC	6.07	50
	1,000 CC	6.07	50
17	200 CC	6.10	70
	1,000 CC	6.10	70
19	200 CC	5.89	60
	1,000 CC	5.89	60
20	200 CC	6.40	30
	1,000 CC	6.40	30
21	200	6.92	10
	1,000	6.92	10
	200 CC	6.92	10
	1,000 CC	6.92	10
22	20	8.07	40
	200	8.07	40
	1,000	8.07	40
	200 CC	8.07	40
	1,000 CC	8.07	40
25	5	7.14	40
	20	7.14	40
	200	7.14	40
	1,000	7.14	40
	200 CC	7.14	40
	1000 CC	7.14	40
27a	1,000	7.84	40
	200 CC	7.84	40
	1,000 CC	7.84	40
27b	200	7.84	20
	1,000	7.84	20
	200 CC	7.84	20
	1,000 CC	7.84	20
29a	200	7.62	40
	1,000	7.62	40
	200 CC	7.62	40
	1,000 CC	7.62	40
29b	200	7.62	60
	1,000	7.62	60
	200 CC	7.62	60
	1,000 CC	7.62	60
30	20	5.79	30
	200	5.79	30
	1,000	5.79	30
	200 CC	5.79	30
	1,000 CC	5.79	30
31	20	7.81	20
	200	7.81	20
	1,000	7.81	20
	200 CC	7.81	20
	1,000 CC	7.81	20
32a	200 CC	5.96	20
	1,000 CC	5.96	20
32b	200 CC	5.96	20
	1,000 CC	5.96	20

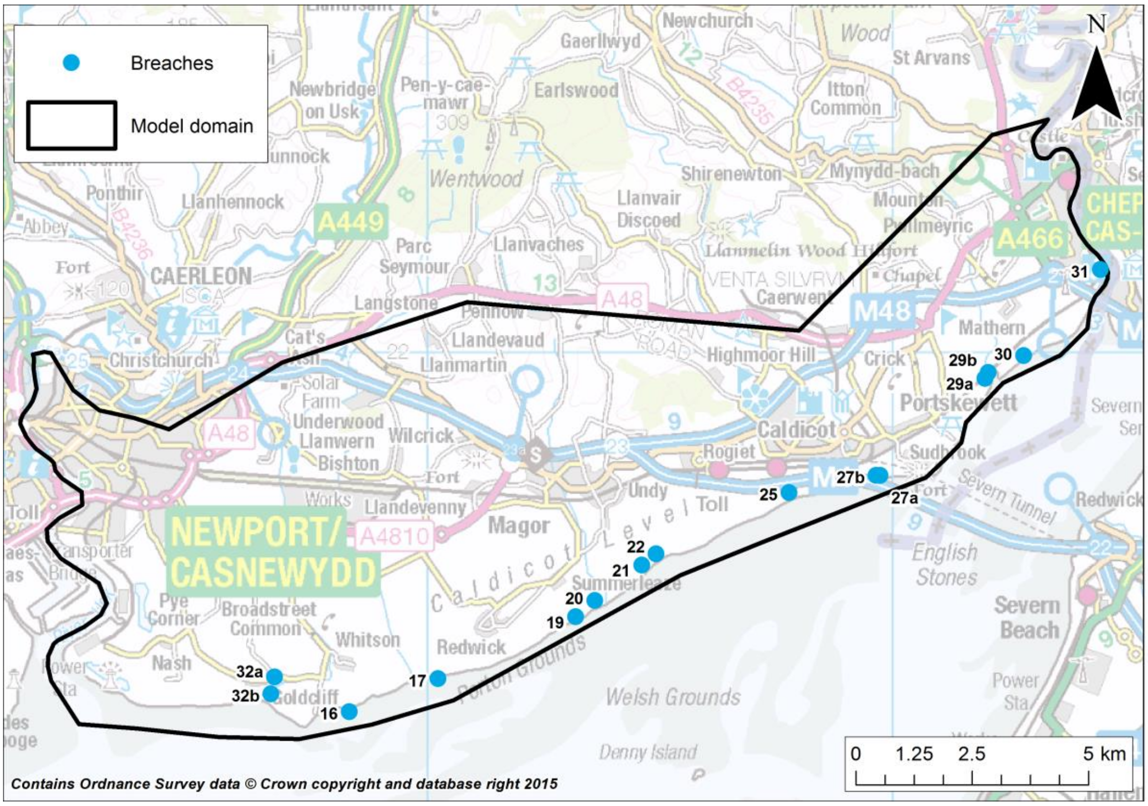


Figure 9-16: Breach locations for the Caldicot TUFLOW model

As a sensitivity test, additional model simulations were completed with a standard 50m breach width applied for all locations for the 200-year event. Two model runs were completed for each model domain, using alternative breaches. Details of the breaches included in each model run can be seen in Table 9-9. Where more than one breach had been used to breach a defence, the breach with a width closest to 50m was used in the sensitivity test runs.

Table 9-9: Details of the breaches included in the 50m breach model simulations

Model domain	Event	Breach numbers used	Total
Caldicot	200-year a	16, 19, 21, 25, 29, 31	5
	200-year b	17, 20, 22, 27, 30, 32	5
Wentlooge	200-year a	3, 5, 9	3
	200-year b	4, 8	2

9.5.2 Breach results

The flood extent for the 1,000-year breach event for the Wentlooge TUFLOW model can be seen in

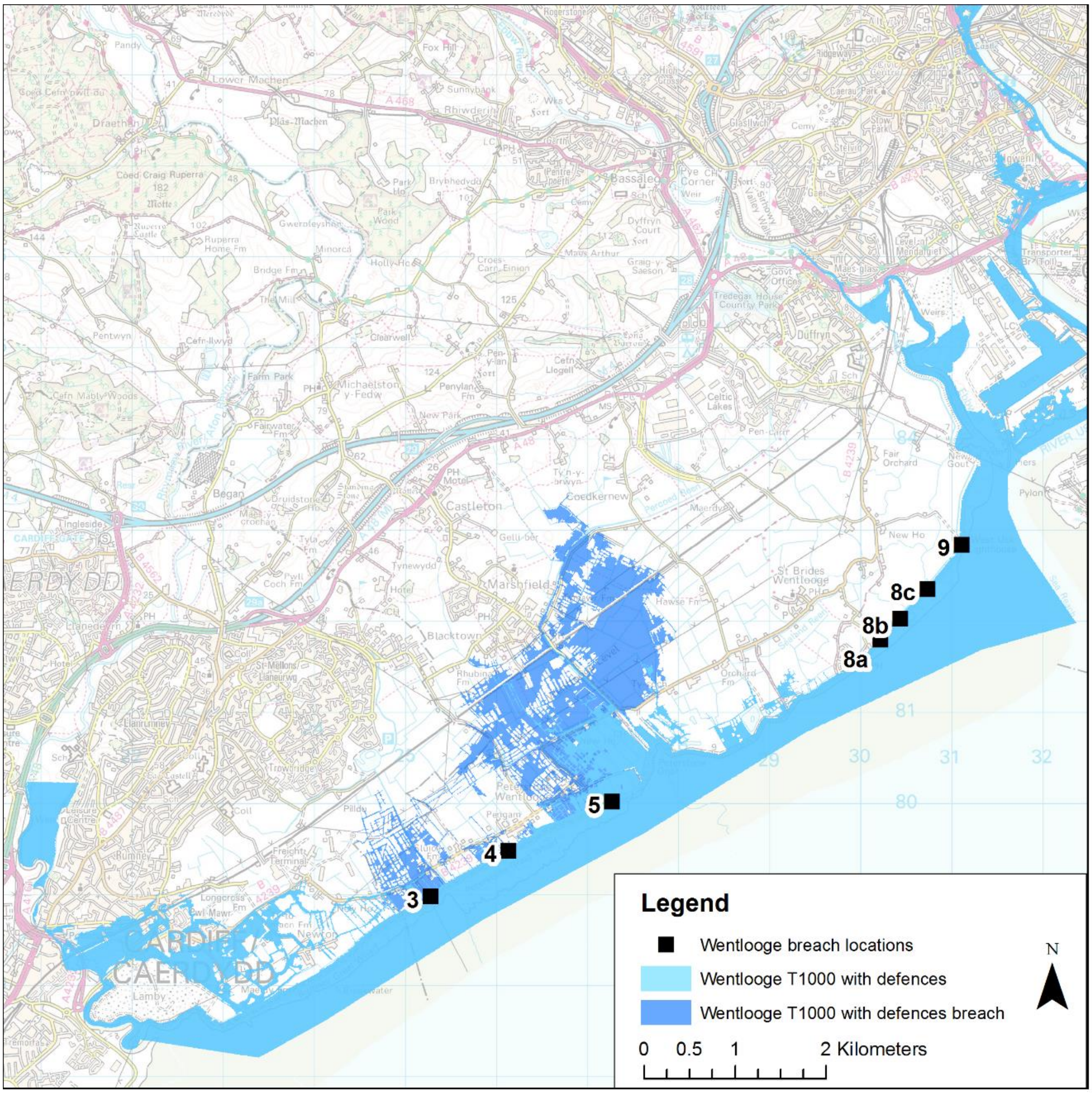


Figure 9-17. This event contained two breaches through defences three and five. The defence breaches are located in the centre of the model domain, so there are no changes to the flood extent around Cardiff in the west and Newport in the east. The flooding mostly affects marshland and farmland to the south of Marshfield and around Broadstreet Common and the number of additional properties affected is negligible. The amount of properties at risk for each breach simulation is summarised in Table 9-10.

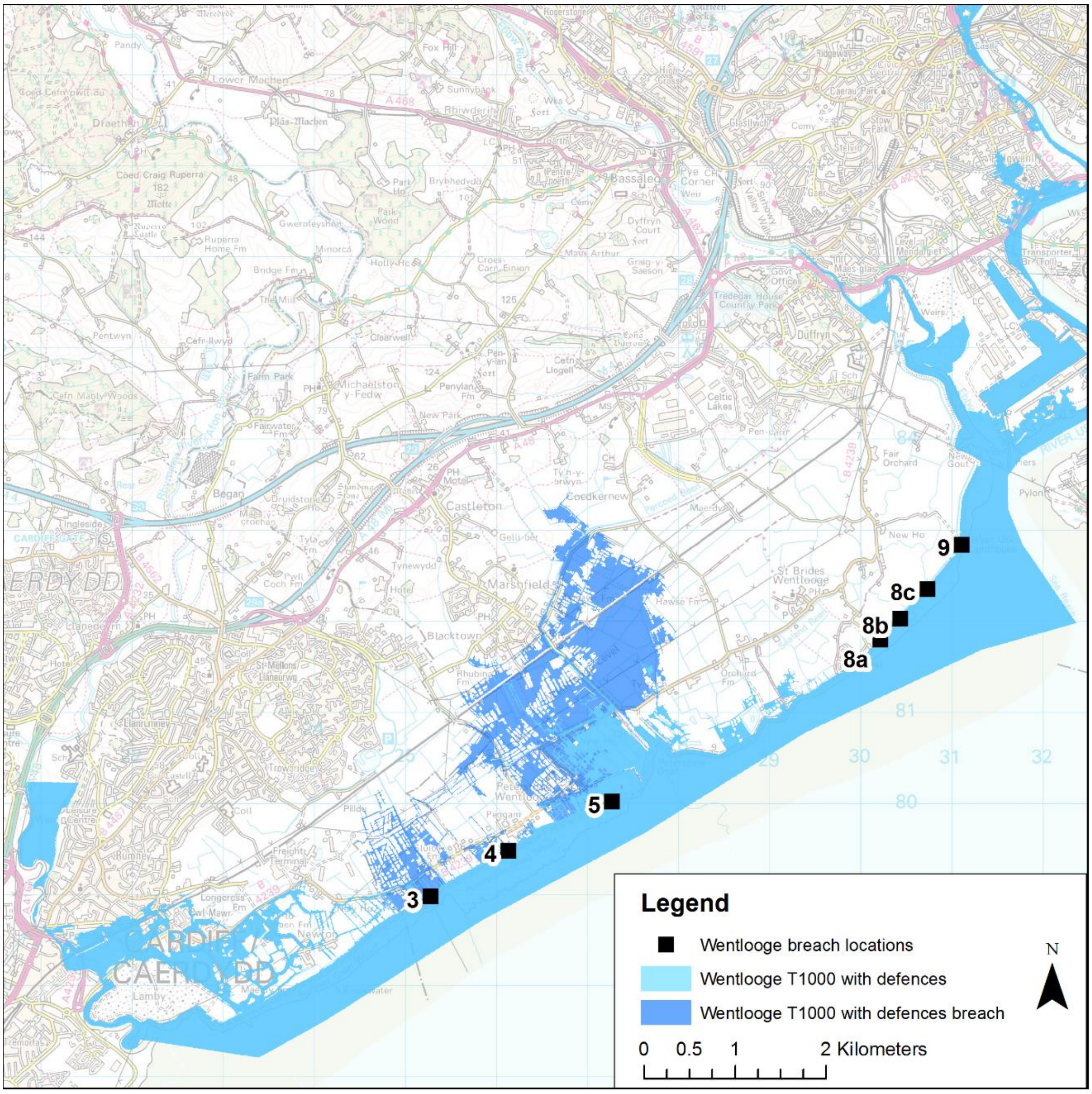


Figure 9-17: Wentlooge 1,000-year breach results

Table 9-10: Wentlooge breach property counts

Model scenario	Residential properties (number)	Commercial properties (number)	Unclassified (number)
Breach model scenario			
T1000	1,392	416	0
T0200 CC	4,981	1,668	0
T1000 CC	7,231	1,883	0
Breach (50m) model scenario			
T0200 a	813	185	0
T0200 b	895	188	0

The flood extent for the 200-year breach event for the Caldicot TUFLOW model can be seen in © Crown Copyright and database rights *2016*. Ordnance Survey Licence No 100030994.

Figure 9-18. This event contained eight breaches of the coastal defences, through defences 21, 22, 25, 27b, 29a, 29b, 30 and 31. As the breaches were located to the east of the model domain there is no change to the flood extent between the River Usk and Magor. However, the flood extent is significantly increased in the east of the model domain, between Caldicot and Chepstow. The flooding mostly affects marshland and fields, although some residential and commercial buildings are affected including buildings in Porkskewett, the Severn Bridge Industrial Estate in Caldicot and the Newhouse Farm Industrial Estate. The amount of properties at risk is summarised in Table 9-11.

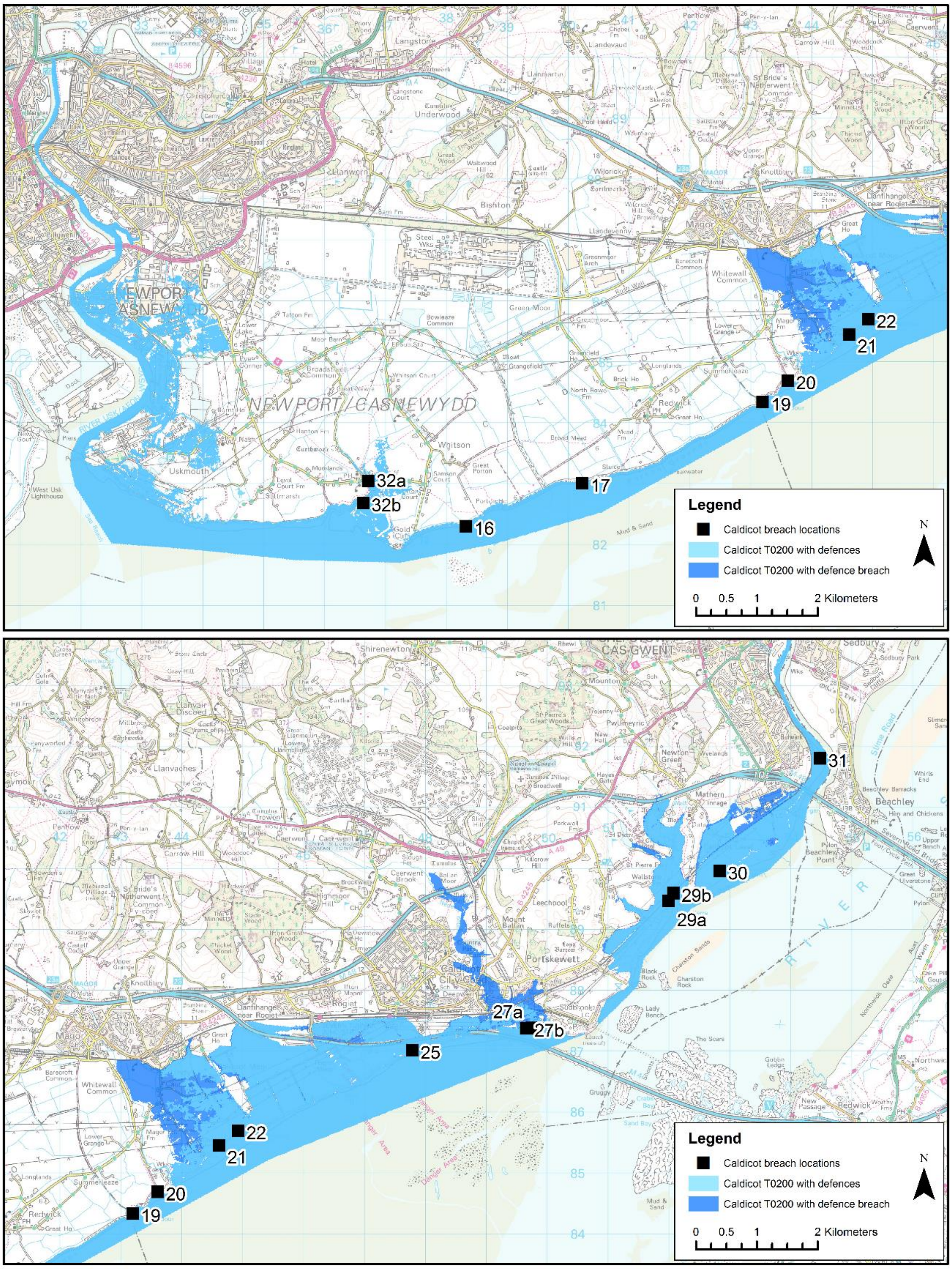


Figure 9-18: Caldicot 200-year breach results

Table 9-11: Caldicot breach property counts

Model scenario	Residential properties (number)	Commercial properties (number)	Unclassified (number)
Breach model scenario			
T0005	5	4	0
T0020	7	9	0
T0200	65	215	0
T1000	5,015	684	1
T0200 CC	10,369	1,460	280
T1000 CC	12,111	1,548	11
Breach (50m) model scenario			
T0200 a	156	153	1
T0200 b	174	230	1

9.5.3 Tidal gate failure modelling

Two scenarios were modelled to assess the impacts of tidal gate failure; (i) tidal gate failure due to blockage of the opening ("Failure Open") and (ii) tidal gate failure due to failure of the gate to close ("Failure Closed"). The "Failure Closed" is the worst case scenario during a fluvial event, as fluvial waters become backed up behind the outfall and are unable to discharge into the Severn Estuary. The "Failure Open" is the worst case scenario during a tidal event, in which high sea-levels are able to pass through the outfall and across the low lying floodplain inland. The details of the tidal gates can be found in Table 9-12.

Tidal gate failure modelling will be based on the "With Defences" model setup. To model tidal gate failure to open, fluvial flows were applied behind the outfall to represent flow to the outfall from the surrounding catchment. The outfall was kept closed throughout the simulation. This was achieved in the model by removing the 1D tidal gate structure from the model. To model the tidal gate failure to close, the 1D tidal gate was kept in the model but the unidirectional element was altered such that flow was permitted in both directions. No fluvial flows were included in this scenario as it was assumed the fluvial flows would be able to drain into the Severn Estuary during the event.

Table 9-12: Tidal gate failure details

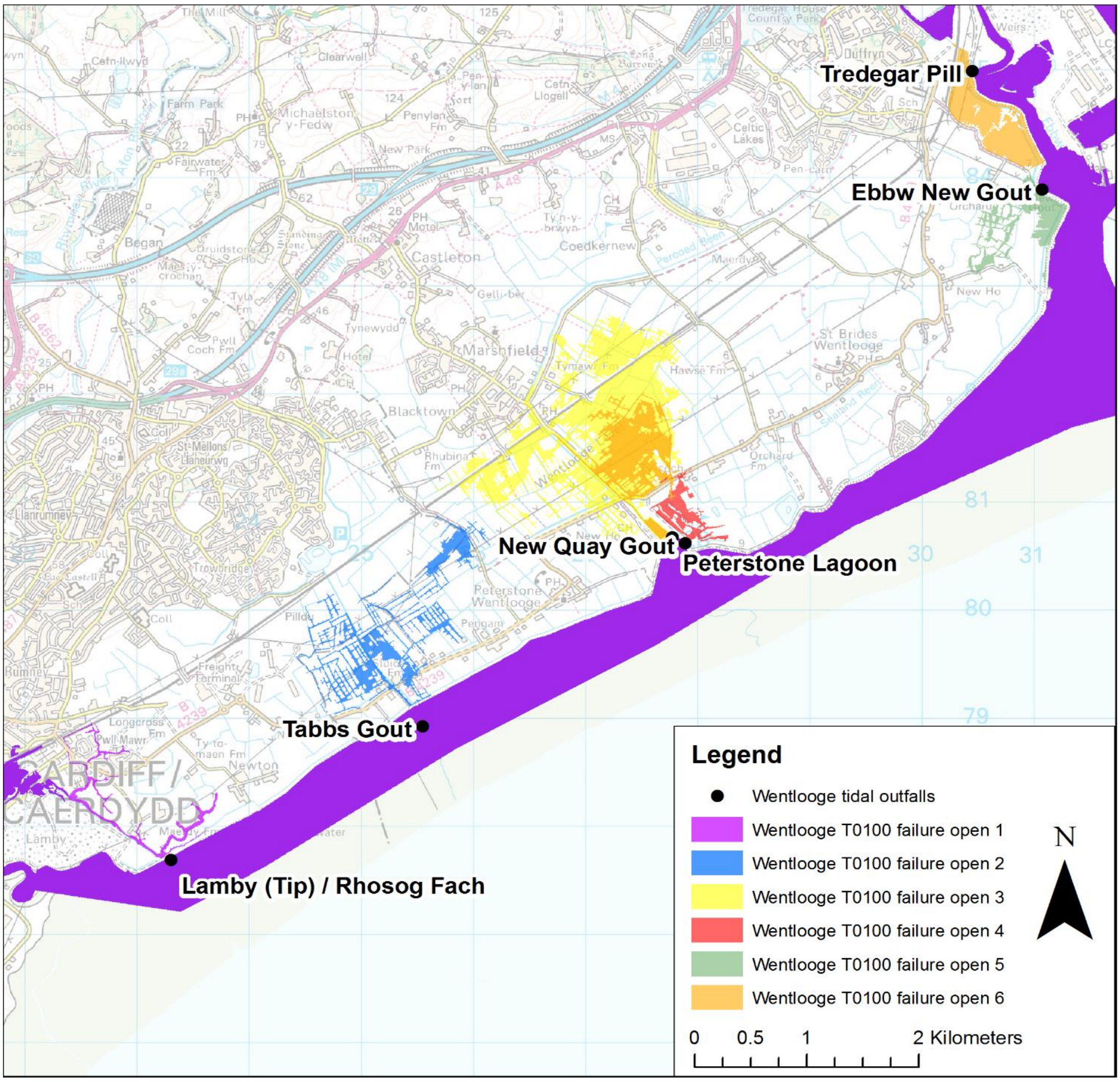
Asset Location	Number	Asset Description	Easting	Northing
Tredegar Pill	1	Unidirectional flapped rectangular outfall	330435	184990
Ebbw New Gout	2	Unidirectional flapped rectangular outfall	331070	183885
New Quay Gout	3	Unidirectional flapped circular outfall	327898	180628
Peterstone Lagoon	4	Unidirectional flapped rectangular outfall	327756	180688
Tabbs Gout	5	Unidirectional flapped circular outfall	325450	179090
Lamby (Tip) / Rhosog Fach	6	Unidirectional flapped circular outfall	323284	177713
Monks' Ditch	7	Unidirectional flapped rectangular outfall	336901	182968
Fisher's Gout	8	Unidirectional flapped circular outfall	336707	183025
Elver Pill	9	Unidirectional flapped circular outfall	339710	182955
Windmill Reen	10	Unidirectional flapped rectangular outfall	340977	183163
Coldharbour	11	Unidirectional flapped circular outfall	343123	184243
Magor Pill	12	Unidirectional flapped circular outfall	343847	184875
Collister Pill	13	Unidirectional flapped circular outfall	345241	185704
Caldicot Pill	14	Unidirectional flapped rectangular outfall	348985	187302
St Pierre Pill	15	Unidirectional flapped circular outfall	351925	189641

9.5.4 Tidal gate failure results

The flood extents for the tidal gate failure in both the open and the closed state foe both model domains can be seen in © Crown domains can be seen in © Crown Copyright and database rights *2016*. Ordnance Survey Licence No 100030994.

Figure 9-19 to © Crown Copyright and database rights *2016*. Ordnance Survey Licence No 100030994.

Figure 9-22. The extents vary depending on the local topography and watercourses located around the tidal gate. Table 9-13 shows the number of properties affected for each tidal gate failure in each state.



© Crown Copyright and database rights *2016*. Ordnance Survey Licence No 100030994.

Figure 9-19: Wentlooge tidal gates failure open results

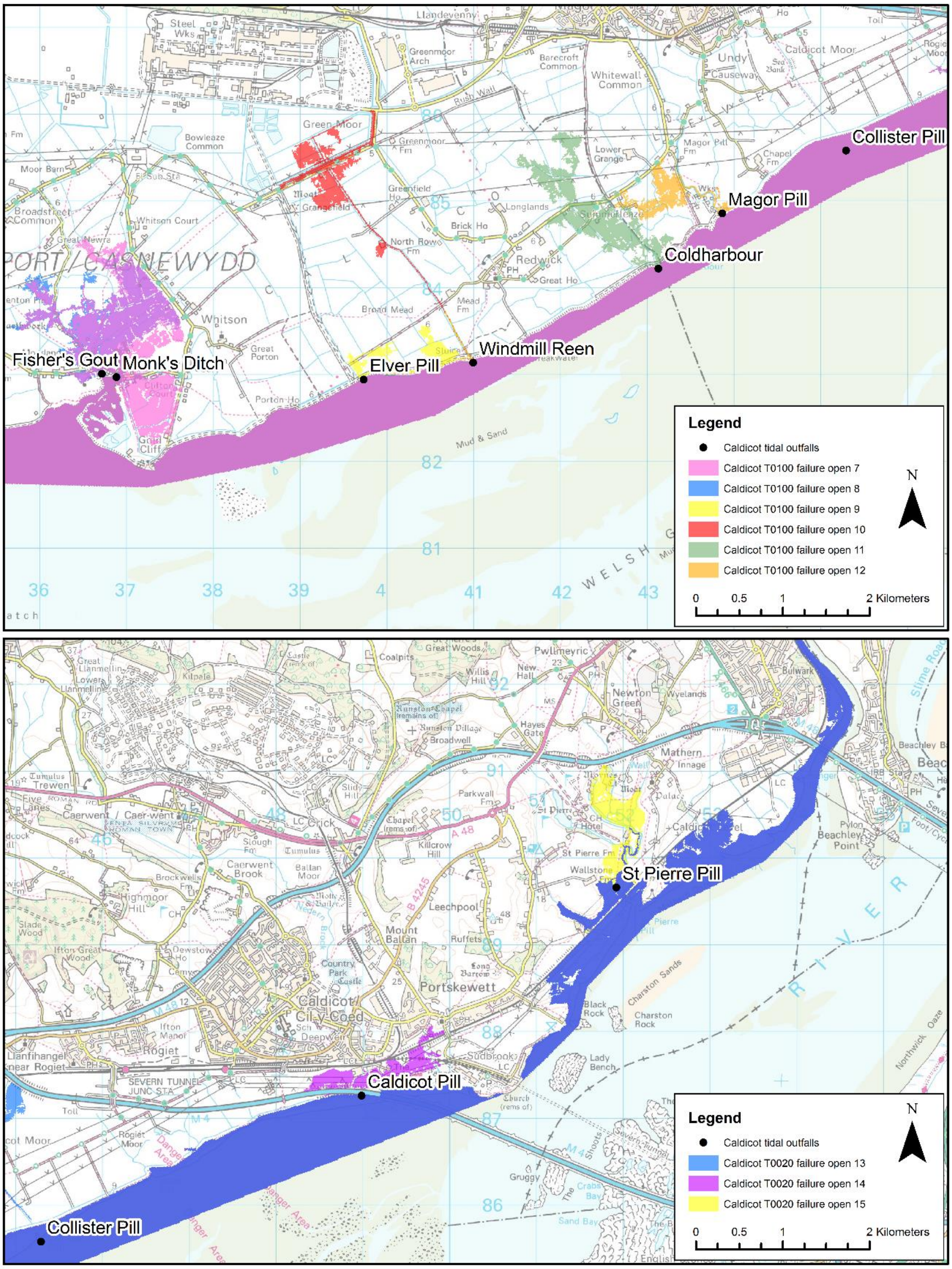


Figure 9-20: Caldicot tidal gates failure open results

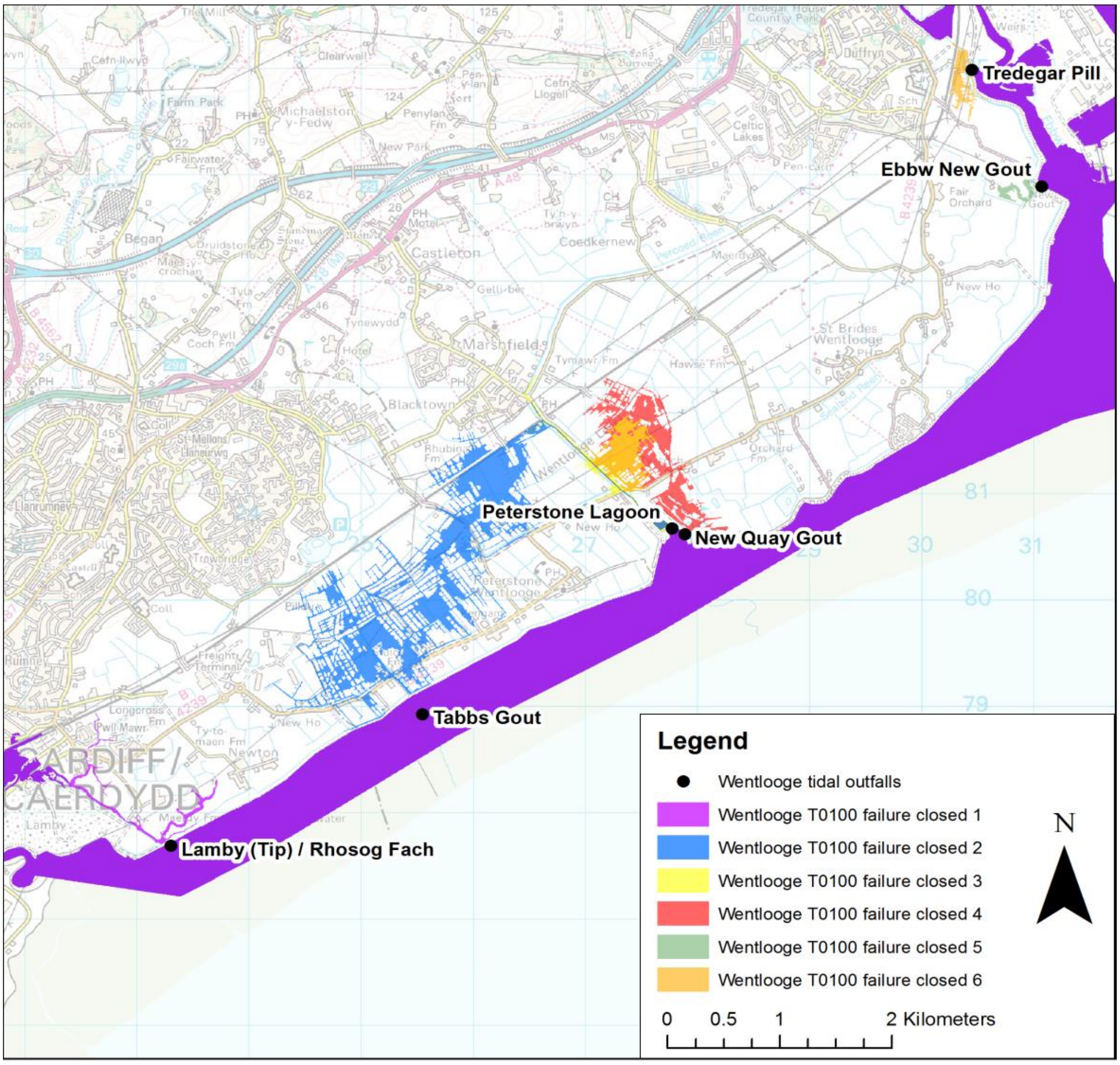


Figure 9-21: Wentlooge tidal outfalls failure closed flood extents

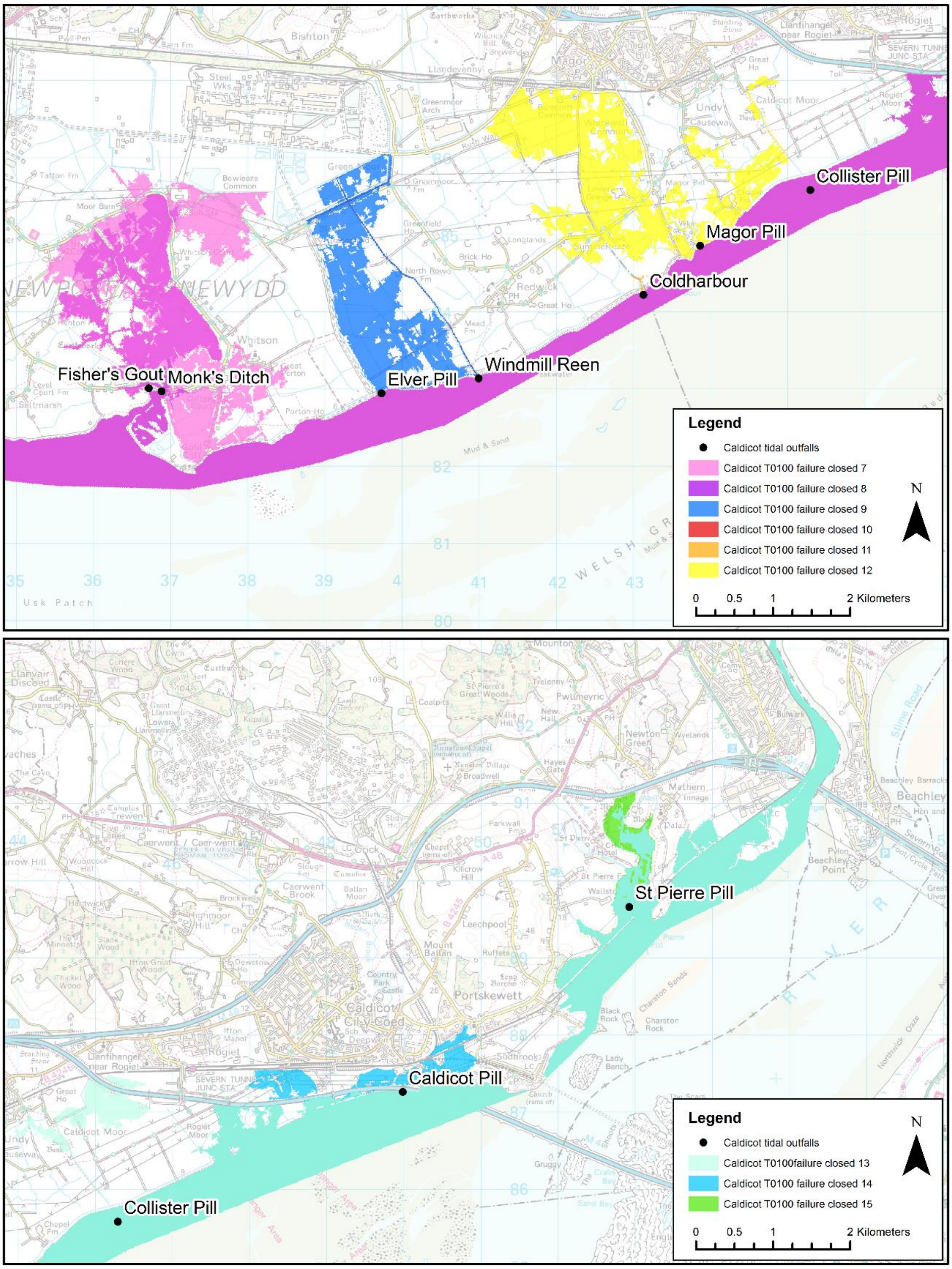


Figure 9-22: Caldicot tidal gates failure closed results

In general, there are very few properties at risk from the outfall failure scenarios. Most failures whether during tidal or fluvial events, result in the flooding of one to two properties. On the Wentlooge Levels all failure open and closed scenarios result in a maximum of one property flooding.

On the Caldicot Levels, failure of the outfalls to open, trapping in fluvial flows, at Fisher's Gout and Monk's Ditch result in 8 and 22 properties flooding around Goldcliff. Failure of the outfall at Magor Pill floods nine properties and failure at Caldicot Pill, 5 properties. Failure of the outfall to close, allowing tidal ingression results in flooding of 13 properties from Fisher's Gout and Monk's Ditch but only a maximum of one property from a failure at any of the other outfalls.

Table 9-13: Caldicot and Wentlooge tidal gate failure property counts

Model scenario	Residential properties (number)	Commercial properties (number)	Unclassified (number)
Tidal gate failure open model scenario			
T100 F1O	0	1	0
T100 F2O	0	0	0
T100 F3O	1	0	0
T100 F4O	0	0	0
T100 F5O	0	0	0
T100 F6O	0	1	0
T100 F7O	13	0	0
T100 F8O	13	0	0
T100 F9O	0	0	0
T100 F10O	0	0	0
T100 F11O	0	0	0
T100 F12O	1	0	0
T20 F13O	0	0	0
T20 F14O	1	1	0
T20 F15O	0	0	0
Tidal gate failure closed model scenario			
T100 F1C	0	0	0
T100 F2C	1	0	0
T100 F3C	0	0	0
T100 F4C	0	1	0
T100 F5C	0	0	0
T100 F6C	0	0	0
T100 F7C	20	2	0
T100 F8C	7	1	0
T100 F9C	0	0	0
T100 F10C	0	0	0
T100 F11C	0	0	0
T100 F12C	7	2	0
T100 F13C	0	0	0
T100 F14C	3	2	0
T100 F15C	0	0	0

9.6 Flood Zone and Areas Benefiting from Defences mapping

The objective of this task was to process the results of the two-dimensional tidal inundation models to create outlines for the Flood Map. This involved producing and editing flood outlines, deriving Areas Benefiting from Defences and Flood Zone extents.

9.6.1 Produce Flood Zones

The output from the "No Defences" 0.5% AEP and 0.1% AEP simulations, and the "With Defences" 0.5% AEP and 0.1% AEP simulations were used to produce Flood Zones 3 and 2. To produce outlines for each simulation a number of processes were undertaken. First, the maximum depth grids for each simulation were converted to polygons that show the maximum extent of flooding. Second, dry islands with an area of less than 200m^2 were removed from the outlines. The 0.5% AEP "No Defences" and "With Defences" outlines were combined to produce the flood extent for Flood Zone 3. The 0.1% AEP "No Defences" and "With Defences" outlines were combined to produce the flood extent for Flood Zone 2.

9.6.2 Produce Areas Benefiting from Defences Polygons

The modelled flood extents were also used to produce the tidal Areas Benefiting from Defences (ABD). The tidal ABDs are simply the difference between the tidal "No Defences" 0.5% AEP and "With Defences" 0.5% AEP outlines.

9.7 Standard of Protection

The Standard of Protection (SoP) provided by the defences were calculated for defences 1-32. Mean overtopping discharge rates were used to determine the SoP against the following Eurotop limits for overtopping:

- 0.0003m^3/s/m (0.03L/s/m) - Unaware Pedestrian, normal conditions but where the pedestrian has no clear view of incoming waves and can be easily alarmed.
- 0.0001m^3/s/m (0.1L/s/m) - Aware Pedestrian, with a clear view of the sea and incoming waves, pedestrian is happy to get wet.
- 0.001m^3/s/m (1L/s/m) - Trained Staff, well protected and expecting to get wet, overtopping flows at low levels and only low risk of fall from walkway.
- 0.00001m^3/s/m (0.01L/s/m) - Fast Moving Vehicle, overtopping at depths with possibility of high velocity jets developing.
- 0.01m^3/s/m (10L/s/m) - Slow Moving Vehicle, overtopping at low depths only the vehicle will not become immersed, unlikely for jets to develop.
- 0.03m^3/s/m (30L/s/m) - As much of the coastal frontage along the Caldicot and Wentlooge Levels is agricultural land and sparsely populated, in the Severn Estuary Strategy 0.03m^3/s/m was set as the acceptable overtopping rate for earth embankments.
- 0.2m^3/s/m (200L/s/m) - As much of the coastal frontage along the Caldicot and Wentlooge Levels is agricultural land and sparsely populated, in the Severn Estuary Strategy 0.2m^3/s/m was set as the acceptable overtopping rate for wave return walls.

Table 9-14 shows the return period mean discharge exceeding these thresholds.

Table 9-14: Standard of Protection Estimates for defences 1-9 for Wentlooge

Defence (based on WO)	Unaware pedestrian (0.00003m^3/s/m)	Aware pedestrian (0.0001m^3/s/m)	Trained staff (0.001m^3/s/m)	Slow moving vehicle (0.01m^3/s/m)	Acceptable rate from SEFRMS for earth embankment (0.03m^3/s/m)	Acceptable rate from SEFRMS for recurve wall (0.2m^3/s/m)
1*	200	500	>1,000	>1,000	>1,000	>1,000
2	>1,000	>1,000	>1,000	>1,000	>1,000	>1,000
3	100	200	500	>1,000	>1,000	>1,000
4	500	500	>1,000	>1,000	>1,000	>1,000
5	5	20	200	>1,000	>1,000	>1,000
6	20	20	50	200	500	>1,000
7*	20	50	500	>1,000	>1,000	>1,000
8	500	500	1,000	>1,000	>1,000	>1,000
9	50	100	1,000	>1,000	>1,000	>1,000
Note: The majority of the defences are grassed embankments. The defences marked with an * have wave return walls.						

Table 9-15: Standard of Protection Estimates for defences 10-32 for Caldicot

Defence (based on WO)	Unaware pedestrian ($0.00003m^3/s/m$)	Aware pedestrian ($0.0001m^3/s/m$)	Trained staff ($0.001m^3/s/m$)	Slow moving vehicle ($0.01m^3/s/m$)	Acceptable rate from SEFRMS for earth embankment ($0.03m^3/s/m$)	Acceptable rate from SEFRMS for recurve wall ($0.2m^3/s/m$)
10	500	500	500	500	500	>1,000
11	5	5	20	50	200	>1,000
12	5	5	1,000	>1,000	>1,000	>1,000
13*	1,000	>1,000	>1,000	>1,000	>1,000	>1,000
14*	5	>1,000	>1,000	>1,000	>1,000	>1,000
15*	5	5	100	>1,000	>1,000	>1,000
16*	20	50	>1,000	>1,000	>1,000	>1,000
17*	20	100	>1,000	>1,000	>1,000	>1,000
18*	50	500	>1,000	>1,000	>1,000	>1,000
19*	>1,000	>1,000	>1,000	>1,000	>1,000	>1,000
20*	>1,000	>1,000	>1,000	>1,000	>1,000	>1,000
21	20	20	50	200	500	>1,000
22	5	5	20	50	100	>1,000
23	5	5	5	20	20	500
24*	20	50	75	200	500	1,000
25	5	5	5	20	50	500
26*	50	75	100	500	1000	>1,000
27	20	50	75	500	500	>1,000
28	5	5	20	50	75	500
29	50	50	75	75	100	500
30	5	5	20	50	200	>1,000
31	5	5	20	50	100	500
32	>1,000	>1,000	>1,000	>1,000	>1,000	>1,000
Note: The majority of the defences are grassed embankments. The defences marked with an * have wave return walls.						

The aim of the defence improvements at Tabbs and Portland Grounds was to achieve a SoP of 1 in 1,000 in 2010 and 1 in 200 in 2030. It can be seen from the results in Table 9-14 that there are several overtopping sections on the Caldicot Levels that have a SoP less than 1 in 1,000 for the acceptable overtopping rates of earth embankments and wave return walls. On the Wentlooge Levels only defence 6, near Peterstone Gout, has a SoP less than 1 in 1,000 for a grassed embankment. On the Caldicot Levels there are several sections with a SoP less than 1 in 1,000. For grassed embankments these are defences 21, 22, 23, 25, 27, 28, 29, 30 and 31. For defences with wave return walls, all of the overtopping discharges are lower than the acceptable discharge of $0.2m^3/s/m$.

The defences with the lowest SoP are numbers 22, 23, 25, 28, 29 and 31 which all have a SoP less than 1 in 200-years and are located between Portland Grounds and Chepstow.

9.8 Mapping and model outputs

In addition to the Flood Map outputs, GIS outputs were produced for each model simulation. Outputs include (i) maximum extent of flooding; (ii) maximum water level grids; (iii) maximum depth grids; (iv) maximum velocity grids and (v) maximum hazard rating grids.

9.8.1 Maximum extent of flooding

An ESRI shapefile that shows the maximum extent of flooding was produced from the maximum depth grid. This involved converting the grid to a polygon. The outlines were then processed to remove isolated wet areas and dry islands less than 200m^2. For coastal frontages experiencing overtopping any gaps between wet areas on the seaward and landward side of the defence were joined.

9.8.2 Flood Hazard Rating

The TUFLOW model control files were set up so that Flood Hazard Ratings were automatically output for each simulation. The following formula was used to determine the Flood Hazard Rating:

Hazard Rating[45] = Depth x (Velocity + 0.5) + (Debris Factor)

Debris Factor = 0.5, for d < 0.25

Debris Factor = 1, for d >= 0.25

9.8.3 Animations

The creation of model animations is a useful way of communicating model results to wider interested parties. Six animations were created across the two model domains.

9.9 Update overtopping spreadsheet and create new visualisation tools

The model results were used to update the SoP spreadsheet and to create new GeoPDF visualisation tools.

The wave overtopping rates at each of the defences were calculated for hundreds of thousands of combinations of conditions from the Monte Carlo 10,000-year event set. The results of these calculations were used to create look-up files which were then grouped into bands of similar flood risk. These groupings represent no flood risk, initiation of flooding, moderate flooding and maximum risk. The groupings are matched to the relevant flood extents from the "With Defences" still water and wave overtopping model scenarios.

Interactive GeoPDFs were created to visualise the results with one map created for each of the Wentlooge and Caldicot Levels. These maps are interactive and allow the user to select the forecast conditions from a range of sea-levels, wind speeds and wind directions and the software then returns the risk to properties, infrastructure and expected flood extent. The map enables the user to toggle between the flood extent, flood depths or the hazards. The locations of the properties and critical infrastructure flooded are highlighted on the maps. Additional functionality has been added to allow the user to switch between the "With Defences", Breach and Outfall Failure model scenarios.

[45] FD2321/TR1 The Flood Risks to People Methodology – Defra 2006

10 Flood warning

10.1 Updates to the Flood Alert and Warning Areas

The Flood Alert and Warning Areas were updated using the modelled flood extents. The Flood Alert area was composed of the Flood Zone 2 outline. The Flood Warning Area updates were based on the existing Flood Warning Areas and updated to match the new modelled flood extents.

The Flood Alert Areas were based on the existing 17 existing Flood Alert areas. New Flood Alert areas were generated for 10 out of the 17 existing Flood Alert Areas. New areas were not generated for other four Flood Alert areas as they were outside of the area covered by the Caldicot and Wentlooge TUFLOW models. The 10 areas and their unique name codes are shown in Table 10-1 and Figure 10-1.

Table 10-1: Flood Alert codes and coverage

Flood Alert Code	Coverage
103FWC198206	Coast at the Wentlooge Levels in the Cardiff Area
103FWCt04	Coast at the Wentlooge Levels in the Newport Area
103FWF024	River Ebbw at Bassaleg
103FWTt10	Usk Estuary at Pill
103FWCt02	Coast at the Caldicot Levels
103FWTt01a	Wye Estuary at Chepstow
103FWTt04	Usk Estuary at Riverside
103FWTt08	Usk Estuary at Maindee, North Liswerry and Spytty Pill
103FWTt09	Usk Estuary at South Liswerry
103FWTt06	Usk Estuary at Uskmouth and Old Town Docks[46]

[46] The Old Town Docks is outside the scope of this model.

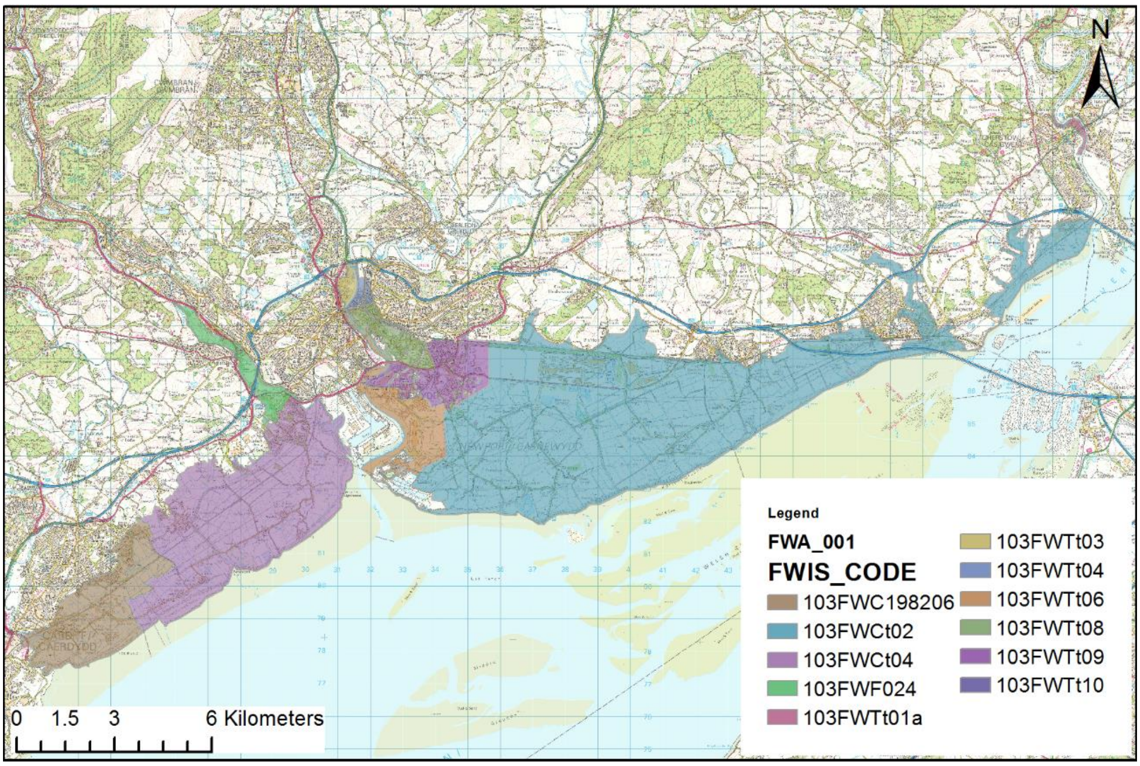


Figure 10-1: Existing Flood Warning Areas

The Wentlooge and Caldicot models were constructed to map the coastal flood risk and due to the modelling approach for the tidal rivers, and the proximity to the model boundaries, it is recommended that the models are not used to update the Flood Warning Areas serving the tidal rivers. Table 10-2 details the FWAs that were updated.

Table 10-2: Flood Warning codes and coverage

Flood Alert Code	Coverage
103FWC198206	Coast at the Wentlooge Levels in the Cardiff Area
103FWCt04	Coast at the Wentlooge Levels in the Newport Area
103FWCt02	Coast at the Caldicot Levels
103FWTt10	Usk Estuary at Pill
103FWTt08	Usk Estuary at Maindee, North Liswerry and Spytty Pill
103FWTt09	Usk Estuary at South Liswerry
103FWTt06	Usk Estuary at Uskmouth and Old Town Docks

10.2 Flood Alert and Flood Warning thresholds identified

The existing Flood Alert and Flood Warning thresholds were analysed against observed water level data from Newport to check on the frequency of exceedance over the period of the observed data, from 1993-2014. There are two sets of thresholds, one for low or no winds and one for high winds from the south west. For the still water thresholds, not taking account of winds the results are presented in Table 10-2.

Table 10-3: Existing Flood Warning thresholds and number of threshold crossings, (the numbers in brackets represent the threshold crossing if the levels are adjusted to account for the sea-level change along the coast)

Level at Newport	Level at Site	FWA code	Target Area	Message Set	Frequency	Average per year
7.2	8.2	Act FALwyeest	Wye Estuary	Flood Alert	177 (83)	8.3 (4)
7.2	n/a	Act FALuskest	Usk Estuary	Flood Alert	177	8.3
7.5	8.5	Act FWt01a	Wye Estuary at Chepstow	Flood Warning	39 (26)	1.8 (1.2)
7.5	n/a	Act FALcoastse	Coast from Aberthaw to Severn Bridge	Flood Alert	39	1.8
7.7	7.55	Act FWt03	Usk Estuary at Crindau and Malpas Road Area	Flood Warning	13 (30)	0.6 (1.5)
7.7	n/a	Act FWt06	Usk Estuary at Uskmouth & Old Town Docks	Flood Warning	13	0.6
8	8.00	Act FWt02	Coast at the Caldicot Levels	Flood Warning	1	0.05
8	8.00	Act FWt04	Coast at the Wentlooge Levels in the Newport Area	Flood Warning	1	0.05
8	7.83	ACT FW198206	Coast at the Wentlooge Levels in the Cardiff Area	Flood Warning	1 (4)	0.05 (0.2)
8.2	8.15	Act FWt04	Usk Estuary at Riverside	Flood Warning	0	0
8.2	8.05	Act FWt07	Usk Estuary at St Julians	Flood Warning	0	0
8.2	8.14	Act FWt08	Usk Estuary at Maindee, North Liswerry & Spytty Pill	Flood Warning	0	0
8.3	8.25	Act FWt09	Usk Estuary at South Liswerry	Flood Warning	0	0
8.3	9.3	actcon SFWt01a	Wye Estuary at Chepstow	Severe Flood Warning	0	0
8.3	8.3	actcon SFW t02	Coast at the Caldicot Levels	Severe Flood Warning	0	0

Level at Newport	Level at Site	FWA code	Target Area	Message Set	Frequency	Average per year
8.3	8.3	ActCon SFWt06	Usk Estuary at Uskmouth & Old Town Docks	Severe Flood Warning	0	0
8.3	8.15	ActCon SFWt03	Usk Estuary at Crindau and Malpas Road Area	Severe Flood Warning	0	0
8.5	8.44	Act FWt10	Usk Estuary at Pill	Flood Warning	0	0
8.5	8.45	ActCon SFWt04	Usk Estuary at Riverside	Severe Flood Warning	0	0
8.8	8.8	ActCON SFWt04	Coast at the Wentlooge Levels in the Newport Area	Severe Flood Warning	0	0
8.8	8.7	ACTCON SFW198206	Coast at the Wentlooge Levels in the Cardiff Area	Severe Flood Warning	0	0
9	8.85	ActCon SFWt07	Usk Estuary at St Julians	Severe Flood Warning	0	0
9	8.94	ActCon SFWt08	Usk Estuary at Maindee, North Liswerry & Spytty Pill	Severe Flood Warning	0	0
9.1	9.05	ActCon SFWt09	Usk Estuary at South Liswerry	Severe Flood Warning	0	0
9.3	9.24	ActCon SFWt10	Usk Estuary at Pill	Severe Flood Warning	0	0

The results from Table 10-2 show that over the 21-year period, the threshold for the issue of a Flood Alert ranges from twice per year on the coast, to eight times per year on the Usk and Wye. If the thresholds are adjusted to take account for the change in sea-levels along the coast, this drops to four times per year for the River Wye. The threshold exceedances for the issue of a Flood Warning are much less common with the average number of threshold crossings being twice per year for the Wye, once every two years for the Usk at Uskmouth and at Crindau and once in 20-years for the Rhymney and the open coast for the Caldicot and Wentlooge Levels.

When the additional criteria for wind speed and direction are taken into account this reduces to down to no zero. The Flood Warning Decision Sheet has lower sea-level thresholds for stormy weather when the wind speeds are greater than 17.1m/s from the south west (between 200-250 degrees). Using wind speeds from WWIII point 625, close to Flat Holm Island, the data was analysed to find periods where the wind conditions exceeded the criteria. There were only five days where the wind was higher than 17.1m/s and none of these were when the winds came from between 200-250 degrees, as summarised in Table 10-3.

Table 10-4: Met Office WWIII point 625 peak wind speeds

Date	Wind speed (m/s)	Wind direction (degrees)
09/12/1993 00:00	17.01	268.65
27/10/2002 12:00	19.64	273.79
13/01/2004 06:00	18.19	256.65
30/12/2006 17:00	19.22	257.99
09/12/2007 09:00	18.30	258.01

11 Conclusions

This project was commissioned by Natural Resources Wales (NRW) to improve understanding of the coastal flood risk between Cardiff and Chepstow in South East Wales. The main objectives of the study were to improve understanding of coastal flood risk using revised sea-levels and advanced multivariate joint probability assessment. A suite of models were used to transform the offshore multivariate conditions into the nearshore, where a detailed wave overtopping assessment was undertaken and two flood inundation models used to map the resulting coastal flood risk. A range of different scenarios was simulated, which included:

- Present day flood risk impacts for "With Defence" and "No Defence" scenarios for a range of return period events.
- The "With Defence" models were simulated with wave overtopping and without wave overtopping to assess the combined coastal flood risk and the still water flood risk.
- The potential impacts of climate change were modelled for the 200-year and 1,000-year return periods.
- The potential failure of the coastal defences was assessed using the RELIABLE software tool to generate fragility curves for a selection of representative defences.
- Breaches through the coastal defences were modelled using the EMBREA modelling software. The breach widths and depths determined from the EMBREA modelling were transferred into the TUFLOW flood inundation models to map the breached flood extents.
- The potential impacts of tidal gate failure for 15 outfalls was assessed by leaving the flapped tidal outfalls open throughout a coastal flood event.
- The potential impacts of a fluvial flood combined with the tidal outfalls failing to open was modelled for the same 15 outfalls.

The outputs from the model simulations were used to update the Flood Zones, ABDs, Flood Alert and Flood Warning Areas. A series of visualisation tools were created to present the results of the model simulations and allow the impacts of a forecast flood event to be visualised. The visualisation tools were created using GeoPDFs. The GeoPDFs contain a database of the model results from the 10,000-year event set; for any forecast conditions the resulting wave overtopping flood risk can be extracted from the database and the closest matching flood extents are shown on the maps, together with information on the numbers of properties and critical infrastructure at risk.

11.1.1 Impact of change in sea-levels

The incorporation of additional years of observed tide gauge data to update the extreme sea-levels, resulted in very similar results for Newport and Avonmouth, when compared to the 2011 CFBD study, but increased levels at Mumbles, of 0.1m at the 200-year and 0.2m at the 1,000-year return periods. The se-levels between Newport and Mumbles are interpolated resulting in gradually increasing levels towards Mumbles. For Cardiff, where there is a known low spot in the defences, the levels are 0.1-0.2m higher for the 200-1,000-year events. All future defence upgrades should use the updated sea-levels.

11.1.2 New flood map components

The new tidal Flood Zones derived for this study are similar to the previous Flood Zones, with the topography dictating the spread of the flood extents. The property counts for the revised and existing Flood Zones are summarised in Table 11-1.

Table 11-1: Property inundation for Flood Zones

Model scenario	Residential properties (number)	Commercial properties (number)	Unclassified (number)	Total (number)
Caldicot				
FZ3 existing	10,692	1,357	6	12,055
FZ3 new	9,654	1,307	3	10,964
FZ2 existing	11,545	1,440	7	12,992
FZ2 new	10,895	1,403	7	12,305
Wentlooge				
FZ3 existing	5,756	632	0	6,388
FZ3 new	5,745	644	0	6,389
FZ2 existing	7,045	1,329	0	8,374
FZ2 new	7,138	904	0	8,042

11.1.3 Impact of Climate Change

Climate change simulations were undertaken for the 200-year and 1,000-year events. These simulations represent the potential increase in flood risk up to the year 2115 based on the WG guidance for sea-level rise estimates (FCDPAG3) which equates to a mean sea level rise of just over a metre. These simulations show the current defence structures to be highly vulnerable to the increased risk of flooding due to climate change, becoming heavily inundated as a result of increased wave overtopping. For the majority of defences the amount of wave overtopping increases by over 100% during climate change scenarios. In the Wentlooge model 926 properties are flooded for the present day 200-year event which increases to over 6,500 in a climate change scenario, and in the Caldicot Levels the number of properties increases from 118 to over 11,000. Table 11-2 shows the number of flooded properties during climate change scenarios for both models compared against the 200 and 1,000-year event property inundation.

Table 11-2: Property inundation for present day and climate change simulations

Event (yr)	Flooded properties (Defended)	Model
200	926	Wentlooge
1,000	1,770	Wentlooge
200CC (2115)	6,502	Wentlooge
1,000CC (2115)	7,933	Wentlooge
200	118	Caldicot
1,000	2,497	Caldicot
200CC (2115)	11,701	Caldicot
1,000CC (2115)	13,468	Caldicot

11.1.4 Impact on critical infrastructure

The first critical infrastructure to flood in the Wentlooge model is on the tidal river, the Rhmney. During a 5-year event on the Rhymney the railway line south of Rumney next to the Parc Tredelerch is at risk. The first critical infrastructure to flood in the Caldicot model is the pumping station at Porton Ho during a 5-year event. As the magnitude of the events increase the amount of critical infrastructure at risk increases. Table 11-3 summarises the amount of critical infrastructure at risk.

Table 11-3: Critical infrastructure at risk

Event (yr)	Number of critical infrastructure at risk	Including	Model
200	14	A4232, railway, pumping stations and telecommunication	Wentlooge
1,000	27	A48, A4232, railway, pumping stations and telecommunication	Wentlooge
200CC (2115)	58	A48, A4232, railway, pumping stations and telecommunication	Wentlooge
1,000CC (2115)	76	A48, A4232, 3 sections of railway, power station, pumping stations and telecommunication	Wentlooge
200	9	M4, 2 sections of railway, power station, pumping stations and telecommunication	Caldicot
1,000	26	A48, M4, 3 sections of railway, power station, pumping stations and telecommunication	Caldicot
200CC (2115)	99	A48, M48, M4, 7 sections of railway, Caldicot train station, power station, pumping stations, sewage works, gas works and telecommunication	Caldicot
1,000CC (2115)	110	A48, M48, M4, 8 sections of railway, Caldicot train station, power station, pumping stations, sewage works, gas works and telecommunication	Caldicot
Note: Counts for critical infrastructure include motorways, A-roads, railways, stations, water and sewage treatment works, pumping stations, telecommunication, power stations, police stations, ambulance stations, emergency services, central government service, fire station, local government service, hospital, medical centres, chemical works, schools, universities, places of worship, dentist and petrol station.			

11.1.5 Impact on coastal communities

The coastal communities at flood risk during present day "With Defences" scenarios are the Rumney area of Cardiff, Peterstone, Liswerry and Uskmouth in Newport, parts of Whitson, Goldcliff, Summerleaze, Cadlicot, Portskewett, Sudbrook, and Chepstow. There are also many individual farms and properties at flood risk throughout the Caldicot and Wentlooge Levels.

When the impacts of climate change are introduced additional communities are at risk. On the Caldicot Levels these include Rogiet, Magor, Undy, Redwick, Greenmoor, Bishton, Wilcrick, Llanwern and Somerton. On the Wentlooge Levels these include Duffryn, Marshfield, St Brides, Lighthouse Park, St Mellons and Trowbridge.

When the defences are removed for the "No Defences" the whole of the Wentlooge and Caldicot Levels are inundated. All the communities inundated within a "With Defence" scenarios remain affected, but to a greater depth and extent.

11.1.6 Impact of defence breaches

The Wentlooge Level defences provide a high SoP and the flood risk is limited to tidal flooding on the Rhymney at Rumney and areas of wave overtopping along the coastal frontage at Newton Farm, Peterstone Gout and Orchard Farm. When defence breaches are simulated the resulting flooding increases with approximately 100 additional properties at risk under a present day event. When the impacts of climate change are included over 1,000 additional properties are at risk during a breach scenario compared to the "With Defence" 1,000-year event.

The Caldicot Levels are more susceptible to flooding during the "With Defence" scenarios and the inclusion of breaches increases the number of properties at risk by approximately 150 during both the present day and climate change scenarios.

11.1.7 Impact of outfall failure

There are very few properties at risk from the outfall failure scenarios. Outfall failure was assessed with outfalls failing to close, allowing tidal ingression and separately with outfalls failing to open and locking in fluvial flows. Most failures, whether during tidal or fluvial events, result in the flooding of one to two properties. On the Wentlooge Levels all failure open and closed scenarios result in a maximum of one property flooding.

On the Caldicot Levels, failure of the outfalls to open, trapping in fluvial flows, at Fisher's Gout and Monk's Ditch result in 8 and 22 properties flooding around Goldcliff. Failure of the outfall at Magor Pill floods nine properties and failure at Caldicot Pill, 5 properties. Failure of the outfall to close, allowing tidal ingression results in flooding of 13 properties from Fisher's Gout and Monk's Ditch but only a maximum of one property from a failure at any of the other outfalls.

11.1.8 Standard of Protection

The SoP provided by the defences was calculated for all of the modelled defences. Mean overtopping discharge rates were used to determine the SoP against a range of thresholds from the Eurotop manual and against the acceptable limits of overtopping from the Severn Estuary Strategy. The target SoP for the defences in the estuary, from the Strategy Study, is protection during a 1 in 1,000-year event in 2010. Acceptable limits of overtopping for grassed embankments and wave return walls were compared against the modelled results. On the Wentlooge Levels only defence 6, near Peterstone Gout, has a SoP less than 1 in 1,000 for a grassed embankment. On the Caldicot Levels there are several sections with a SoP less than 1 in 1,000. For grassed embankments these are defences 21, 22, 23, 25, 27, 28, 29, 30 and 31. The majority of these defences also have a SoP less than 1 in 200-years, with defence 23 having the lowest SoP of 1 in 20-years. For defences with wave return walls, all of the overtopping discharges are lower than the acceptable discharge of $0.2m^3/s/m$.

11.2 Limitations

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

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

Other assumptions include:

- The tidal graphs were created by aligning the peak of the surge profile with the low point in the trough prior to the peak tide. The [surge magnitude] was then modified to ensure the peak water levels match the derived extreme sea levels from the Improved Coastal Flood Boundary (CFB) Conditions for the UK mainland and islands report.
- The overtopping discharges assume that the wind and wave conditions remain constant throughout the duration of the tidal event.
- Defence crest data used in both the overtopping calculations and the flood inundation models are based on topographic survey where available, or alternatively AIMS and point heights extracted from LIDAR data.
- For the wave overtopping calculations, the flood defence profiles, toe and berm levels are adjusted as the magnitude of the events change throughout the modelled scenarios.
- The model grid size of 10m for the Caldicot model is relatively coarse when it comes to representing small flow paths and small drainage channels. Due to the limitations of present day computing this is a trade-off between model accuracy and manageable run times. All efforts have been made to minimise the impacts of this through modelling techniques such as the use of 1D structures to model flow through floodplain

embankments taking account of the channel dimensions rather than stamping holes through the structures equal to the modelled grid resolution.

- The main river and the main drainage channel networks have been modelled through the use of gully lines which may not always represent the channel conveyance accurately. There are many narrow reens which have not been directly modelled.
- Infiltration losses into the ground within the flood inundation model have not been considered.
- The tidal rivers have not been directly modelled in the 2D inundation model. The water level increases up the channels were taken from existing models. Given the size of the area, it was not possible to create a single model with high resolution, including all low lying areas and the tidal rivers.
- No account has been taken of the joint flood risk from tidal, fluvial, surface water and groundwater events, only coastal flooding from tides and waves have been modelled.
- The rates of sea-level rise used in the climate change calculations are based on the FCDPAG3 average figures for the south west and Wales.

Overall, the work undertaken to update the Flood Map should provide users with a map that can be applied with greater confidence than previous versions. In light of the limitations highlighted above there are a number of recommendations for future work and updates which could be undertaken to improve confidence in the modelling.

11.3 Recommendations

It is recommended that the modelling is periodically revisited, particularly following large flood events. Significant new event data may alter the range of extreme values in the statistical analysis and may also provide evidence to validate the performance of the coastal models, for the wave transformation, wave overtopping and flood inundation models.

As computer processing power increases it is recommended that consideration is given to the creation of a single high resolution model covering the Wentlooge and Caldicot Levels, which incorporate the tidal rivers and enable the modelling of the change in water levels up the channels.

If new flood alleviation schemes are built, the model should be updated to account of the new defences.

The results of this study should not be used for design purposes and should a flood risk assessment be required for a specific location within the modelled domain a separate investigation should be undertaken to investigate the specific risks and considerations for each site.

11.4 Conclusions

This study has used the most up to date methods and data and has improved the confidence in the mapping of flood risk in the Caldicot and Wentlooge flood cells.

New visualisations have been created in terms of animations and interactive GeoPDFs for use in incident management.

The study has highlighted that the area between Sudbrook and Chapel Farm is defended to a lower standard of protection than was identified in the Severn Estuary Flood Risk Management Strategy.

Appendices

A Updated extreme sea-levels

A.1 Introduction

As part of the Caldicot and Wentlooge Coastal Modelling Study we have investigated the impact of updating the sea-levels in the CFB dataset from the current base year of 2008 to 2014. The principal driver for the investigation is the availability of new data incorporating significant additional tidal event peaks.

In February 2011, the Environment Agency's Evidence Directorate published the study *Improved Coastal Flood Boundary (CFB)*[47] *Conditions for the UK mainland and islands*. This project, which resulted in the derivation of new extreme sea-level estimates for the whole of the UK, was undertaken as part of the joint Environment Agency/Defra Flood and Coastal Erosion Risk Management Research and Development Programme (SC060064). The CFB data study produced sea-level estimates for a range of return periods at 2km intervals around the coast based on tide gauge data up to the end of 2008 and numerical modelling.

Large coastal storm events in December 2013, January and February 2014 resulted in the highest ever recorded sea-levels at many locations around the UK, including sites where the sea-levels from the 1953 event were exceeded. In addition to these recent large events, there is also an additional five years of tide gauge data available which can be used to add to the datasets used in the original CFB data study and enable the calculation of revised extremes.

A.2 Data used in the analysis

To revise the extreme sea-level estimates, the first step was to collect the surrounding tide gauge data from the local sites. These data spanned the period 2009 to August 2014. Data from the three Class A tide gauge sites at Newport, Mumbles and Avonmouth were used. The Class A tide gauges are a network of 43 gauges which are owned by the Environment Agency and maintained by the Tide Gauge Inspectorate at the NOC. The data are processed and quality checked and provide the most accurate and reliable source of sea-level data around the UK. The use of all three gauges allowed for consistency checks between the calculated sea-levels and is in line with the original CFB study within which the aim was to produce a spatially consistent set of sea-levels around the country.

The data used in the original analysis was quality checked to identify erroneous peaks or datum shifts within the data. These quality checks were applied to the additional data and any erroneous data was removed from the records. The erroneous peaks were given a -99 null value flag, these values are not used in the calculations but do show up as low values in the plots below. An example of some data from the Mumbles gauge in 2013 is shown in Figure A. 11-1 with the corrected data shown in Figure A. 11-2 to remove the spike.

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

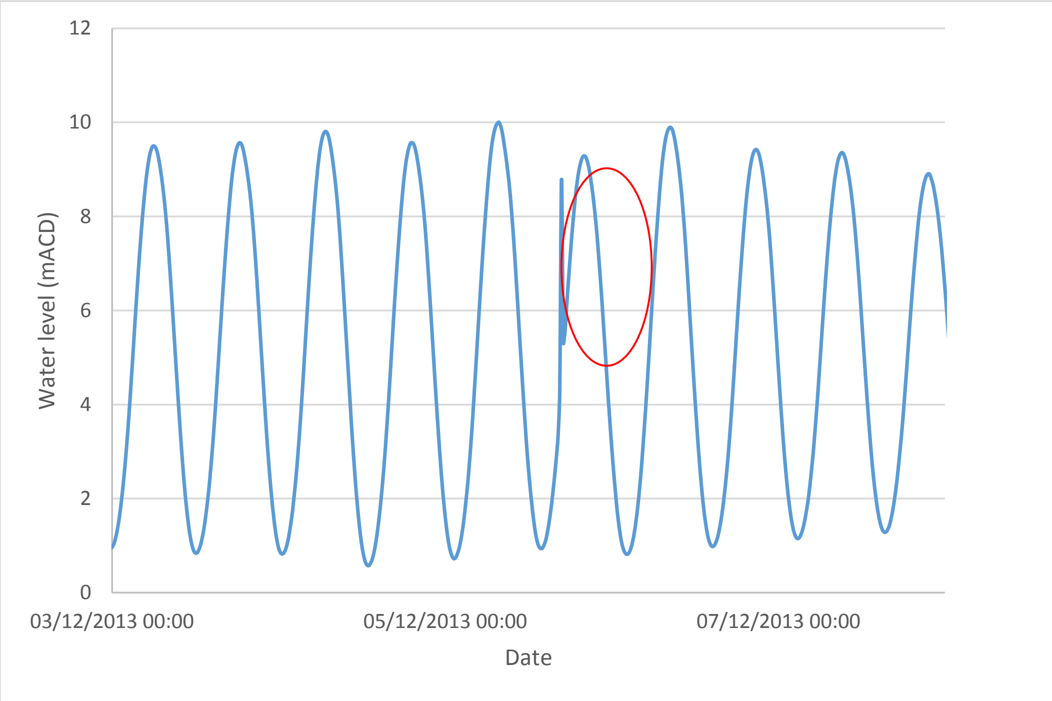


Figure A. 11-1: Mumbles 2013 tide levels with erroneous data

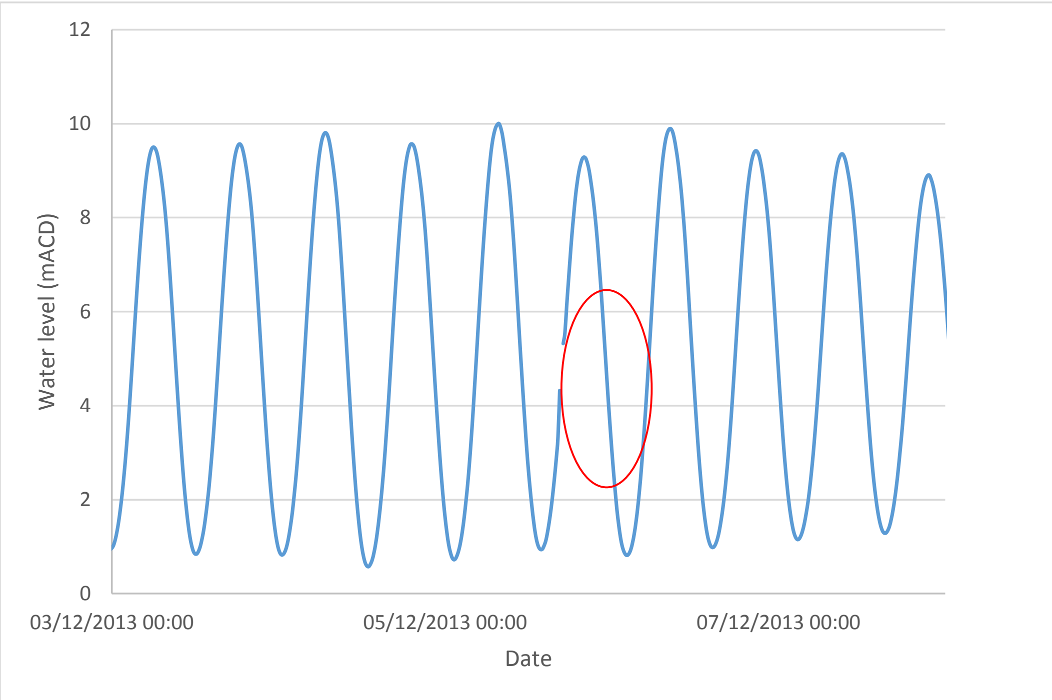


Figure A. 11-2: Mumbles 2013 tide levels with erroneous data removed

A.3 Statistical analysis

The data from 2009-2014 were appended to the existing data used in the CFB data study to provide a continuous time series of level data at each site. The information in Table A. 11-1 summarises the record lengths of the data used in the analysis.

Table A. 11-1: Data used for sea-level updates

Tide gauge name	Record length
Newport	1993 - present
Mumbles	1988 - present
Avonmouth	1961 – 2012

Using the methods employed for the original CFB data study, statistical analysis was performed to generate probabilities of predicted high tide and of skew surge. These were combined to determine the overall extreme sea-level probabilities using the Skew Surge Joint Probability Method (SSJPM).

A.4 Skew surge

The approach used to derive extreme sea-levels around the coastline is the SSJPM.

Surge can be caused by a number of factors including atmospheric pressure variations, wind acting along the surface of the water and the Coriolis Effect. This can result in an increase or decrease in sea-level. As meteorological processes are independent of the astronomical tidal process their influence can occur at any stage of the tide. The peaks in the predicted astronomical tide and recorded total water level will therefore not always coincide. Skew surge is the difference between the two peak values, independent of whether the timing coincides, as demonstrated in Figure A. 11-3.

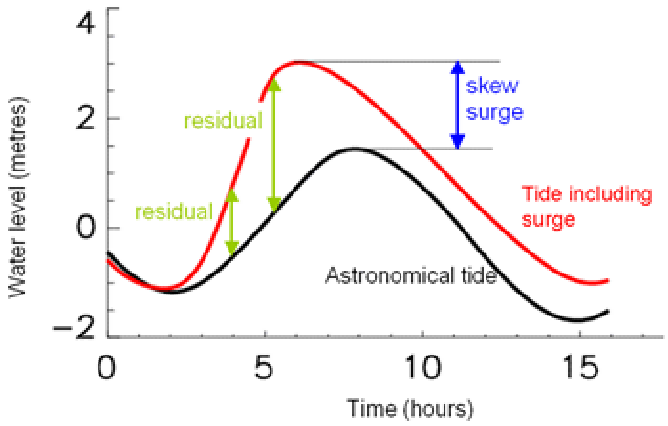


Figure A. 11-3: Illustration of the skew surge

A.5 Application of SSJPM

The tide gauge sites used in this study were analysed using the SSJPM. All data were de-trended using historical sea-level rise data to a base year of 2014. This is erroneously reported in the CFB data study as being equivalent to 2mm per year but this does vary at each tide gauge and is based on analysis provided by NOC. The skew surge analysis was applied for all high water events for each year of data.

The results of this analysis enabled the production of frequency histograms for each site. The probability density function is derived from the histograms by dividing by the total number of observations as demonstrated in Figure A. 11-4.

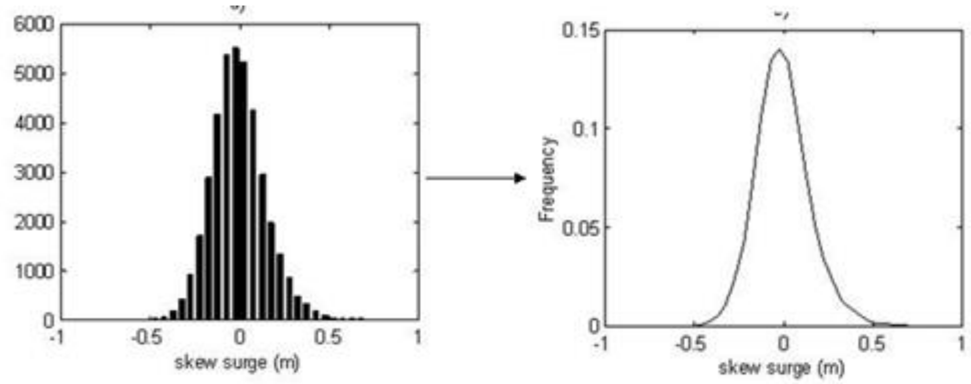


Figure A. 11-4: Smoothing the Skew Surge Histogram

Due to the rarity of extreme events, the tails of the probability density function can be poorly defined. A statistical model (the Generalised Pareto Distribution (GPD)) is used to fit a smooth upper tail to the probability density function. This is illustrated by the schematic presented in Figure A. 11-5.

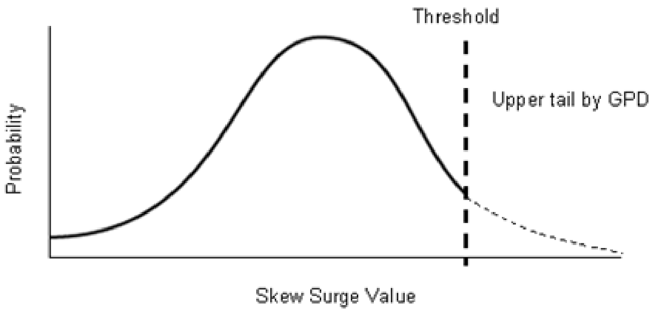


Figure A. 11-5: Schematic of the Generalised Pareto Statistical Distribution

Astronomical tides are deterministic and have known absolute maxima. The distribution of peak tide levels therefore exhibits well-resolved tails and there is no requirement to fit a GPD. At each gauge, the lunar nodal cycle of high tides was derived from harmonic constituents. This cycle has an approximate period of 18.6 years, caused by the precession of the plane of the lunar orbit. Changes by this variation in lunar declination can alter the range of the tide by ±3.7 per cent when the declination amplitudes are greatest. In 2015 the astronomical tides reach the peak of the nodal cycle and high tidal events are predicted through the year. Only one nodal cycle is used in the analysis at each gauging station, therefore if 85 years of recorded data is available at a site, only one 18.6 year portion of the data is used to create the probability distribution function.

Joint probability analysis was used to form a probability distribution of all possible total sea-levels from the skew surge distribution (with GPD tail fit) and peak tide levels from the full nodal cycle. The joint probability analysis assumed independence between skew surge and peak tide levels.

The duration of storm surges can encompass multiple high tides. This means that there may be a degree of dependence between extreme skew surge levels in the tide gauge record, for example, two extreme skew surge level observations may have occurred during the same storm. A correction factor was derived to account for this dependence in the calculation of return period.

The final stage of the SSJPM method was expression of the probability distribution of total sea-levels in terms of return periods. The results from the analysis are described in the following chapter.

A.6 Interpolation of levels between gauged locations

The statistical analysis provided estimates of return period sea-levels at the Class A and other supplementary primary sites. In order to extend the coverage between these sites, it was necessary to interpolate using results from these primary sites as the corrector. In the original study a coastal chainage line was created running clockwise around the country with an origin point at Newlyn. The trendline was set at a distance offshore and chainage points were created at 2km intervals. Numerical models were then used as a dynamic interpolator to interpolate values along the chainage line with the growth to higher return periods assumed to vary linearly between the gauged locations. It was not within the remit of this study to re-create the dynamic interpolation, therefore, the same percentage increment adjustments were applied to the revised sea-level estimates for the intermediate points along the same chainage line.

A.7 Results

In this section we present a number of interpretations of the results to find the bounds of the data. The results from the analysis for the four gauged locations used to produce the revised sea-levels are summarised in Table A. 11-2 to Table A. 11-5.

A.7.1 Newport

Newport is located on the South Wales coast and has 21-years of recorded sea-level data. Within the 21-years of data the event of 2014 was the highest sea-level recorded at the gauge. The results in Table A. 11-2 show that the updated levels have increased for most return periods. The levels are 0.03m higher for the 2-year return period and are the same for the 200-year return period. For the 200-year return period the 95% confidence intervals are ±0.40m at Newport, therefore the changes are well within the uncertainty bands.

The additional data from the recent events are generally small at around the 1-year return period apart from 2014 which was equivalent to a 22-year return period when compared to both the 2008 levels and the updated levels.

Table A. 11-2: Return period results for Newport

Return period (years)	Original 2008 CFB levels (mAOD)	Updated 2014 CFB levels (mAOD)	Difference (m)
1	7.54	7.57	0.03
2	7.64	7.67	0.03
5	7.78	7.80	0.02
10	7.89	7.90	0.01
20	8.00	8.01	0.01
25	8.04	8.05	0.01
50	8.16	8.16	0.00
75	8.23	8.23	0.00
100	8.28	8.28	0.00
150	8.35	8.36	0.01
200	8.41	8.41	0.00
250	8.45	8.45	0.00
300	8.48	8.49	0.01
500	8.58	8.59	0.01
1,000	8.72	8.73	0.01
10,000	9.22	9.28	0.06

Table A. 11-3: Return period 95% confidence intervals for Newport

Return period (years)	Original 2008 CFB confidence bounds (m)	Updated 2014 CFB confidence bounds (m)	Original 2008 CFB confidence intervals (m)	Updated 2014 CFB confidence intervals (m)	Difference (m)
1	0.02	0.02	0.0	0.0	0.0
2	0.04	0.03	0.0	0.0	0.0
5	0.06	0.05	0.0	0.0	0.0
10	0.09	0.07	0.0	0.0	0.0
20	0.13	0.11	0.1	0.1	0.0
25	0.15	0.13	0.1	0.1	0.0
50	0.24	0.20	0.1	0.1	0.0
75	0.28	0.26	0.1	0.1	0.0
100	0.33	0.30	0.2	0.2	0.0
150	0.39	0.37	0.2	0.2	0.0
200	0.45	0.43	0.2	0.2	0.0
250	0.51	0.47	0.3	0.2	0.0
300	0.56	0.51	0.3	0.3	0.0
500	0.73	0.62	0.4	0.3	-0.1
1,000	0.99	0.83	0.5	0.4	-0.1
10,000	2.25	1.79	1.1	0.9	-0.2

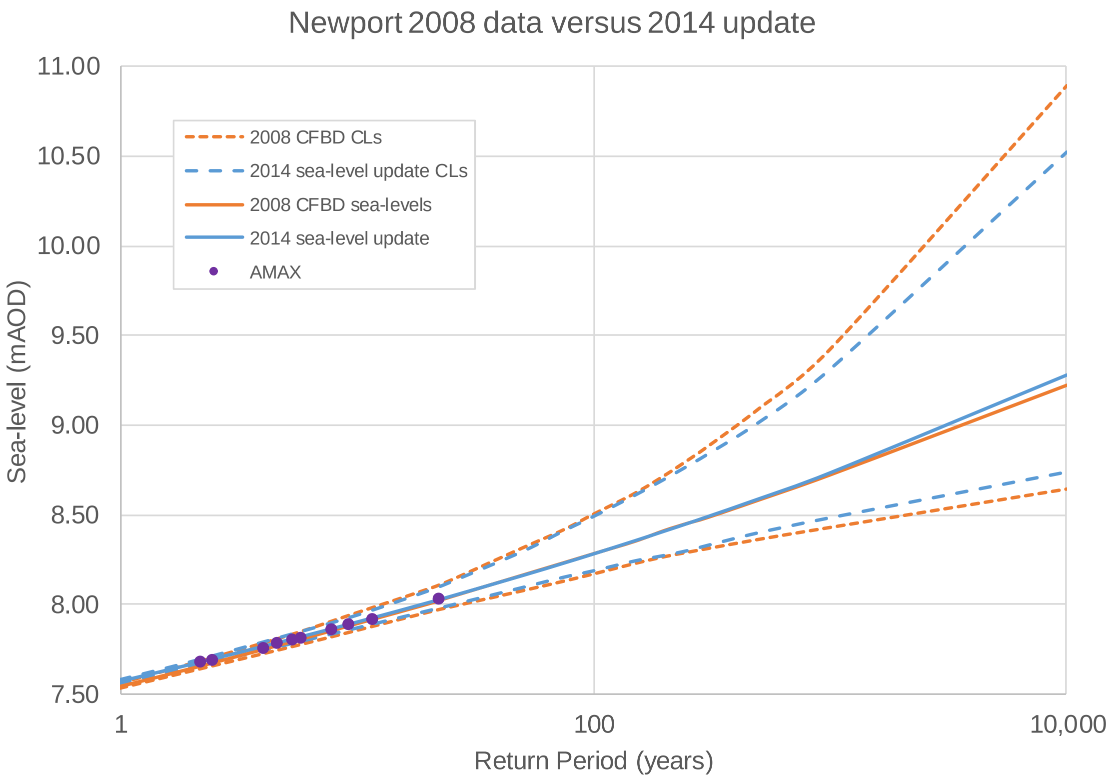


Figure A. 11-6: Newport 2008 and 2014 sea-level return periods

A.7.2 Mumbles

Mumbles is the furthest west of the gauges and is used in the interpolation of levels along the Wentlooge frontage Wentlooge frontage between Newport and Mumbles. The gauge has a record spanning 26-years back to back to 1988. The event of February 2014 is ranked as the largest event to be recorded in the gauged gauged record. In relation to the return period estimates, the 2014 event was approximately a 1 in 10-year in 10-year event against both the updated and previous sea-levels. The levels using the additional years of years of tidal data are summarised in

Table A. 11-3. The updated levels similar to those of the previous analysis at low return periods but increase to higher values for the higher return periods. The difference for the 200-year event is 0.11m. For the 200-year return period the 95% confidence intervals are ±0.30m at Mumbles, therefore the changes are well within the uncertainty bands.

Table A. 11-4: Return period results for Mumbles

Return period (years)	Original 2008 CFB levels (mAOD)	Updated 2014 CFB levels (mAOD)	Difference (m)
1	5.47	5.47	0.01
2	5.54	5.55	0.00
5	5.65	5.66	0.00
10	5.74	5.75	0.01
20	5.83	5.85	0.02
25	5.86	5.88	0.02
50	5.95	5.99	0.04
75	6.01	6.06	0.06
100	6.05	6.12	0.07
150	6.11	6.20	0.09
200	6.15	6.26	0.11
250	6.18	6.30	0.12
300	6.21	6.34	0.14
500	6.28	6.46	0.18
1000	6.39	6.63	0.24
10000	6.77	7.34	0.58

Table A. 11-5: Return period 95% confidence intervals for Mumbles

Return period (years)	Original 2008 CFB confidence bounds (m)	Updated 2014 CFB confidence bounds (m)	Original 2008 CFB confidence intervals (m)	Updated 2014 CFB confidence intervals (m)	Difference (m)
1	0.02	0.01	0.1	0.1	0.0
2	0.03	0.02	0.1	0.1	0.0
5	0.05	0.05	0.1	0.1	0.0
10	0.08	0.09	0.1	0.1	0.0
20	0.10	0.15	0.1	0.1	0.0
25	0.11	0.18	0.1	0.1	0.0
50	0.19	0.29	0.1	0.1	0.0
75	0.27	0.38	0.2	0.2	0.0
100	0.33	0.46	0.2	0.2	0.0
150	0.41	0.59	0.3	0.3	0.0
200	0.48	0.70	0.3	0.4	0.1
250	0.53	0.79	0.3	0.4	0.1
300	0.57	0.87	0.3	0.4	0.1
500	0.72	1.12	0.4	0.6	0.2
1,000	0.94	1.51	0.5	0.8	0.3
10,000	1.96	3.49	1.0	1.7	0.8

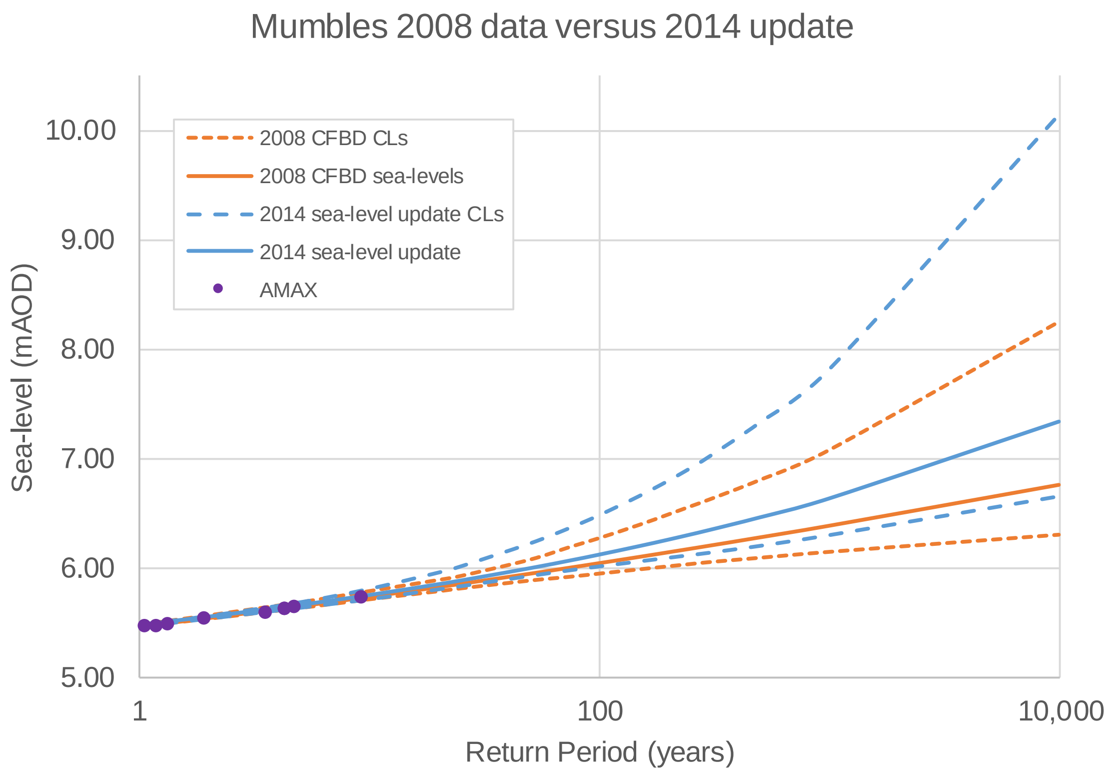


Figure A. 11-7: Mumbles 2008 and 2014 sea-level return periods

A.7.3 Avonmouth

Avonmouth is on the English side of the Severn Estuary. The Avonmouth gauge was closed in 2012 and has been replaced by a gauge further west at Portbury. There are only four additional years of data to add at this site and the events of 2013 and 2014 are not included. The four additional years of data included at this site rank as 17th, 22nd, 30th and 35th out of the 40-years of available data. Inclusion of the smaller events has resulted in a reduction in the estimates of the extremes for most events and a slight increase at the 10,000-year return period. At the 200-year event the new estimates are -0.02m lower than previous. For the 200-year return period the 95% confidence intervals are ±0.40m at Avonmouth, therefore the changes are within the uncertainty bands. The results are summarised in Table A. 11-4 and the growth curves illustrated in Figure A. 11-8.

Table A. 11-6: Return period results for Avonmouth

Return period (years)	Original 2008 CFB levels (mAOD)	Updated 2014 CFB levels (mAOD)	Difference (m)
1	8.16	8.16	0.00
2	8.27	8.27	-0.01
5	8.43	8.41	-0.01
10	8.55	8.53	-0.02
20	8.67	8.66	-0.02
25	8.72	8.70	-0.02
50	8.85	8.83	-0.02
75	8.92	8.90	-0.02
100	8.98	8.96	-0.02
150	9.06	9.04	-0.02
200	9.11	9.10	-0.02
250	9.16	9.14	-0.01
300	9.19	9.18	-0.01
500	9.29	9.28	-0.01
1,000	9.43	9.43	0.00
10,000	9.89	9.91	0.03

Table A. 11-7: Return period 95% confidence intervals for Avonmouth

Return period (years)	Original 2008 CFB confidence bounds (m)	Updated 2014 CFB confidence bounds (m)	Original 2008 CFB confidence intervals (m)	Updated 2014 CFB confidence intervals (m)	Difference (m)
1	0.01	0.01	0.0	0.0	0.0
2	0.03	0.03	0.0	0.0	0.0
5	0.05	0.05	0.0	0.0	0.0
10	0.07	0.08	0.0	0.0	0.0
20	0.10	0.11	0.1	0.1	0.0
25	0.12	0.13	0.1	0.1	0.0
50	0.16	0.19	0.1	0.1	0.0
75	0.22	0.24	0.1	0.1	0.0
100	0.25	0.28	0.1	0.1	0.0
150	0.30	0.36	0.1	0.2	0.0
200	0.34	0.43	0.2	0.2	0.0
250	0.37	0.49	0.2	0.2	0.1
300	0.40	0.53	0.2	0.3	0.1
500	0.49	0.68	0.2	0.3	0.1
1,000	0.65	0.91	0.3	0.5	0.1
10,000	1.40	1.91	0.7	1.0	0.3

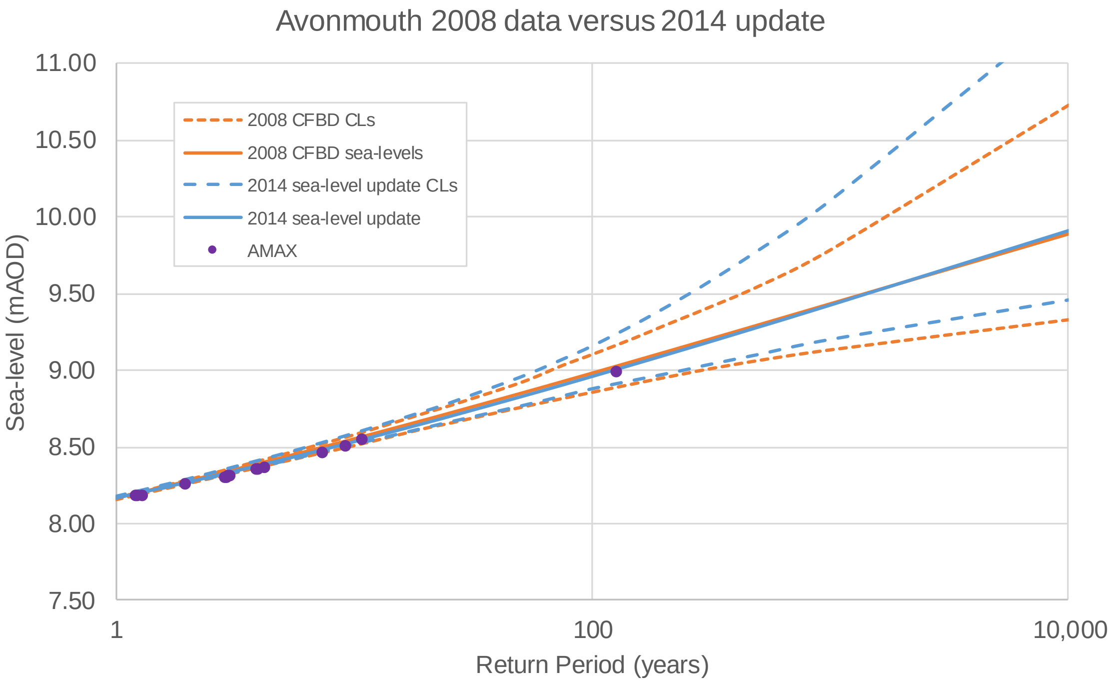


Figure A. 11-8: Avonmouth 2008 and 2014 sea-level return periods

A.8 Analysis of results

It can be seen from the results and the plots that the general trend is an increase in the return period sea-level estimates when the additional years of gauged data has been included. However, this increase is within the confidence limits of the return period estimates and so is not considered a significant change. It is worth noting that the extreme tail of the skew surge distribution was extrapolated as part of the previous estimates and the additional data from these recent events was predicted within the extrapolated zone. As a result, the skew surge distribution and return period estimates are not significantly affected by the inclusion of the more recent additional data. Indeed this analysis has shown the skew surge joint probability method for estimating extreme sea-levels is robust and the recent events were indeed extreme events resulting from both high tides and extreme surge.

By plotting the growth curves on a logarithmic scale for the three sites, it can be seen that Avonmouth and Newport are further up the estuary and have higher sea-levels but the growth rate at Mumbles is higher to the higher return periods (Figure A. 11-9).

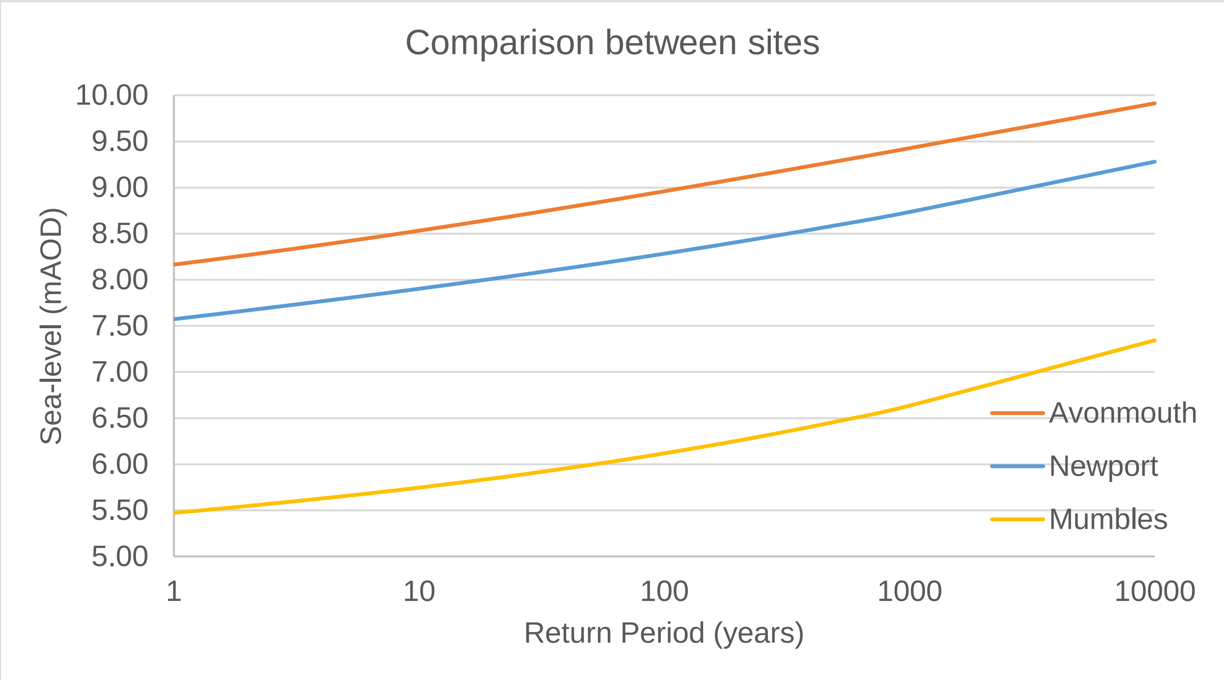


Figure A. 11-9: Comparison of 2014 sea-level return periods for all sites

A.9 Results

Based on the updated sea-levels for Newport, Mumbles and Avonmouth, the sea levels have been interpolated around the coast.

Table A. 11-8: Return periods at CFB locations around the coastline

CFB chainage number	5-year sea-level (mAOD)	200-year sea-level (mAOD)	1,000-year sea-level (mAOD)	Difference 2014 to 2008 CFB levels for 5yr (m)	Difference 2014 to 2008 CFB levels for 200yr (m)	Difference 2014 to 2008 CFB levels for 1,000yr (m)
380 (Avonmouth)	8.41	9.10	9.43	-0.02	-0.01	0.00
382	8.29	8.96	9.29	-0.01	-0.01	0.00
384	8.23	8.90	9.23	0.00	0.00	0.01
386	8.18	8.83	9.17	0.01	0.00	0.02
388	8.11	8.77	9.10	0.01	0.01	0.02
390	8.06	8.70	9.02	0.02	0.01	0.02
392	8.00	8.63	8.95	0.02	0.01	0.02
394	7.93	8.56	8.88	0.02	0.01	0.02
396 (Newport)	7.87	8.48	8.81	0.02	0.00	0.02
398	7.80	8.41	8.73	0.02	0.00	0.01
400	7.77	8.37	8.72	0.04	0.02	0.04
402	7.72	8.34	8.69	0.05	0.04	0.06
404	7.67	8.30	8.67	0.06	0.06	0.08
406	7.64	8.26	8.65	0.08	0.07	0.10
408	7.59	8.22	8.62	0.09	0.09	0.12
410	7.55	8.18	8.60	0.10	0.10	0.14
412	7.50	8.14	8.57	0.11	0.12	0.15
414 (Cardiff)	7.46	8.10	8.54	0.12	0.13	0.17

B Catchment descriptors for reen catchments

The key changes to the catchment descriptors were as follows:

- AREA – adjusted for the catchment draining to all outfalls. Changed the .csv and .cd3 files to represent the manually derived catchment area (i.e. the shapefile polygons in ArcGIS)
- DPLBAR (Mean drainage path length (km)) – ID 9, 10, 11 and 12. DPLBAR can be calculated using the FEH equation DPLBAR = $AREA^{0.538}$, where no other information is known. Alternatively, if you have a local catchment that is of a similar shape, you can calculate what the exponent would be from its AREA and DPLBAR values and then apply this exponent to the new catchment. DPLBAR was changed for ID 9, 10 and 11 using $AREA^{0.538}$ and ID 12 used the exponent from the original AREA and DPLBAR.
- FARL (Flood Attenuation by Reservoirs and Lakes) index was altered for IDs 12 and 14 as the FEH catchment boundary placed Wentwood reservoir in catchment 14, whilst the manually derived catchments placed it the catchment draining to outfall ID 12.

Table A. 11-9: Key catchment descriptors

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT 1990	URBEXT 2000	FPEXT
Reens										
1	1.000	0.47	0.712	0.67	21.6	1002	27.91	0.214	0.303	0.293
2	1.000	0.47	0.676	1.43	19.4	1006	32.03	0.110	0.180	0.479
3	1.000	0.47	0.666	1.82	19.9	1019	33.26	0.059	0.065	0.361
4	0.955	0.47	0.762	2.28	20.0	1018	25.76	0.007	0.005	0.350
5	1.000	0.47	0.734	1.48	5.7	968	25.30	0.000	0.000	0.474
6	1.000	0.47	0.754	2.46	11.8	1023	25.07	0.091	0.099	0.692
7	0.968	0.42	0.595	6.33	90.3	989	24.49	0.046	0.055	0.124
8	1.000	0.47	0.734	2.45	4.3	905	25.30	0.015	0.017	0.583
9*	0.782	0.44	0.667	2.42	20.5	921	29.10	0.178	0.310	0.701
10	1.000	0.35	0.734	2.93	2.6	865	25.30	0.022	0.008	0.936
11	1.000	0.35	0.726	1.60	2.8	862	25.33	0.000	0.000	0.742
12	0.987	0.35	0.713	8.79	83.7	964	21.79	0.019	0.026	0.139
13	1.000	0.35	0.694	2.18	33.0	880	27.54	0.039	0.064	0.185
14	1.000	0.35	0.731	9.74	87.7	997	20.07	0.014	0.019	0.057
15	0.988	0.35	0.745	8.67	89.4	1000	19.50	0.010	0.016	0.027
* catchment 9 is made up of 3 smaller sub catchments combined together										

Table A. 11-10: Flow volumes for the seven storm durations

Site code	Storm Duration (Hours) / Flow volume (m^3)						
	6.25	12.25	24.25	36.25	48.25	60.25	72.25
1	61,997	97,268	151,038	199,799	242,523	282,502	318,414
2	208,101	300,476	440,675	552,324	648,738	739,145	819,916
3	264,291	378,157	547,446	683,333	802,423	913,733	1,014,386
4	205,948	294,607	432,293	546,353	649,589	743,709	830,550
5	33,705	47,305	67,747	84,338	99,122	112,792	125,287
6	35,663	51,145	75,558	95,650	113,609	129,423	143,780
7	557,996	762,701	1,063,312	1,301,110	1,510,339	1,694,753	1,861,890
8	149,607	203,600	287,014	354,026	413,599	467,346	516,418
9	225,331	331,321	493,987	641,973	774,807	890,509	995,318
10	94,264	125,595	175,639	215,440	251,037	282,149	310,816
11	30,327	405,26	58,055	72,325	85,208	95,977	105,891
12	326,043	444,137	622,464	765,596	894,596	1,008,330	1,114,011
13	96,584	132,503	193,981	244,850	290,823	328,037	361,923
14	572,553	794,989	1,130,859	1,402,768	1,649,618	1,864,624	2,064,376
15	485,420	683,026	985,775	1,233,545	1,459,955	1,650,581	1,828,490

C Multivariate extreme value method

The method used for undertaking the multivariate extreme value modelling, comprises 2 steps:

1. Event extraction
2. Fitting of statistical models and stochastic simulation.

These stages are outlined in more detail below.

C.1 Event extraction

Prior to undertaking the fitting of the statistical models it is first necessary to extract the events from the time series data, a process known as de-clustering. Initially, the time series wave and water level data were matched based on time. Peak events were identified separately for significant wave height, wind speed and water level. For each of these variables, local cluster maxima were identified using the blocks method. The blocks method involves identifying the largest event in the dataset and all observations within a fixed time window were removed from the dataset. The outcome of this process is shown in Figure C.1. Where for example, the green dots represent the peaks extracted using the blocks method. This was repeated until all peak values of interest were identified (typically stopping when the next peak would be below the mean value of the variable). An additional check was added to ensure each identified peak would be considered a local maximum within the time window.

For wave height and wind speed a time separation of 1 day was used. For water level 11 hours was used which ensured that every high tide was selected. Each peak value was paired up with the concurrent values of every other variable. Therefore, the same event may be identified multiple times but represented slightly differently in the three sets of peak events. There is no concern with event duplication since each set of peak events is used only to extrapolate peaks of the same variable and duplicates are ultimately removed when sampling from these.

An example of this process is shown in Figure C.1.

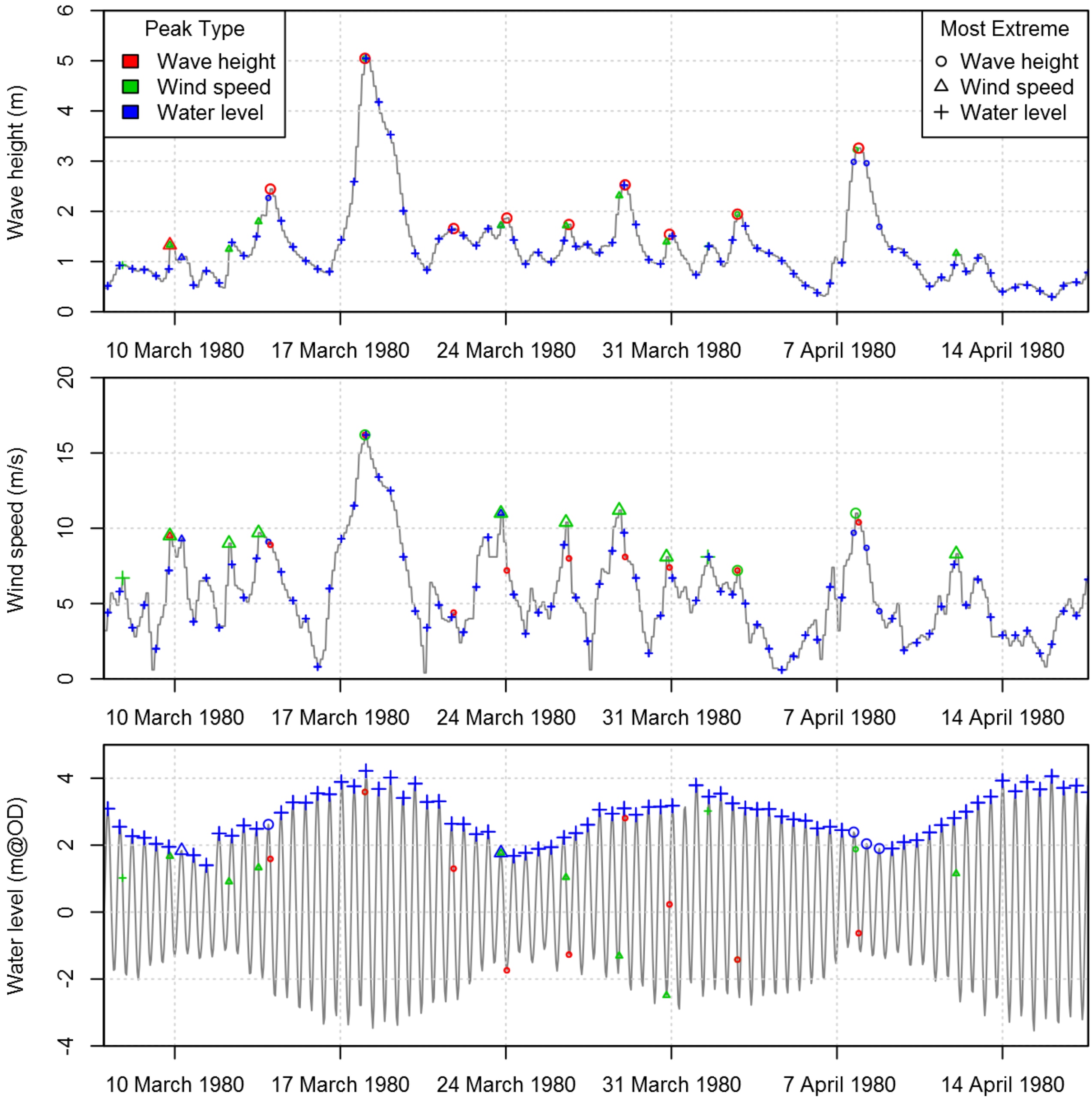


Figure C.1: Example events from the time series wave, wind and water level data

C.2 Fitting of statistical models

The objective of the multivariate extreme value model is to extrapolate the joint probability density of the waves, winds and water level information to extreme values whist ensuring the appropriate dependence between the variables is captured. This approach is preferred to the simplified joint exceedence approach for reasons described above. There are wide range of approaches for undertaking this analysis. The approach adopted on this project is that of Heffernan and Tawn (2004). Further description on the justification for the use of this model in the context of coastal wave and water level analysis is provided by Gouldby et al (2014). The implementation of this method for this project is described below.

Let Xo be a vector of offshore sea condition variables, wave height, period, direction, sea level, wind speed and direction (Xo1, Xo2….Xon), for example. The problem then is to estimate the probability of exceeding some specified value of wave overtopping discharge. The wave overtopping variable is denoted as Z. Xo is related to Z through the function Δ:

$$\Pr(Z > z) = \Pr(\Delta(\mathbf{X}^o) > z)$$

The Joint probability method requires extrapolation of the joint density of Xo to extremes and then integration over the region Δ(Xo)>z:

$$\Pr(Z > z) = \int_{Z>z} f_{X^o}(\boldsymbol{X}^o)\, d\boldsymbol{X}^o$$

Where $f_{X^o}(\boldsymbol{X}^o)$, is the joint probability density of the offshore sea condition variables.

The method of Hefferan and Tawn (2004) enables extrapolation of the joint probability density of the offshore sea condition variables to extreme values with appropriate consideration of the dependence structure. Prior to analysis of the dependencies between each variable, the marginals are first analysed. For this, the standard peaks-over-threshold (POT) approach of (Davison and Smith, 1990) is used, whereby cluster maxima are identified and the excesses above a suitably high threshold are fitted to the Generalised Pareto distribution (GPD). This defines a probability model for large values of the variable Xi.

To provide a full specification of the marginal distributions of the sea condition variables, the empirical distribution of the Xi values, below the threshold, is combined with the GPD above the threshold to provide the following semi-parametric function for the cumulative marginal distribution Coles and Tawn (1991):

$$\hat{F}_i(x) = \begin{cases} \tilde{F}_i(x) & x \le u_i, \\ 1-(1-\tilde{F}_i(u_i))\left[1+\xi_i \dfrac{(x-u_i)}{\beta_i}\right]_+^{-1/\xi_i} & x > u_i. \end{cases}$$

Where, βi andχi are the GPD parameters and ui is a high threshold. The GPD is a well-established model for analysing extremes for POT sea condition variables (Hamm et al., 2010; Jonathan and Ewans, 2013).

In common with other copula approaches that separate the marginal characteristics from the dependence analysis, it is usual to standardise the data to common margins. Within the Heffernan and Tawn (2004) model the standard Gumbel marginal scales are used. The sea condition data are therefore transformed (transformed variables denoted as Y), from their original scales to Laplace Scales using the standard probability integral transformation.

The method proceeds by analysis of the dependence between the variables on the transformed scales. If Y–i denotes the vector of all variables excluding Yi, the method is applied using the multivariate non-linear regression model:

Y–i | Yi = a Yi + Yib W for Yi > v,

Where a and b are vectors of the parameters from the fitted pair-wise regression model, v is a specified threshold and W is a vector of the residuals. The model is fitted using maximum likelihood assuming the residuals follow a normal distribution with a mean and standard deviation to be found. Once fitted, a Monte Carlo simulation procedure is used whereby samples from the residuals are combined with the parameter estimates to obtain realisations of Y. The steps involved in the Monte Carlo sampling procedure are summarised below:

1. Sample a value of Yi (i.e. from the variable on the transformed scales) conditioned to exceed threshold v.
2. Independently sample a joint residual, W.
3. Calculate Y-i, from the regression equation, using the sampled W and the fitted regression parameters.
4. Reject if Yi is not a maximum.

These steps are repeated until the relative proportion of events where Yi is a maximum, conditional on being above the threshold, is consistent with the empirical distribution. This process is then repeated conditioning on each variable in turn, to ensure the appropriate proportion of events is simulated. The output of this process is a large sample of simulated data on the transformed scales.

These data are then transformed back to the original scales by reversing the previously applied transformations. The resulting output is a large multivariate sample of extreme (in at least one variable) offshore sea condition data that captures the characteristics of dependencies between the variables, as well as preserving the marginal extremes.

Table C.1: Summary of statistical treatment of the sea condition variables

Sea condition variable	Statistical treatment for simulation
Significant Wave Height (Hs)	Heffernan and Tawn (2004)
Wave Period (Tm-10)	f(Hs, steepness)
Wave Steepness	Empirical Distribution
Water level	Heffernan and Tawn (2004)
Wind Speed (U)	Heffernan and Tawn (2004)
Wind Direction (θ_U)	Empirical distribution
Wave Direction (θHs)	Empirical Distribution
Directional spreading	Empirical Distribution

D Background to joint probability methods

The simplified joint exceedence method has been widely used in practice and is detailed in the Environment Agency's Best Practice Guide. It is however, known that this method is approximate and there are limitations associated with this. For this reason, the method has not been used on this project. An explanation of the simplified nature of this method is however, provided below for reference purposes.

The simplified joint exceedence approach, conceptualised in Figure D.2, generates contours of the extreme variables (e.g. waves and sea levels) that have an equal likelihood of simultaneous exceedence (sometimes known as joint probability contours). Combinations of the variables (e.g. wave and sea levels) that lie on a contour are then applied to the response function (overtopping in this case) to determine which combination yields the worst case (highest) overtopping rate. This overtopping rate is then assigned the same return period as the joint exceedence contour.

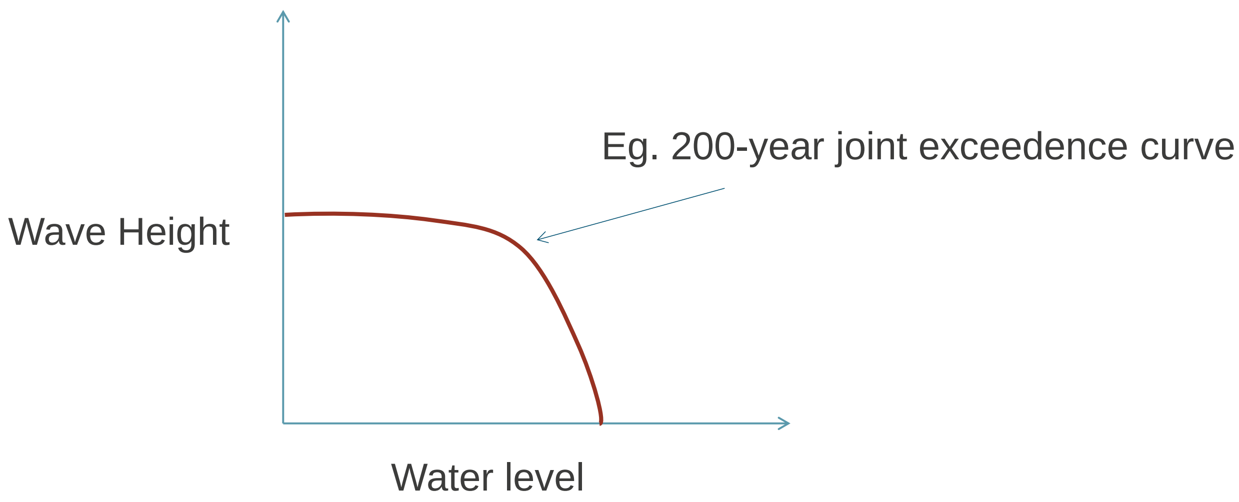


Figure D.2: Conceptual diagram illustrating the joint exceedence approach

This method was developed by HR Wallingford for "broad-brush" applications and contains a known error that results in an underestimation of overtopping rates (Hawkes et al 2002) unless correction factors are applied. The error arises as a result of the method of probability integration implicitly assumed in the simplified joint exceedence approach. The error is shown in concept in Figure D.3.

If X1 and X2 represent the random variables of water levels and wave heights respectively the task is to estimate the probability of exceeding a specified overtopping rate, y (or alternatively identify the overtopping rate associated with a particular return period). A constant value of overtopping is represented by the green contour. The probability space associated with exceeding y is represented by the green shading. To determine this probability, it is necessary to integrate the joint probability density of X1 and X2 over the region that exceeds y (i.e. the green shaded region). The simplified joint exceedence approach does not however, seek to do this directly. The simplified approach assumes a particular form and is defined by assessing the probability of exceeding a specified value of X2 at the same time a value of X1 is exceeded. This is represented by the red shaded region in Figure D.3. The error that arises as a result of implementing the joint exceedence approach can be viewed as the difference between the green and red shaded areas.

The magnitude of the error varies with spatial location and structure type but invariably, results in an under-estimate of the overtopping rates. For the cases analysed by Hawkes et al (2002) the error was typically found to be around a factor of 3 on return period. Or, in other words, the simplified approach generated estimates of overtopping rate with a return period of 100 years that should have been closer to 30 years.

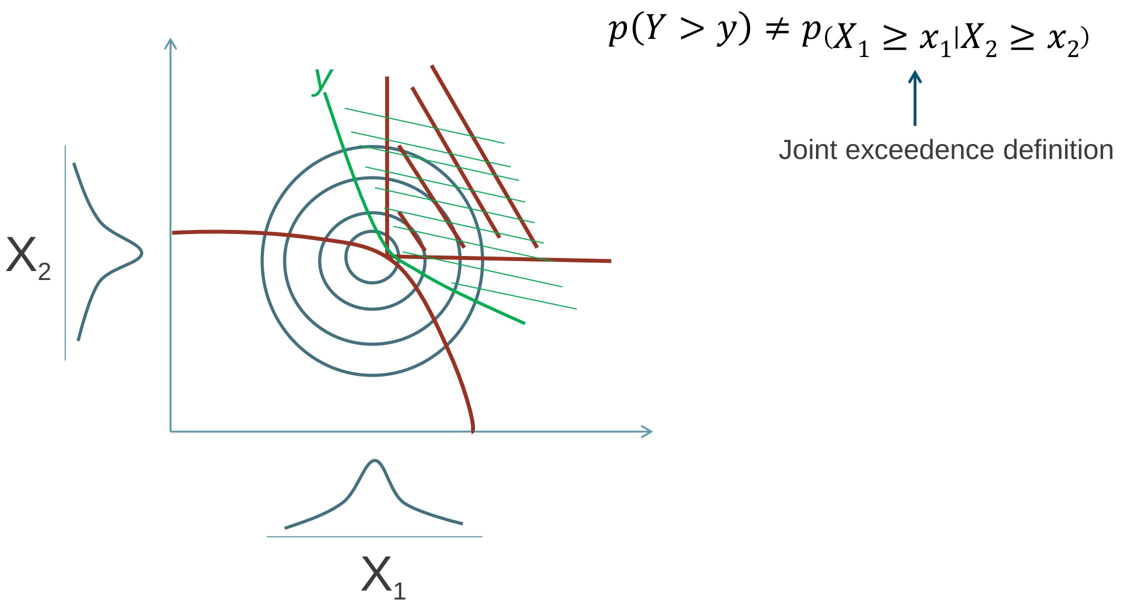


Figure D.3: Approximate integration method used by the joint exceedence approach

Whilst it is possible to make manual adjustments to provide an approximate correction to the error. For example, the return periods of overtopping rates obtained using the joint exceedance approach could be multiplied by 3. It is complex and cumbersome to make these adjustments spatially consistent (i.e. the adjustment is not always a factor of 3, it varies spatially and with structure type). In particular, to establish the degree of error it is necessary to undertake a more robust joint probability analysis. For these reasons the more robust approach was preferred for this project using the methodology of Heffernan and Tawn (2004) applied with an emulation method. The more robust approach uses a Monte-Carlo method to undertake the integration. This method of integration is shown in Figure D.4. The blue dots represent events from the MC simulation that exceed a specified value of overtopping rate. The probability of exceeding this rate can then be determined by comparing the number of blue events to the total number of (red and blue) events.

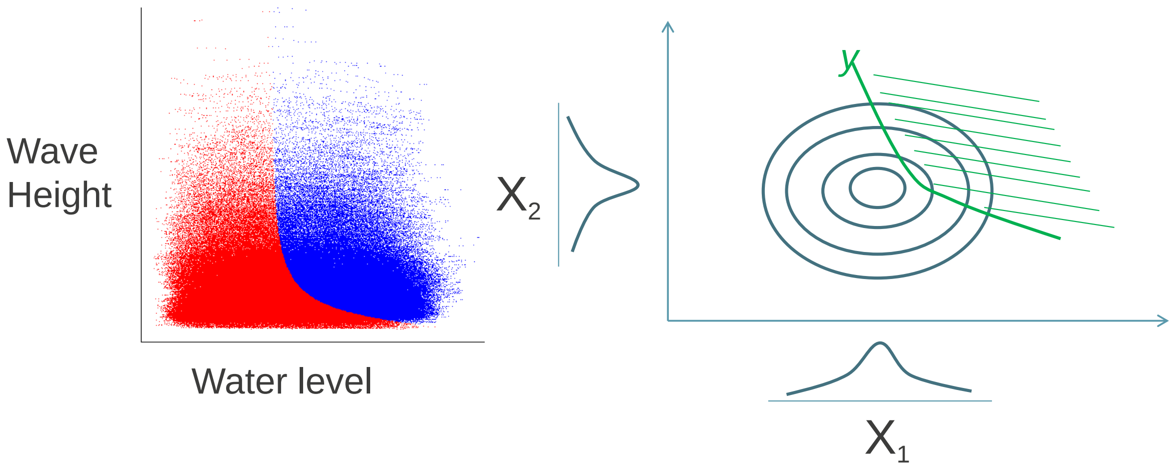


Figure D.4: Monte-Carlo simulation used to integrate the joint probability density of waves and water levels

E Emulator description

Gaussian process emulators are used to predict the SWAN model output at each nearshore point for every event in the large offshore simulated dataset. A separate emulator is applied for each nearshore point and for each output variable of interest. Let $y = f(\boldsymbol{x})$ represent the SWAN output for a single nearshore variable as a function of the offshore variables $\boldsymbol{x} = (x_1, \dots, x_p)$. It is not computationally practical to run the model for every event in the simulated dataset but we can run a relatively small subset of events to obtain $y_i = f(\boldsymbol{x}_i)$ for $i = 1, \dots, n$ where n is typically 500.

The offshore events $\boldsymbol{x}_i$ for which SWAN is run are known as the design points of the emulator. These are selected as a subset of the entire simulated dataset using the Maximum Dissimilarity Algorithm (MDA). This ensures that the selected events cover the entire simulated space as efficiently as possible.

The Gaussian process emulator approximates the unknown function $f(\boldsymbol{x})$ by treating it as a random Gaussian process. The statistical model is defined by a mean function $\boldsymbol{h}(\boldsymbol{x})$ satisfying

$$\mathrm{E}(f(\boldsymbol{x})|\boldsymbol{\beta}) = \boldsymbol{h}(\boldsymbol{x})^T\boldsymbol{\beta}$$

and a covariance function $c(\boldsymbol{x}, \boldsymbol{x}')$ satisfying

$$\mathrm{Cov}(f(\boldsymbol{x}), f(\boldsymbol{x}')|\sigma^2) = \sigma^2 c(\boldsymbol{x}, \boldsymbol{x}')$$

where $\boldsymbol{\beta}$ and σ^2 are parameters. The mean function is typically taken to be a linear function of the input variables. For State of the Nation, the Gaussian covariance function is used which is defined by

$$\mathrm{c}(\boldsymbol{x}, \boldsymbol{x}') = \exp\left(-\sum_{i=1}^{p} \theta_i (x_i - x_i')^2\right)$$

for smoothing parameters θ_i.

A Bayesian formulation has been used to estimate the function output probabilistically given the n known outputs at the design points. For a simulated offshore event $\boldsymbol{x}$, the best estimate of $f(\boldsymbol{x})$ in light of the known outputs is given by

$$\boldsymbol{h}(\boldsymbol{x})^T\boldsymbol{\beta} + \boldsymbol{t}(\boldsymbol{x})^T A^{-1}(\boldsymbol{y} - H\boldsymbol{\beta})$$

where

$$\boldsymbol{t}(\boldsymbol{x})^T = (\mathrm{c}(\boldsymbol{x}, \boldsymbol{x}_1), \dots, \mathrm{c}(\boldsymbol{x}, \boldsymbol{x}_n)),$$

$$A = \left[\mathrm{c}(\boldsymbol{x}_i, \boldsymbol{x}_j)\right]_{i,j=1}^{n},$$

$$\boldsymbol{y}^T = (f(\boldsymbol{x}_1), \dots, f(\boldsymbol{x}_n)),$$

$$H^T = (\boldsymbol{h}(\boldsymbol{x}_1), \dots, \boldsymbol{h}(\boldsymbol{x}_n)).$$

If applied to one of the design events $\boldsymbol{x}_i$, this formula returns the known SWAN output with zero error. The prediction equation is applied for every offshore event to produce a large simulated dataset of nearshore events. On occasions the emulator can yield results at the nearshore locations that are beyond the range of results output from the design point simulations. Where results from the emulator that were beyond a specified threshold were identified and run within the SWAN model.

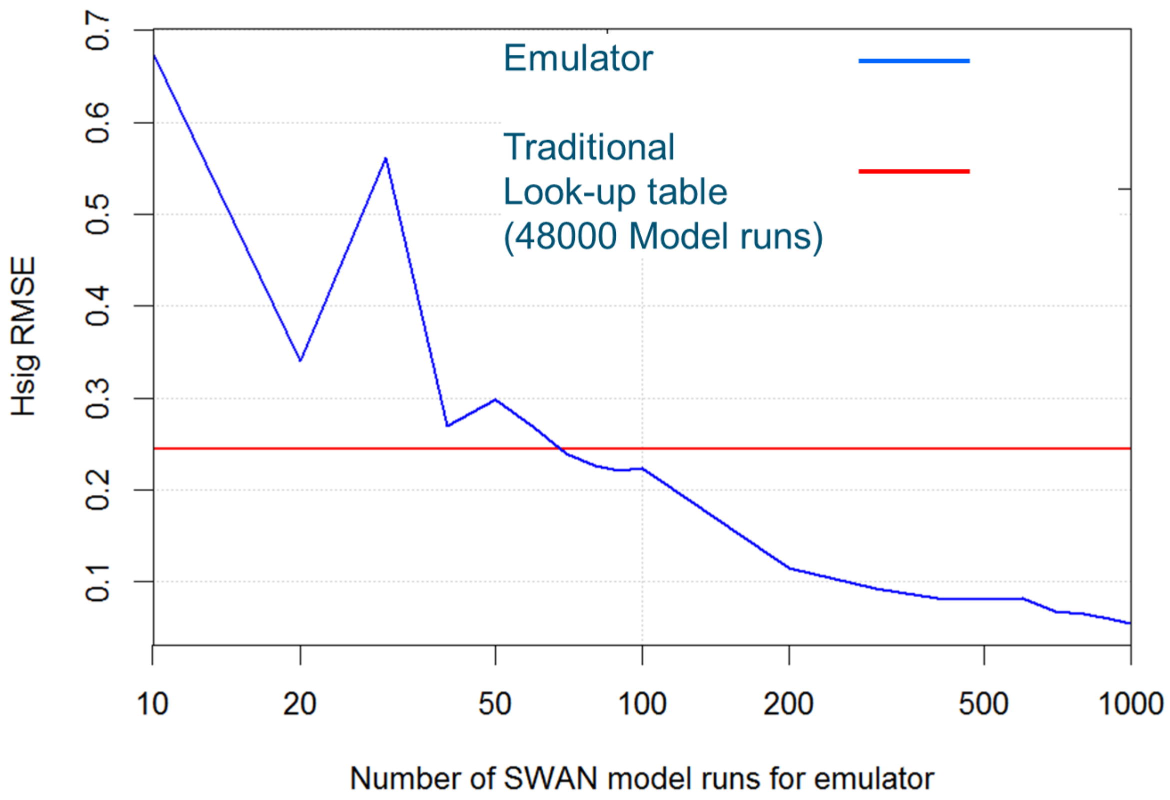
F Design point event selection

To develop the GPE of the nearshore wave transformation process, it is necessary to run the simulation model (SWAN) for a set of m design points. The design points are required to cover the input boundary condition space. The Maximum Dissimilarity Algorithm, Kennard and Stone (1969), with further refinements described by Willett, 1999), is particularly well-suited for this task as it enables the outer limits of the input boundary space to be appropriately represented. This is not the case for other clustering methods, SOM and K-means, for example, (Camus et al., 2011a).

The MDA algorithm was applied to the Monte Carlo realisations output from the multivariate extreme value analysis of the offshore wave conditions. Prior to implementation, it was however, first necessary to transform the data onto standard scales, including the directional component. The standardisation involves making a linear transformation to scale the variables between 0 and 1, using the maximum and minimum value for each variable. For the directional variables, the maximum distance is in radians, hence the directions have been divided by 2π to scale between 0 and 1. Further information on these transformations are described in detail by (Camus et al., 2011b).

The output from the application of the MDA defines a subset of m points that are uniformly distributed in the transformed space across the offshore boundary (i.e. the SWAN model input boundary space). This subset of design points is constructed sequentially. Firstly, the point with the maximum significant wave height is identified. The next stage is to calculate the point in the data set that is furthest, in terms of Euclidean Distance, from this point. Then, the algorithm determines the point in the data set that is furthest from these two points, and so on, until a subset of size m points is defined.

For this analysis m was defined as a minimum of 500 based on analysis that showed diminishing returns in terms of error reduction for further simulations see Figure below.



G Wave climate at structure toes

This brief note gives a general outline of the wave climate at the toes of two defence structures along the Wentlooge to Caldicot levels. One of these sections is exposed to winds across the estuary, section 7, which is typical for most of this frontage. The second section, 28, is in a more sheltered location, at a noticeably more oblique angle to the predominant wave direction. This section highlights how wave conditions can change in relatively more sheltered locations.

These sections are shown on Figure 1 below.

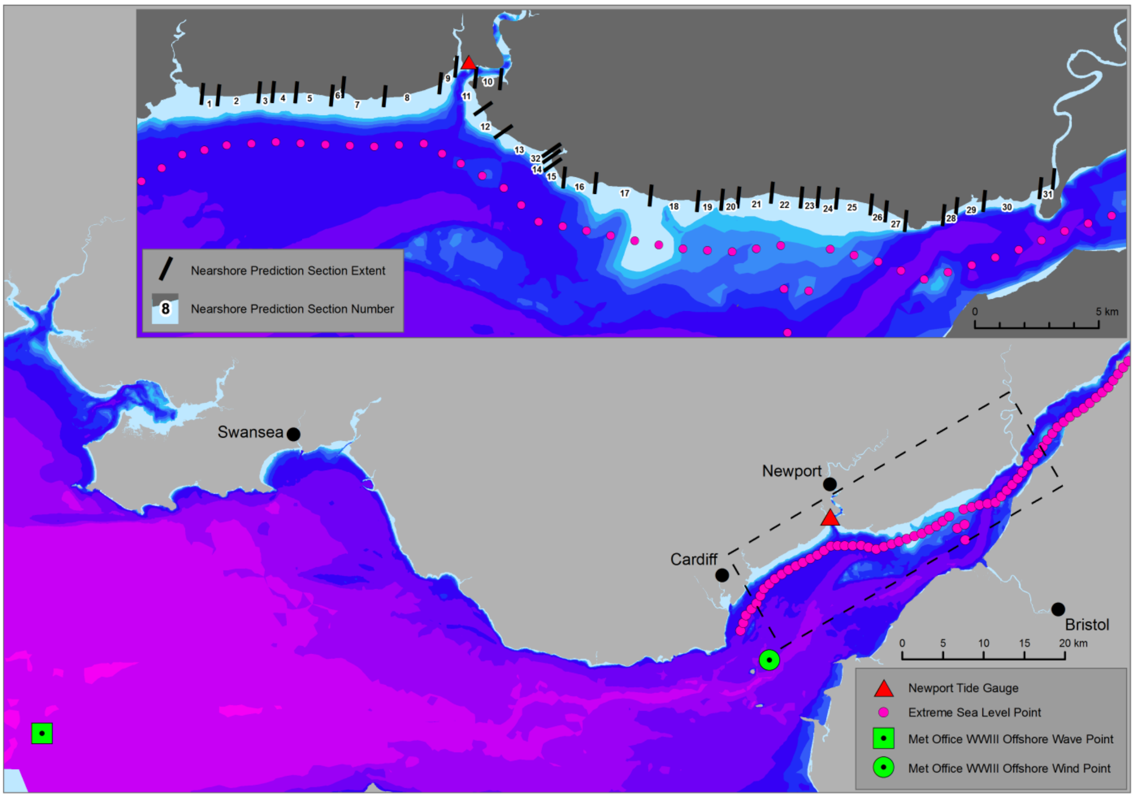


Figure 1: Location of Wentlooge to Caldicot levels and sections considered in this SWAN model grid and location of joint probability data sets.

G.1 Offshore wave climate

Figure 2 shows the offshore climate for the Wentlooge to Calidicot frontage. This is based on the Met. Office wind and wave points shown on Figure 1. This highlights the predominant wind and wave direction from about an angle of 250ºN, where the direction is measured from the north in a clockwise direction. Waves are noted to be their largest when travelling from this direction, which is also the predominant wind direction. There are also relatively large waves corresponding to a direction of about 50ºN, which corresponds to winds blowing down the Severn Estuary across the study site.

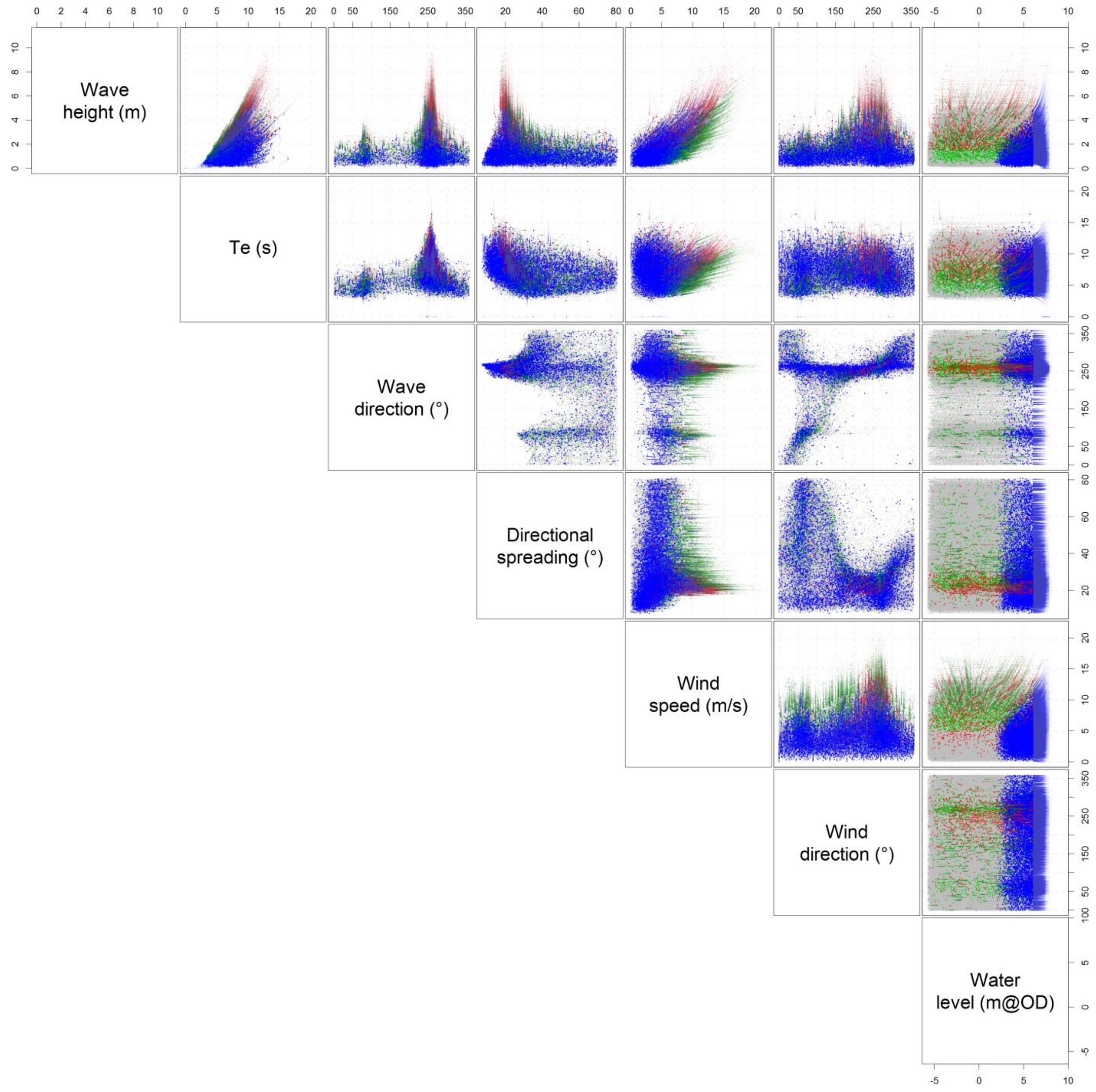


Figure 2: Offshore wave climate based on the multivariate extreme value analysis for Wentlooge Caldicot

Note: Grey points are underlying time-series data. The red, green and blue points are simulated points from the fitted and extrapolated statistical model.

However, as the offshore waves travel up the estuary, they tend to refract and diffract and reduce significantly in energy. The predominant driver of wave conditions along the Wentlooge frontage is therefore a result of local winds which causes waves to grow from the direction that they blow as energy is imparted into them from the winds. Like offshore waves, these waves tend to align themselves with the beach contours, which is most pronounced in shallow water conditions. This typically results in a normal, or near normal angle of approach to coastal defence structures. This can be seen in Figure 3, where the largest waves are from an angle of about 180ºN, which is roughly head-on to the shoreline. Waves also undergo a level of wave breaking, which is an approximate function of wave depth. The higher the water level therefore the less wave breaking, and vice versa. This can be seen in Figure 3, where the largest waves tend to occur at larger water levels.

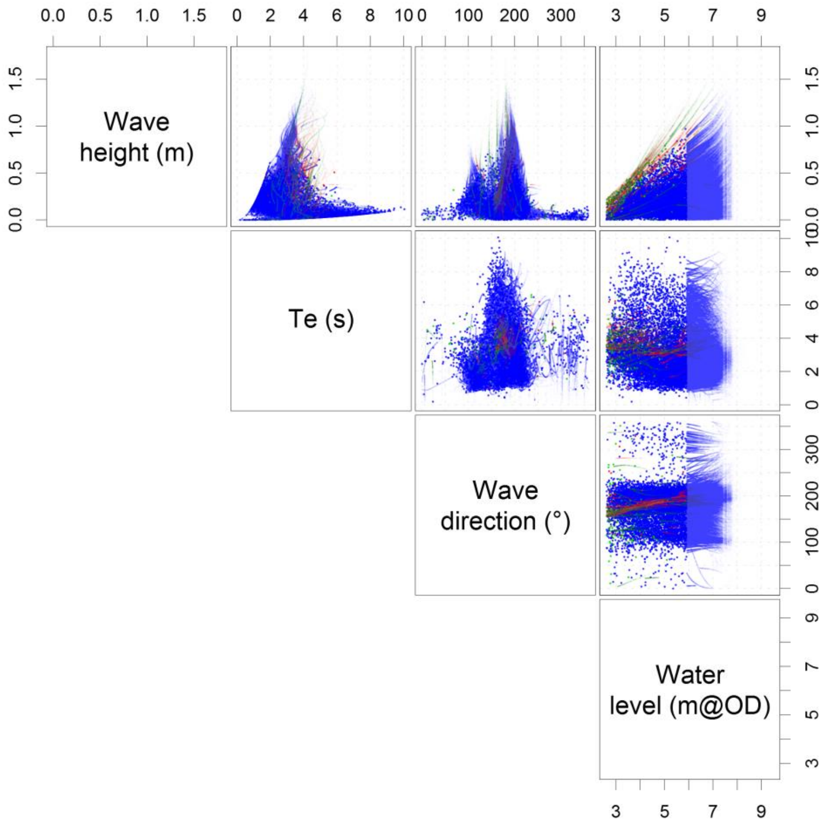


Figure 3: Wave climate at the structure toe for section 7.

Note: Grey points are underlying time-series data. The red, green and blue points are simulated points from the fitted and extrapolated statistical model.

In some cases, the defences are partially sheltered from significant wave action by their orientation to the predominant main direction. In these cases, a combination of wave refraction and breaking will significantly reduce the largest waves as the waves approach the shoreline from an oblique angle. Waves generated by lower wind speeds generated over smaller fetch lengths may therefore produce the largest wave activity at the structure toes. This is demonstrated by section 28 shown on Figure 4. The largest wave heights for this section correspond to waves approaching the site from approximately of 150ºN[48]. These waves may be refracted waves from the predominant wave direction. However, considering the relatively high saltmarsh levels typical in this area (which would significantly refract and break these waves), are more likely to be waves generated by winds blowing from an approximate 150ºN

[48] This is based on the wave direction at the structure toe, not the wave direction offshore of the toe (i.e. before it refracts).

direction. Figure 4 also shows the effect of water depth on the wave heights, with, as with Figure 3, the largest wave heights tending to occur at the higher water levels.

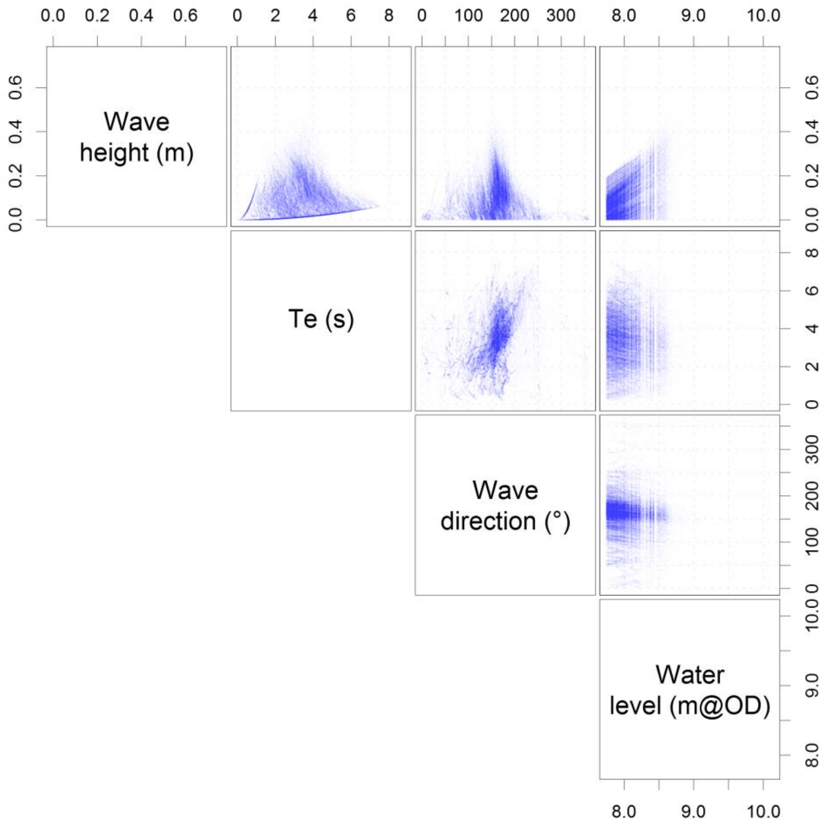
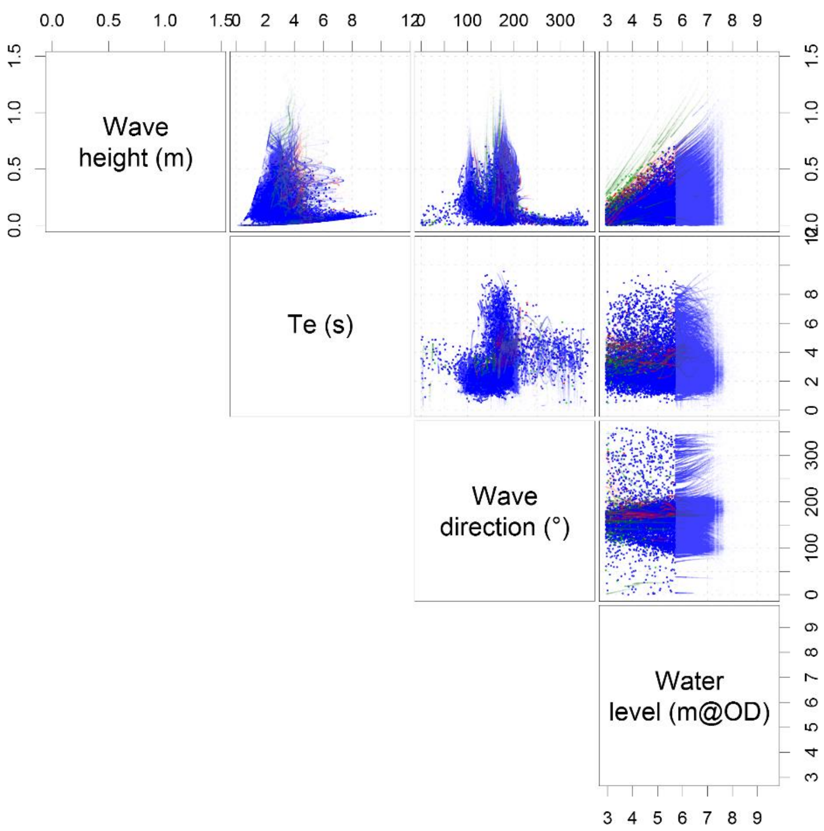


Figure 4: Wave climate at the structure toe for section 28.

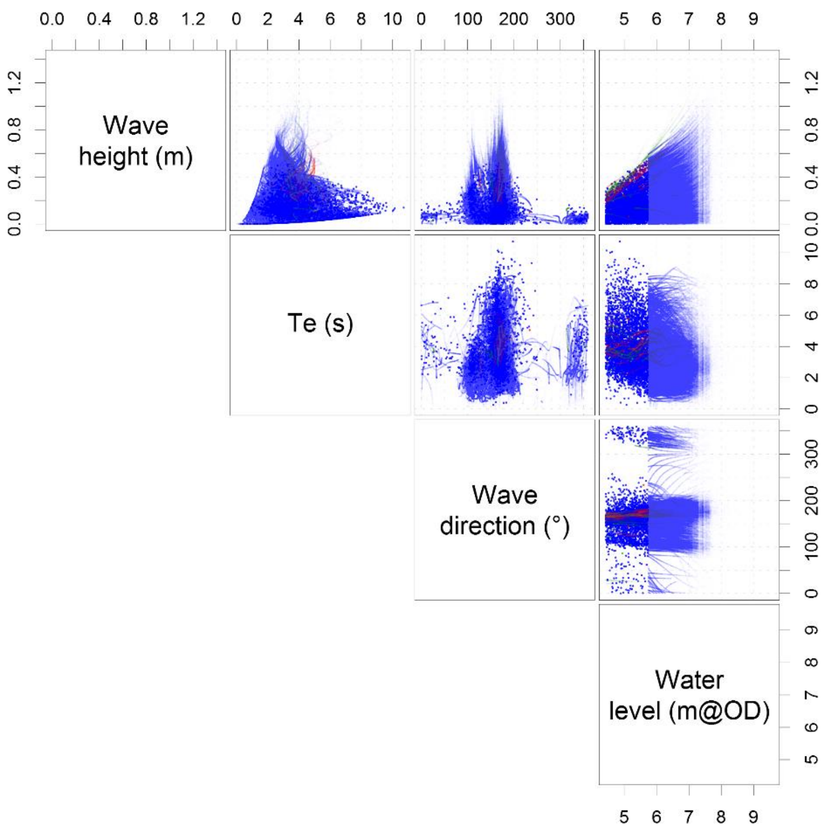
Note: Grey points are underlying time-series data. The red, green and blue points are simulated points from the fitted and extrapolated statistical model.

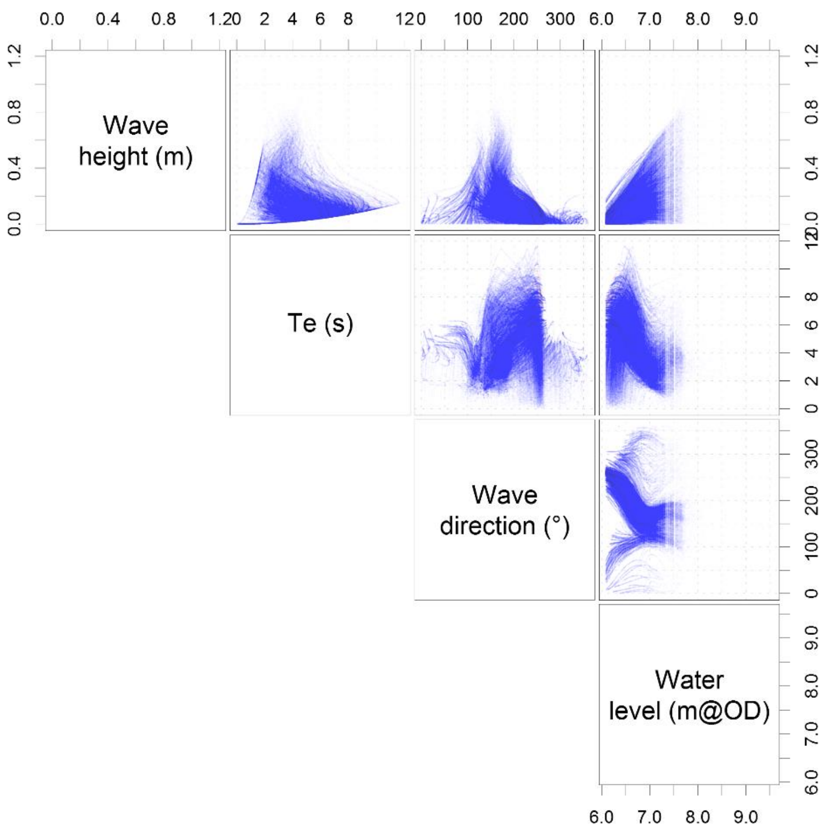
Pairs plots for each of the defences are provided below.

Section 1

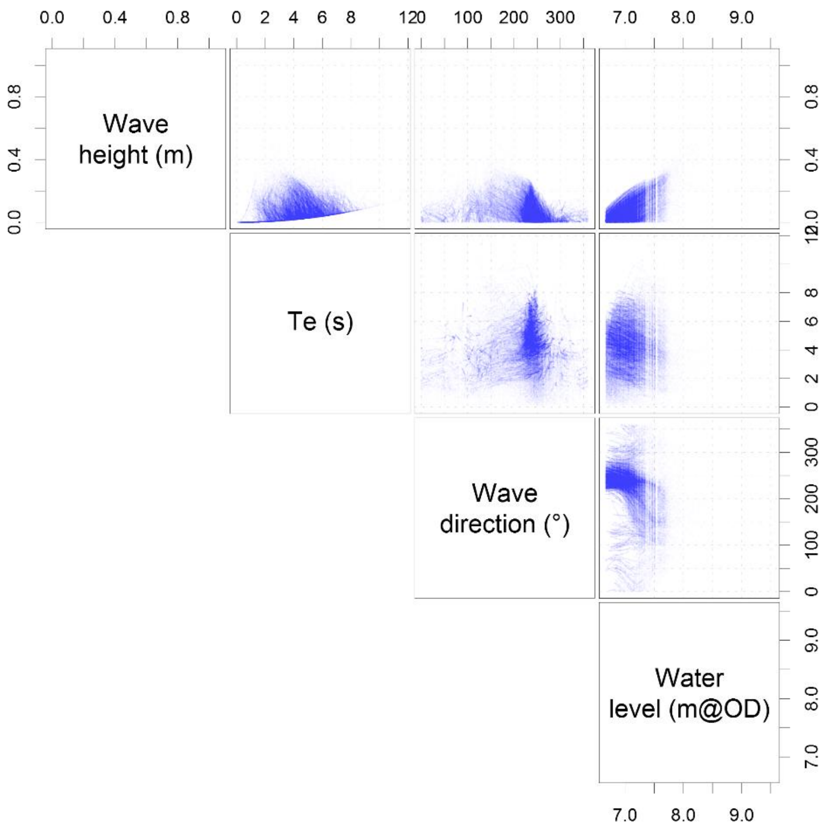


Section 2

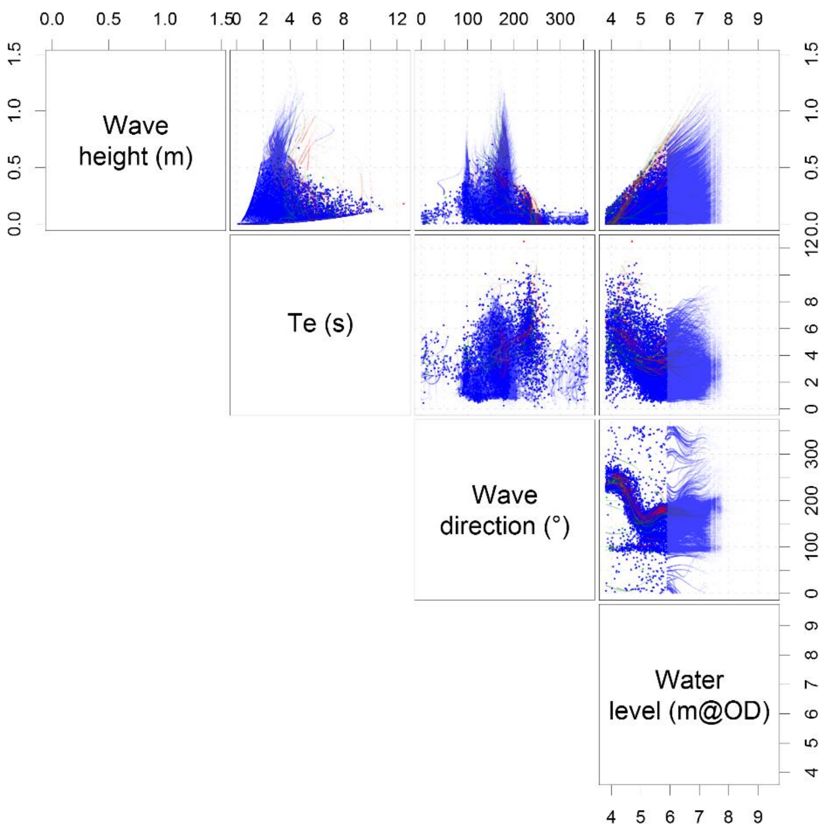




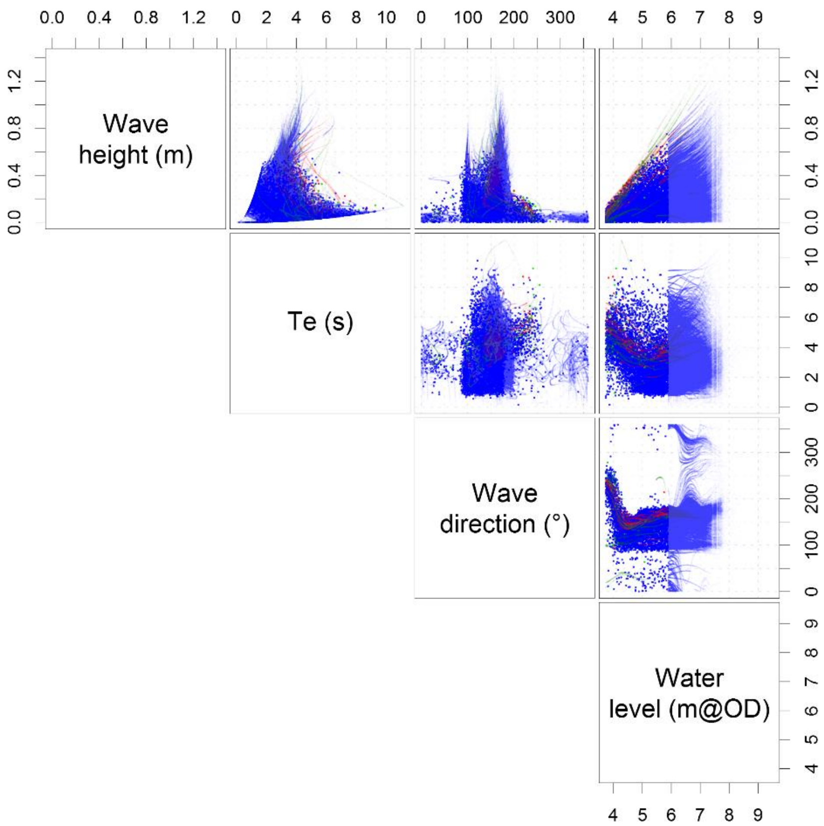
Section 3



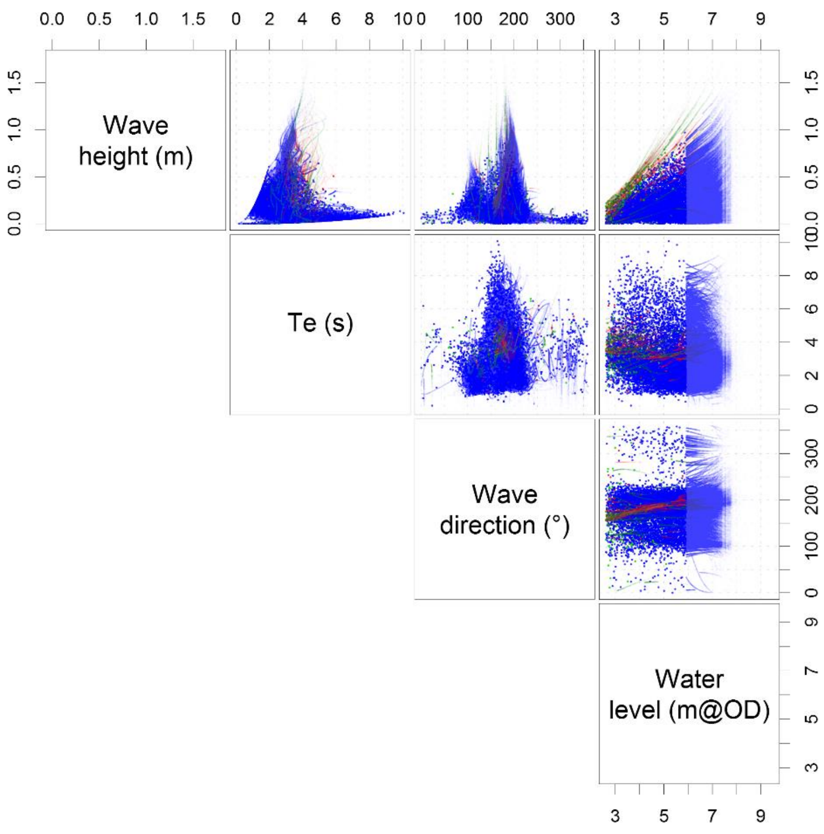
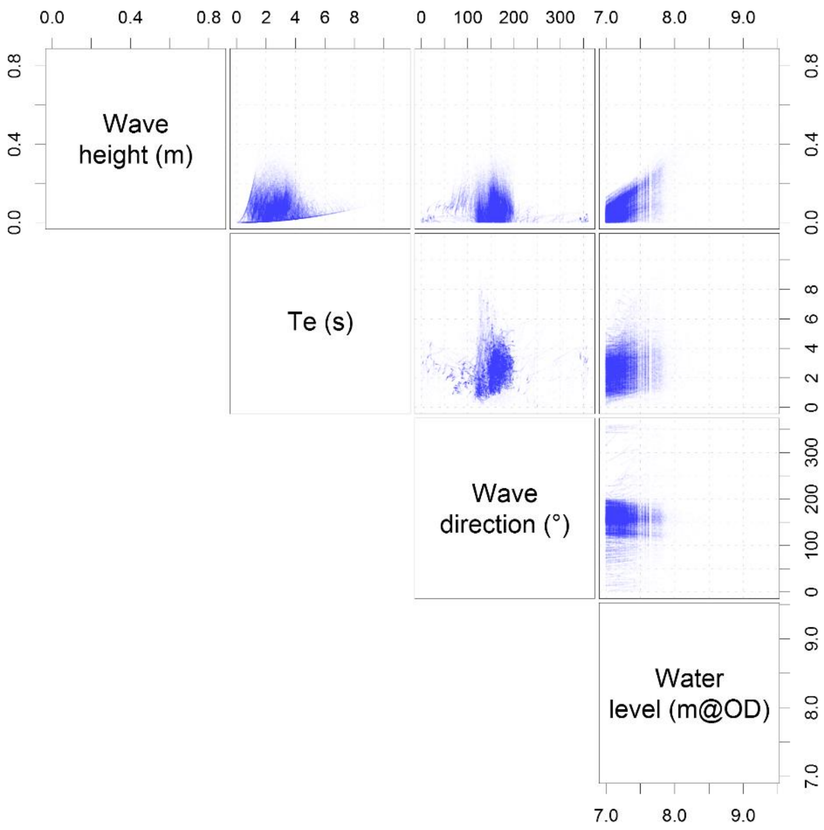
Section 4

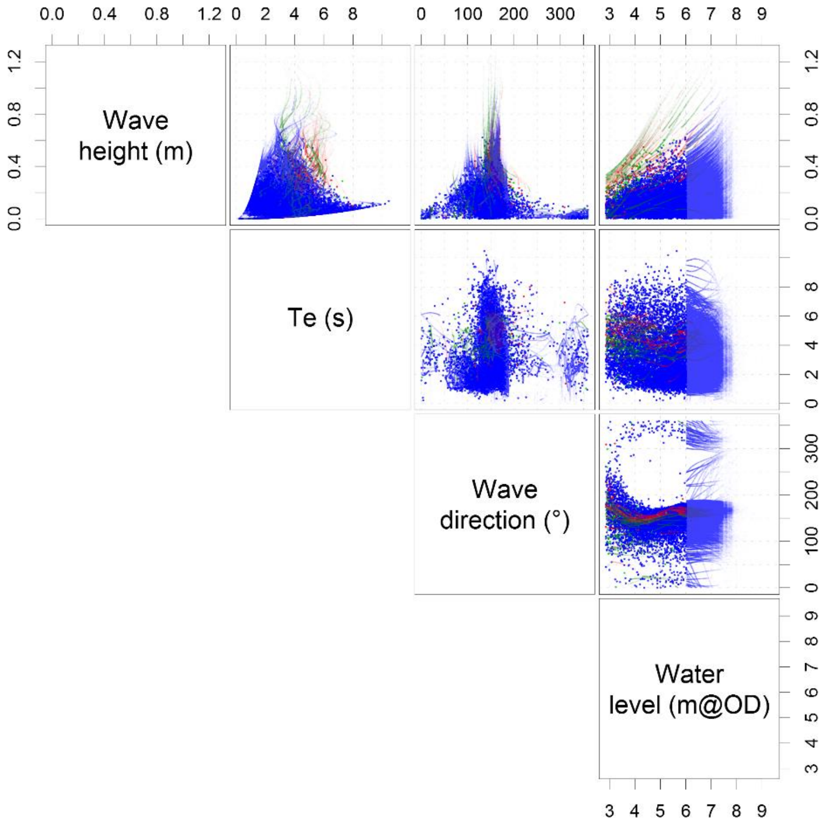
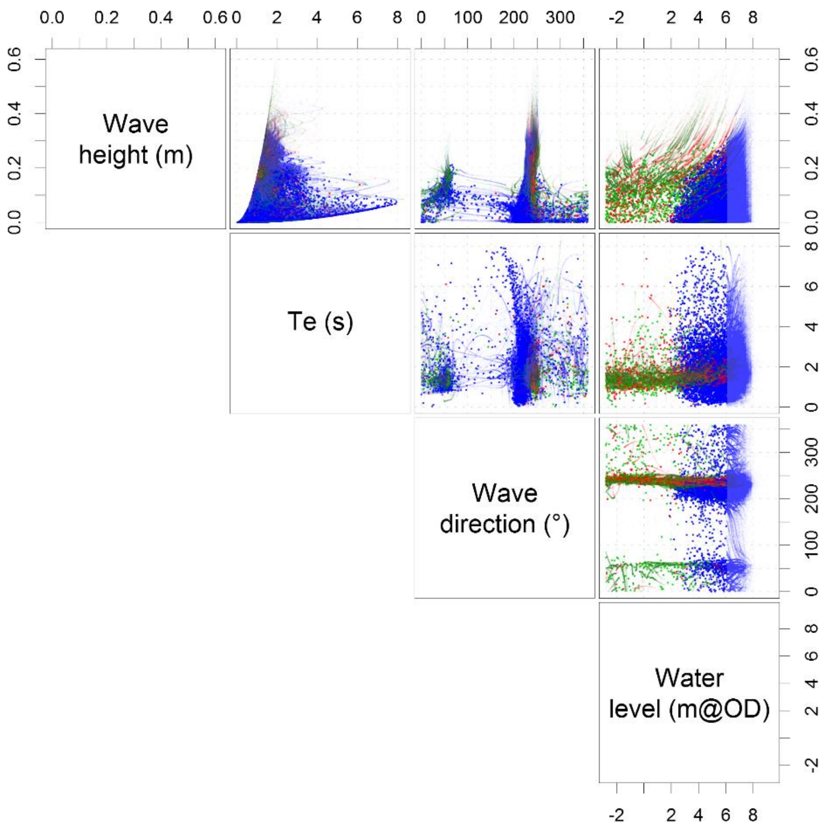


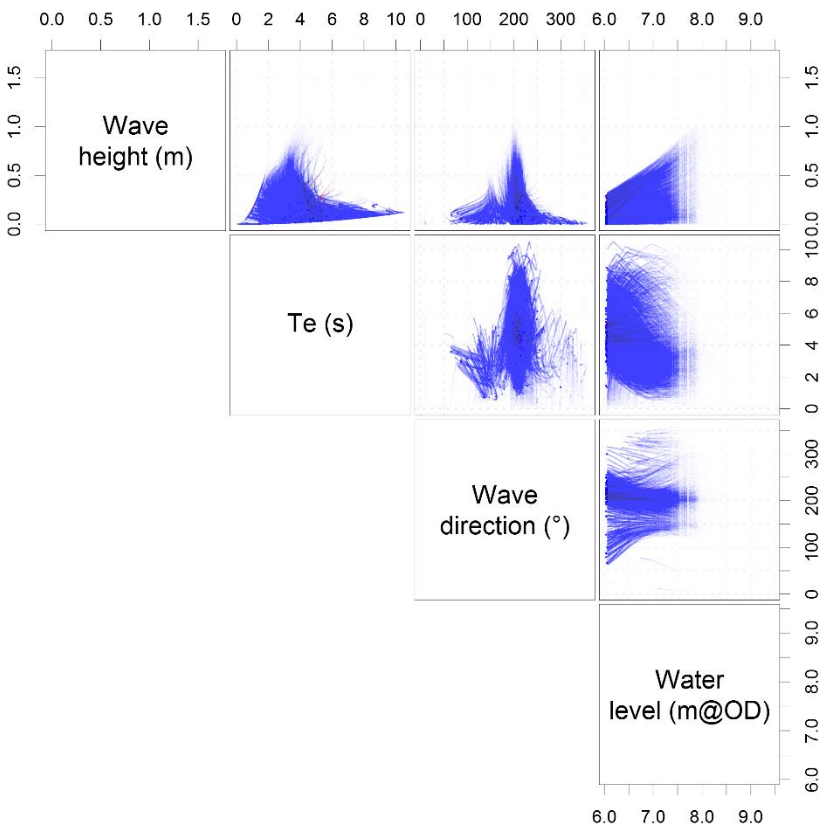
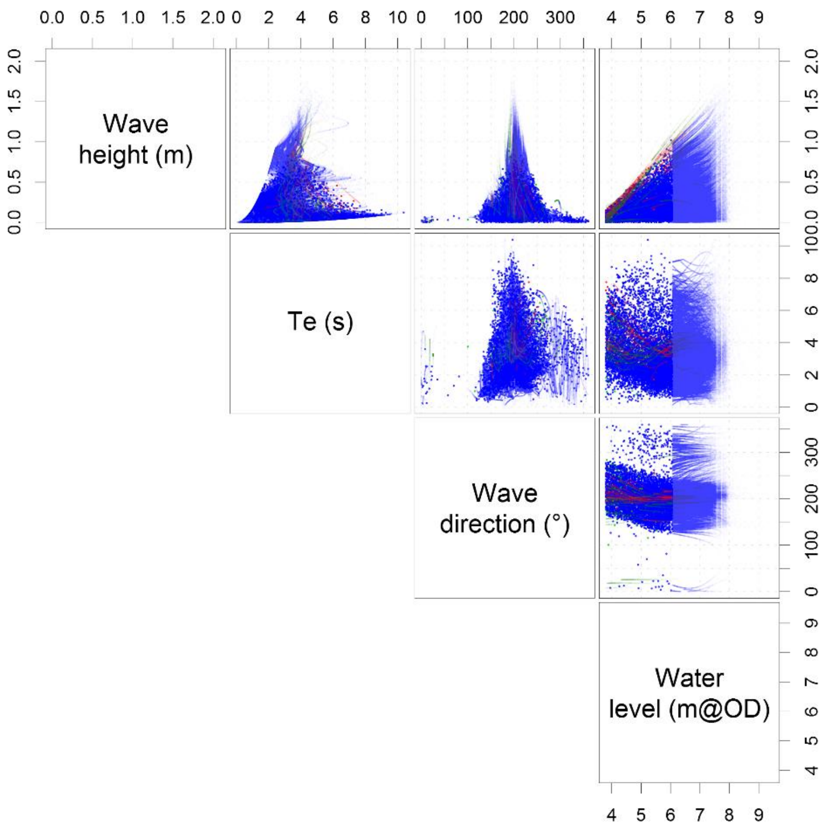
Section 5



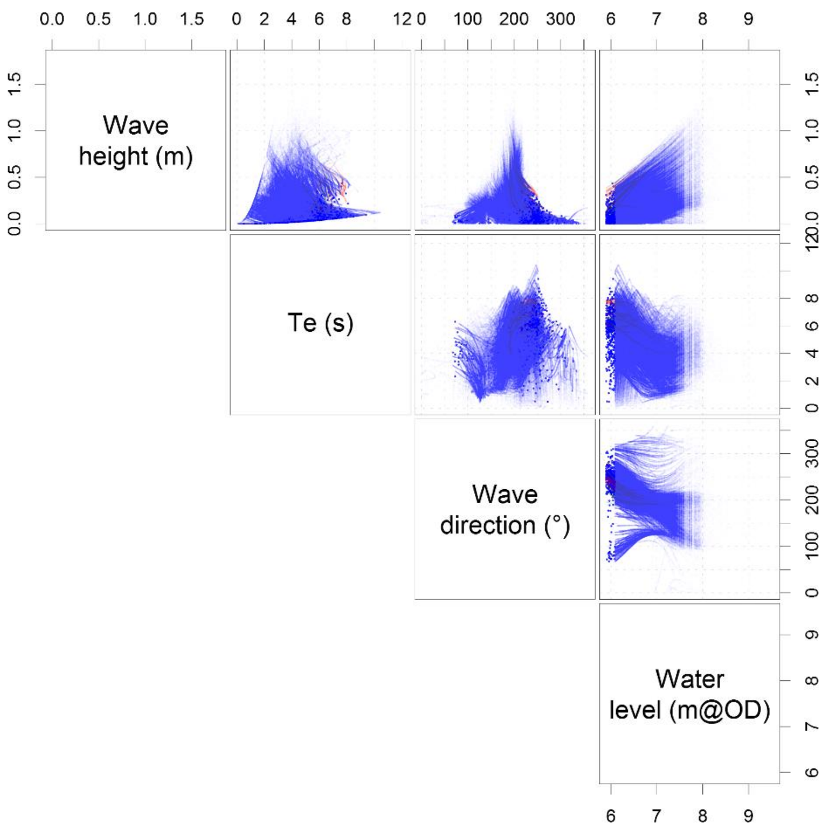
Section 6

	Section 7
	Section 8

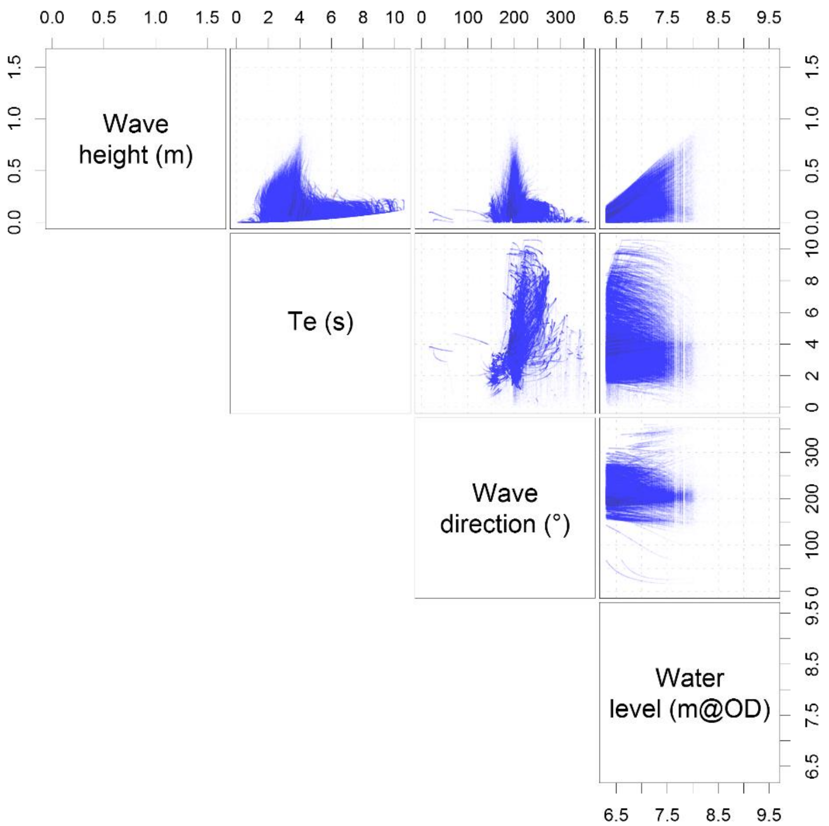
	Section 9
	Section 10

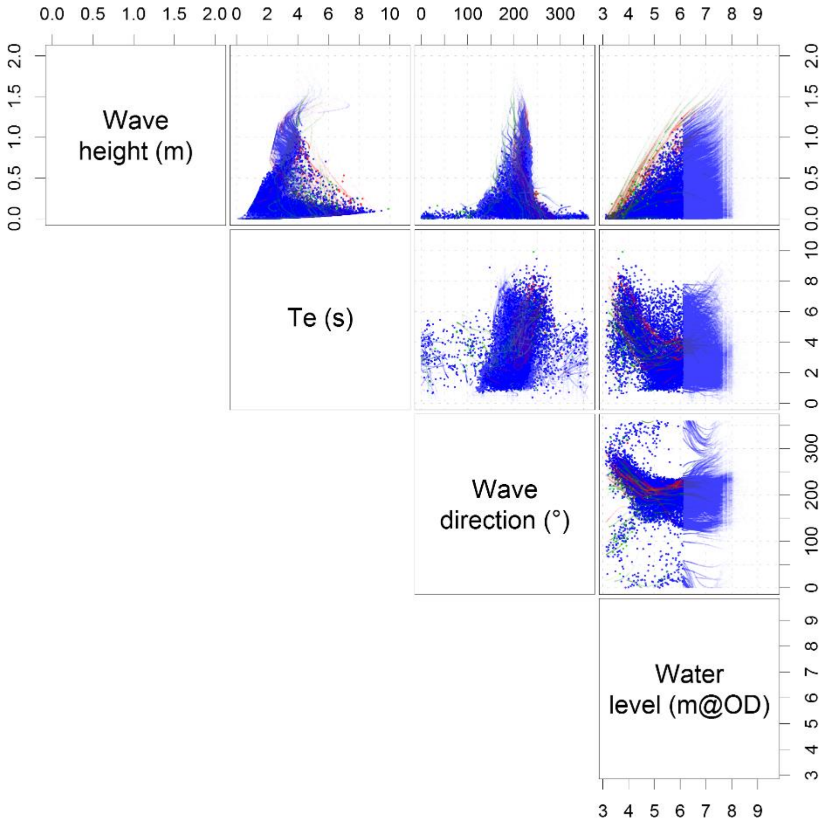
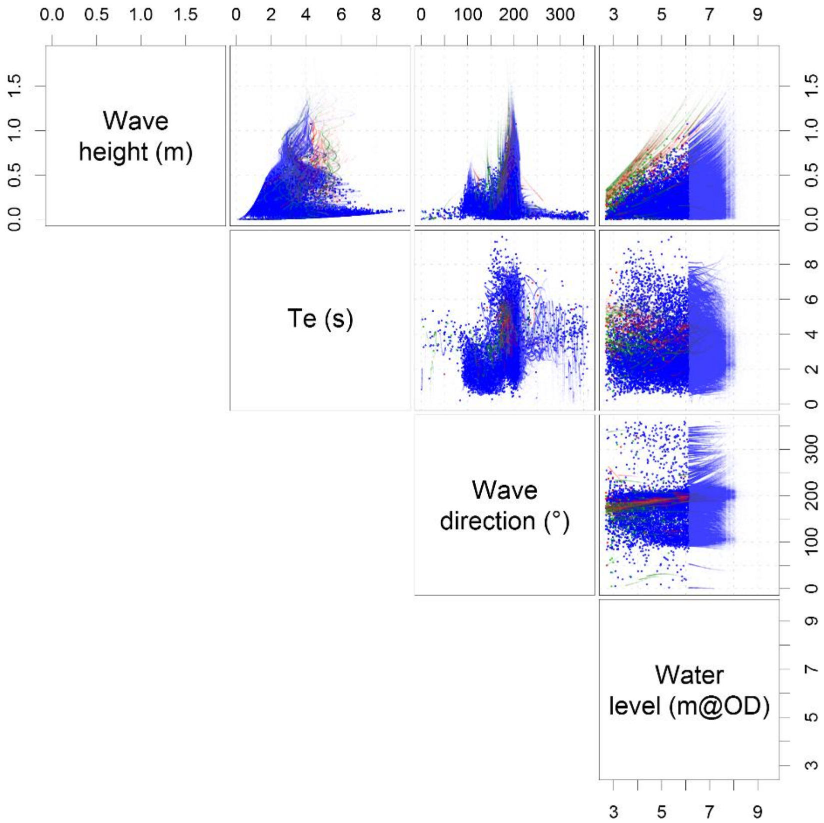
	Section 11
	Section 12

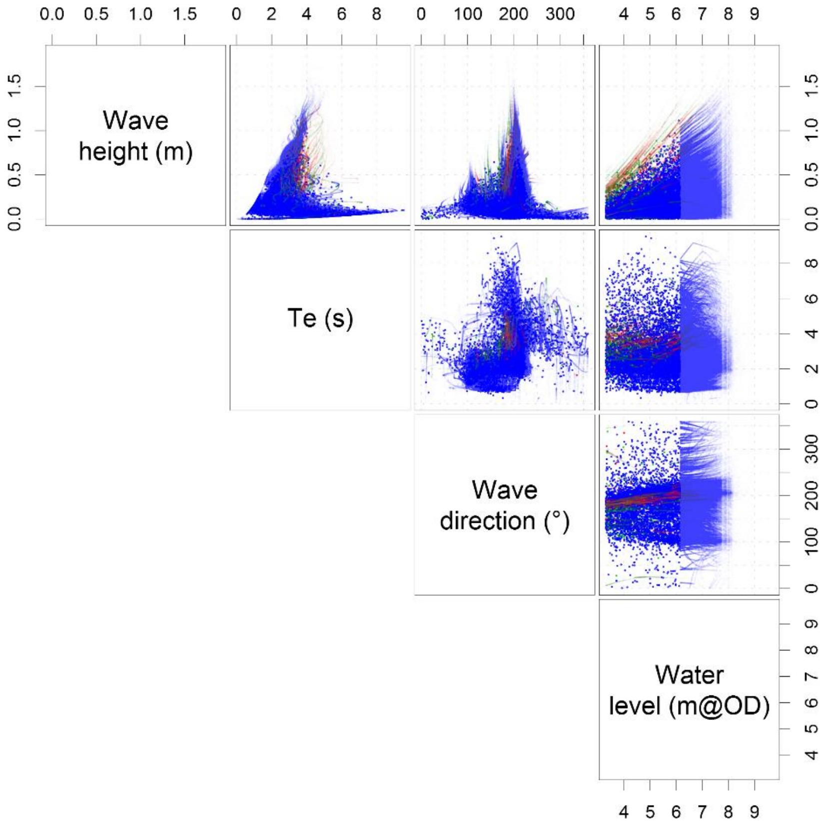
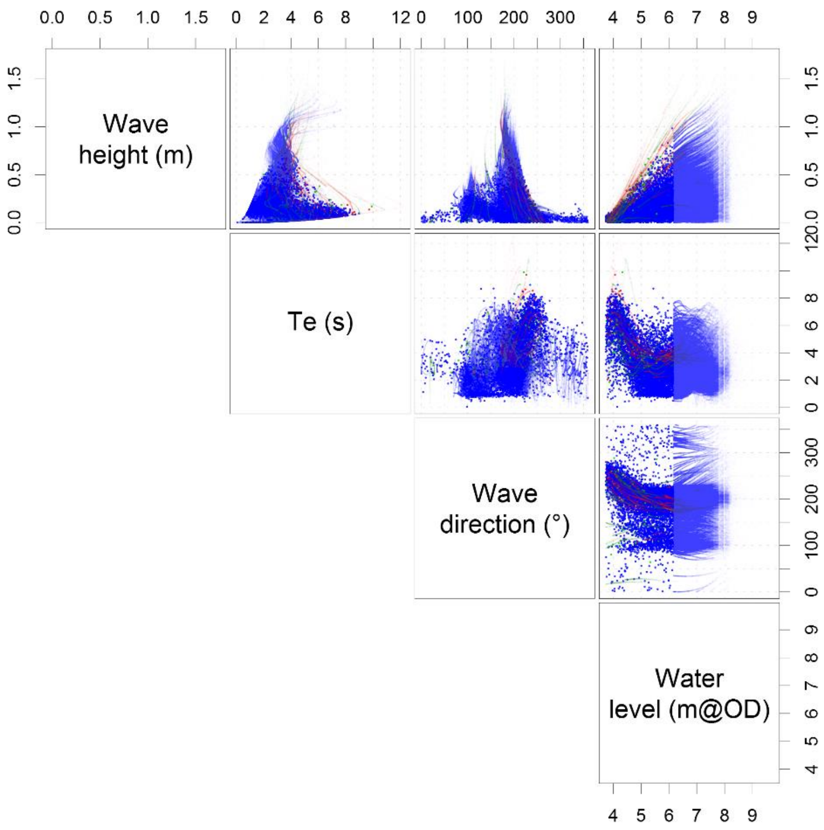
Section 13

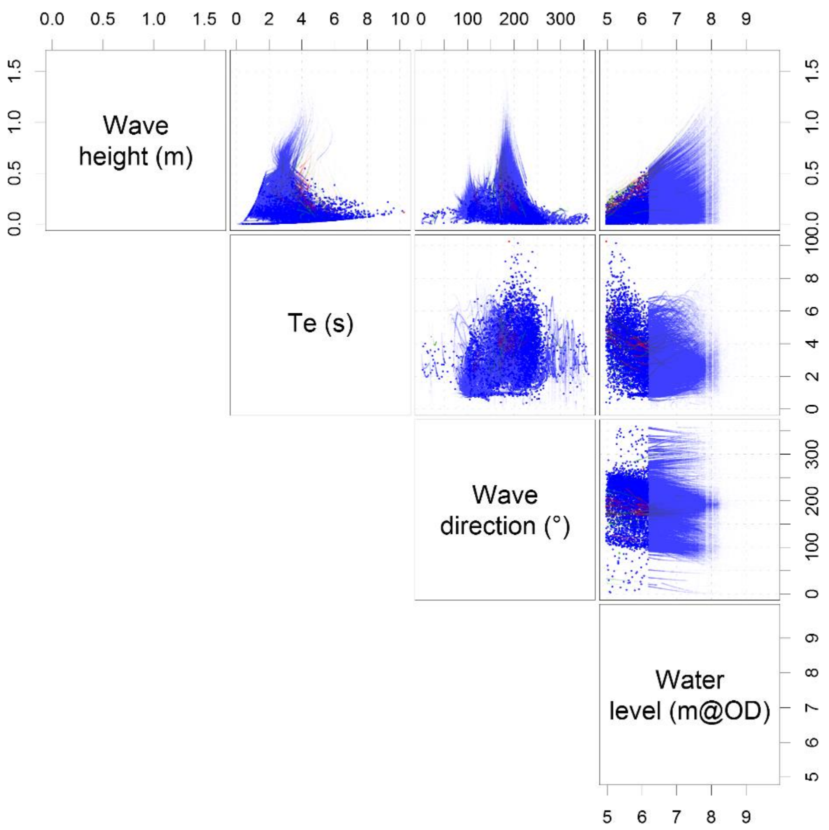
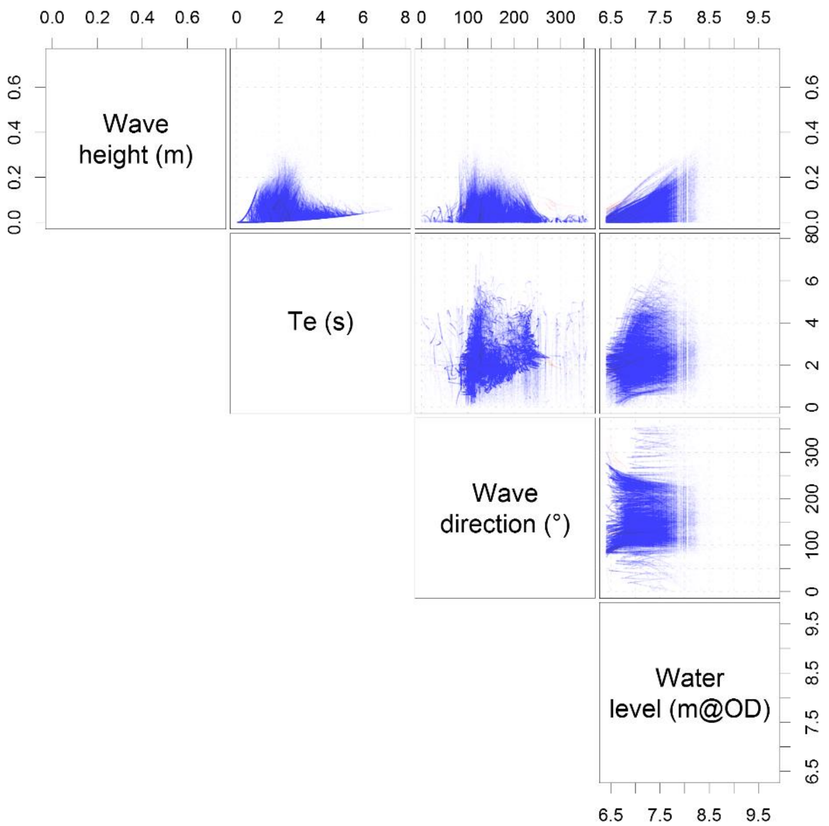


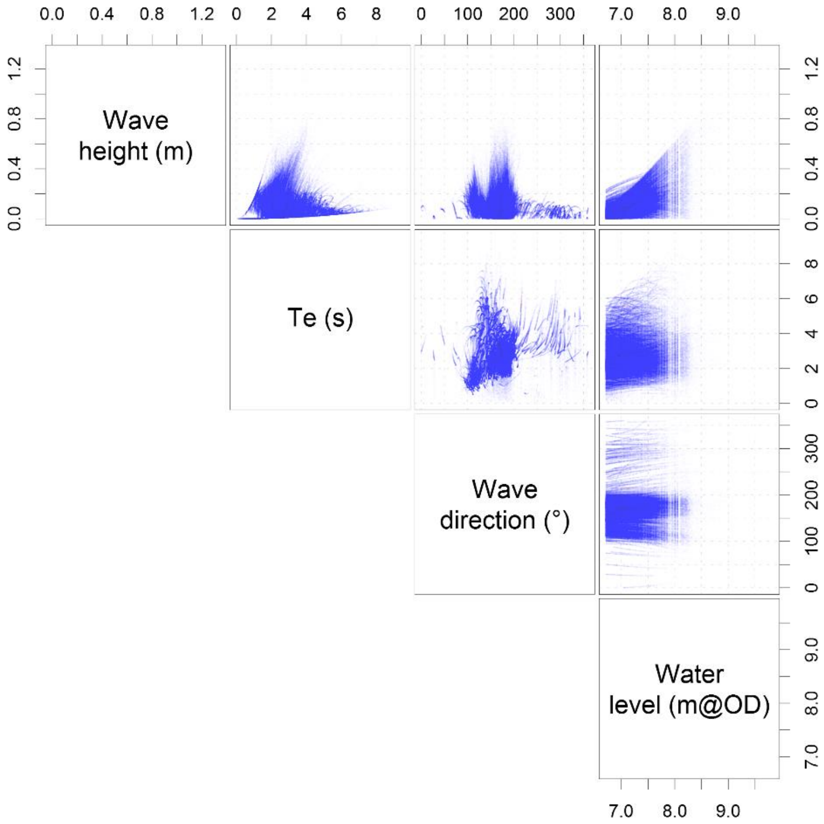
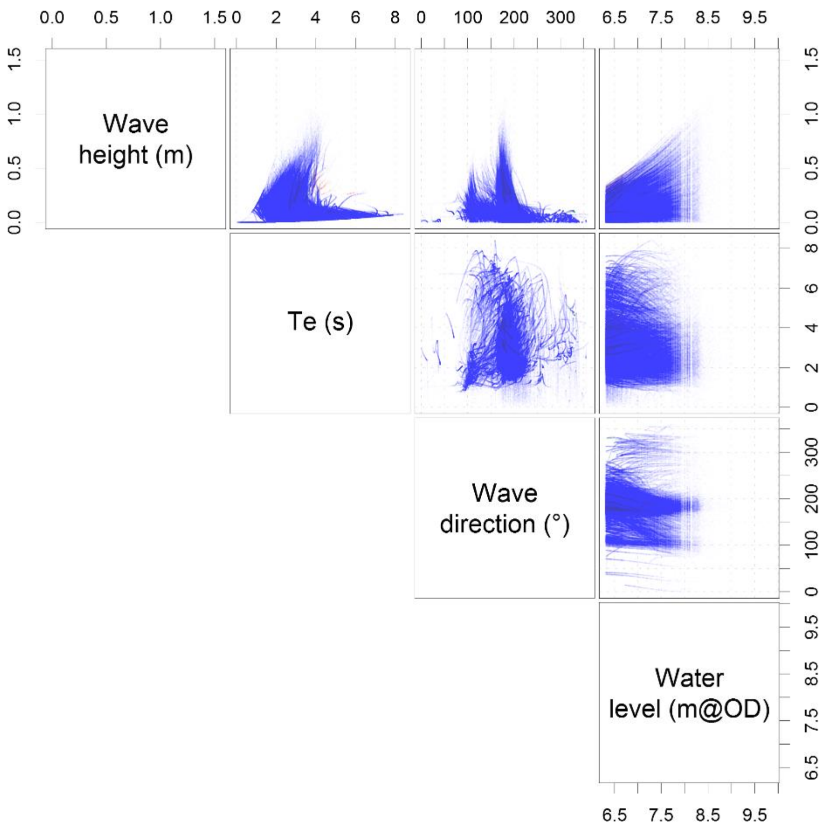
Section 14

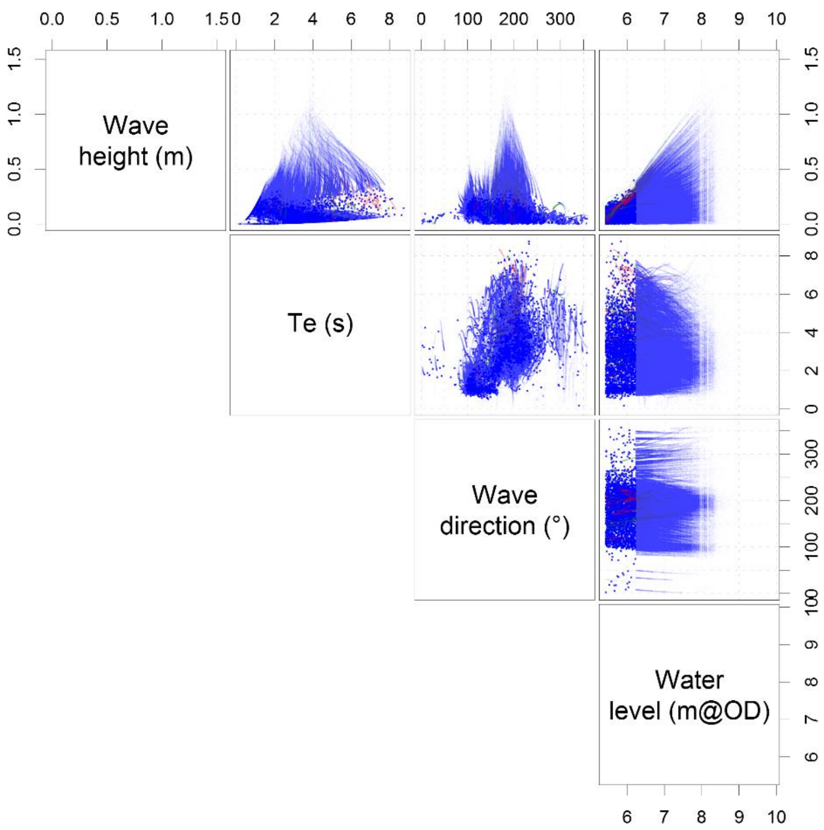
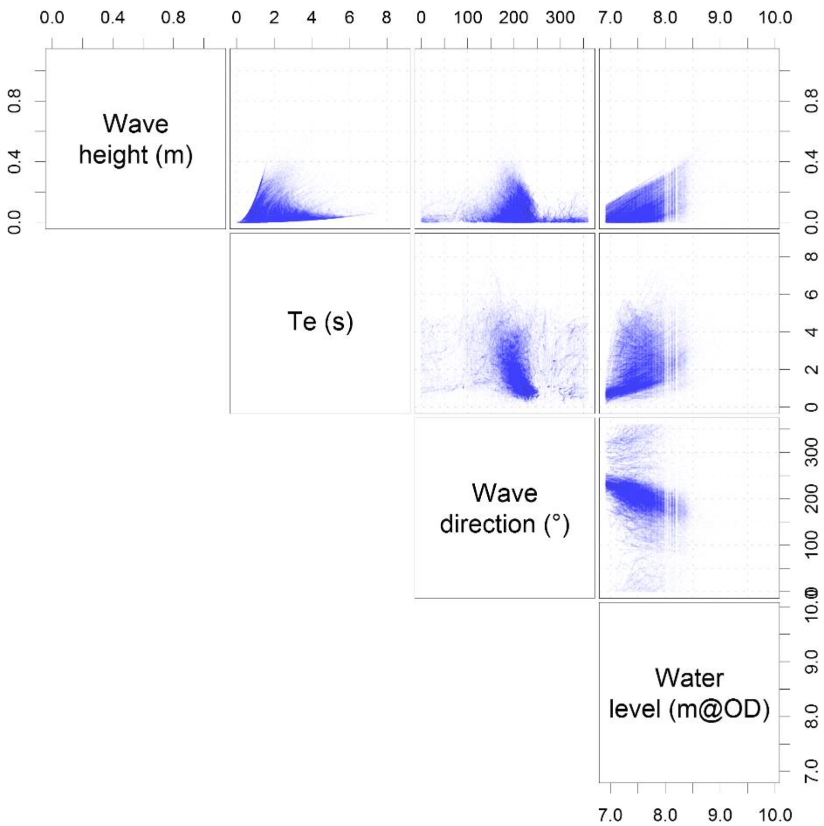


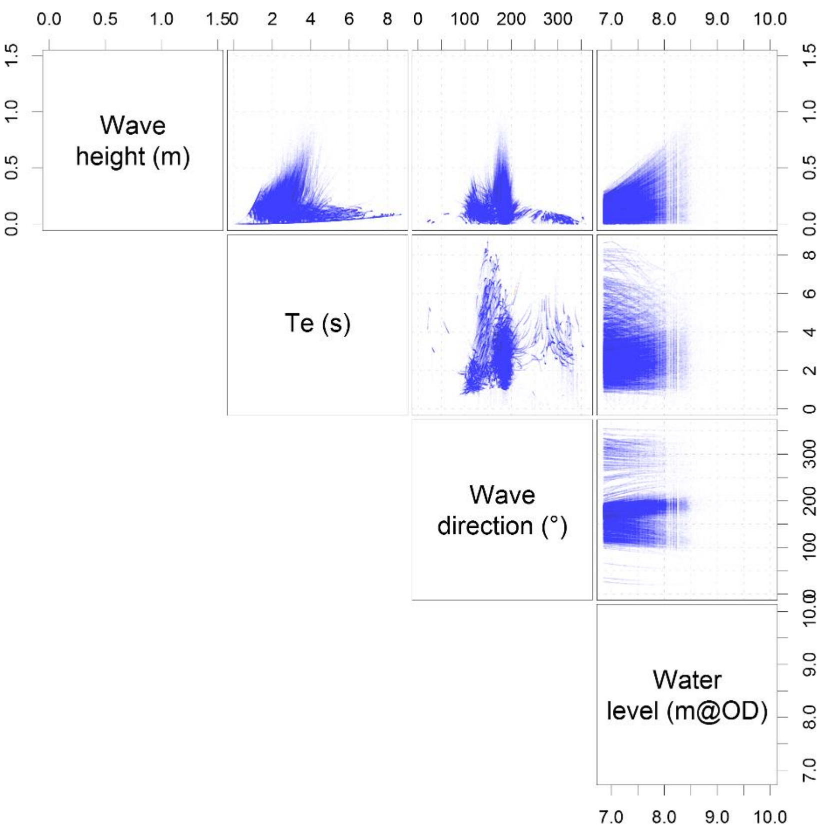
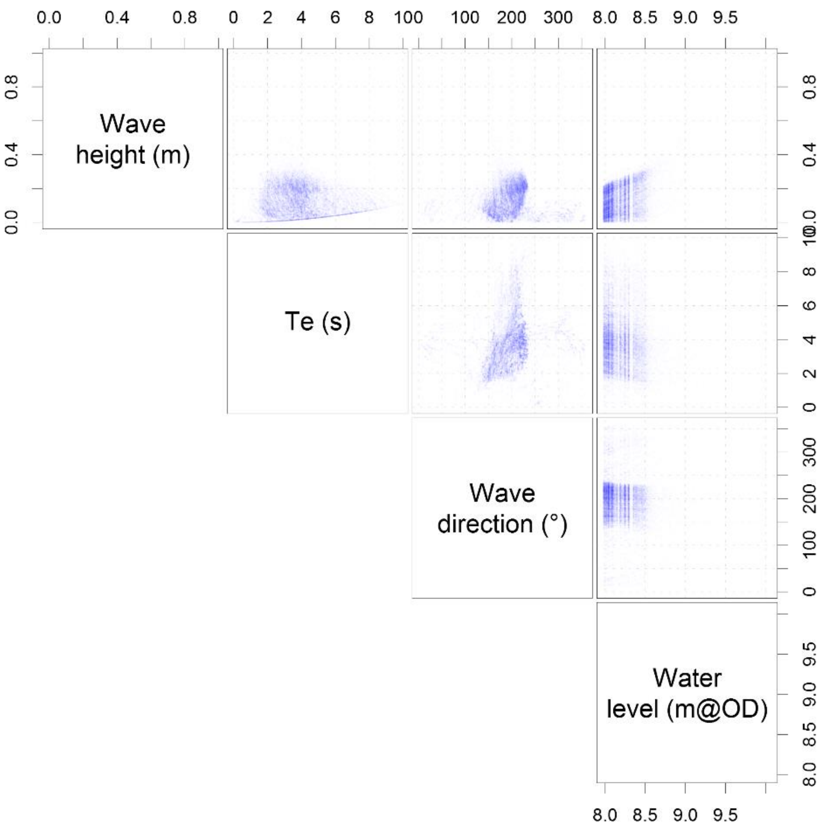
	Section 15
	Section 16

	Section 17
	Section 18

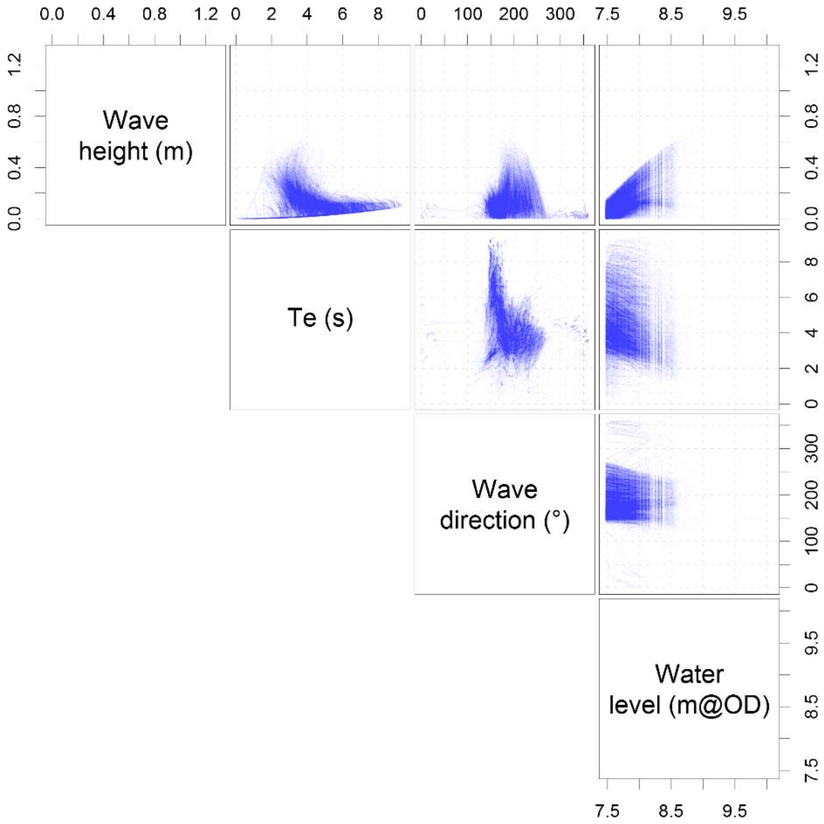
	Section 19
	Section 20

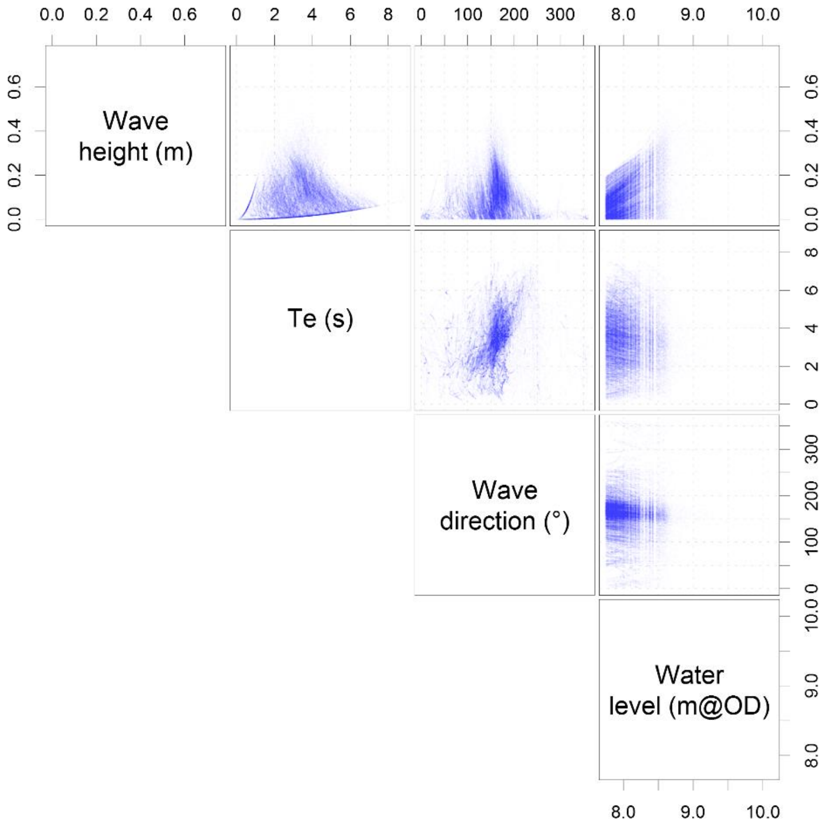
	Section 21
	Section 22

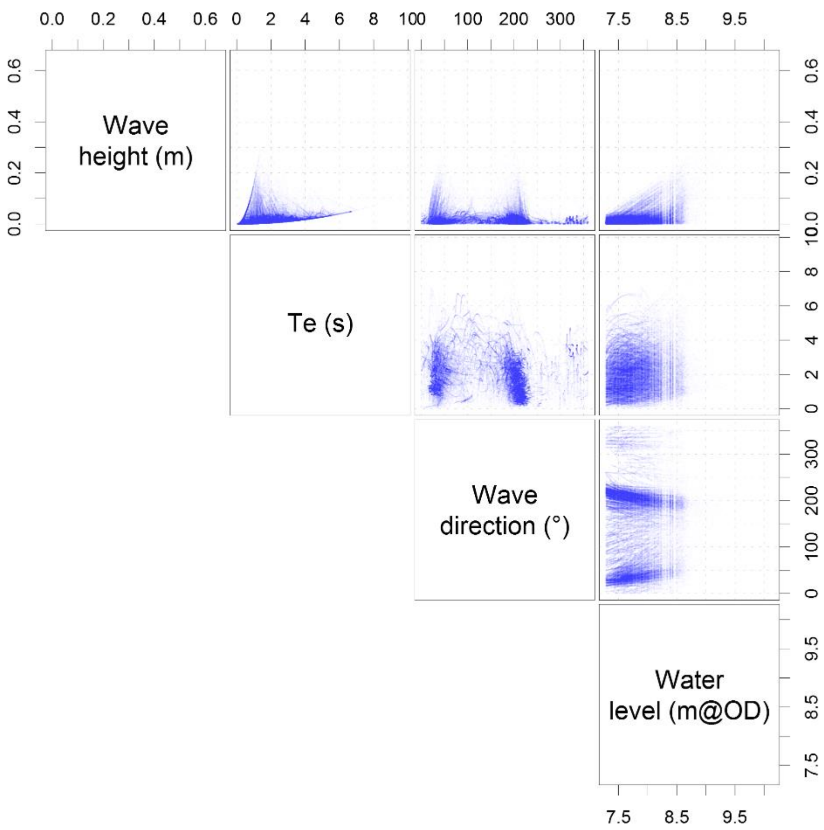
	Section 23
	Section 24

	Section 25
	Section 26

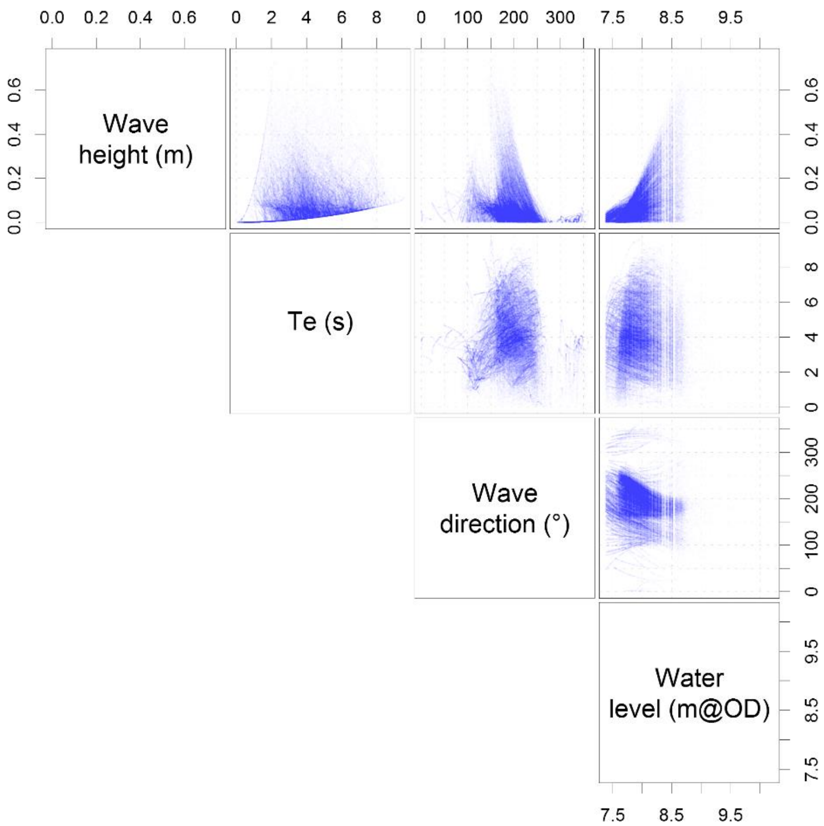
	Section 27
	Section 28



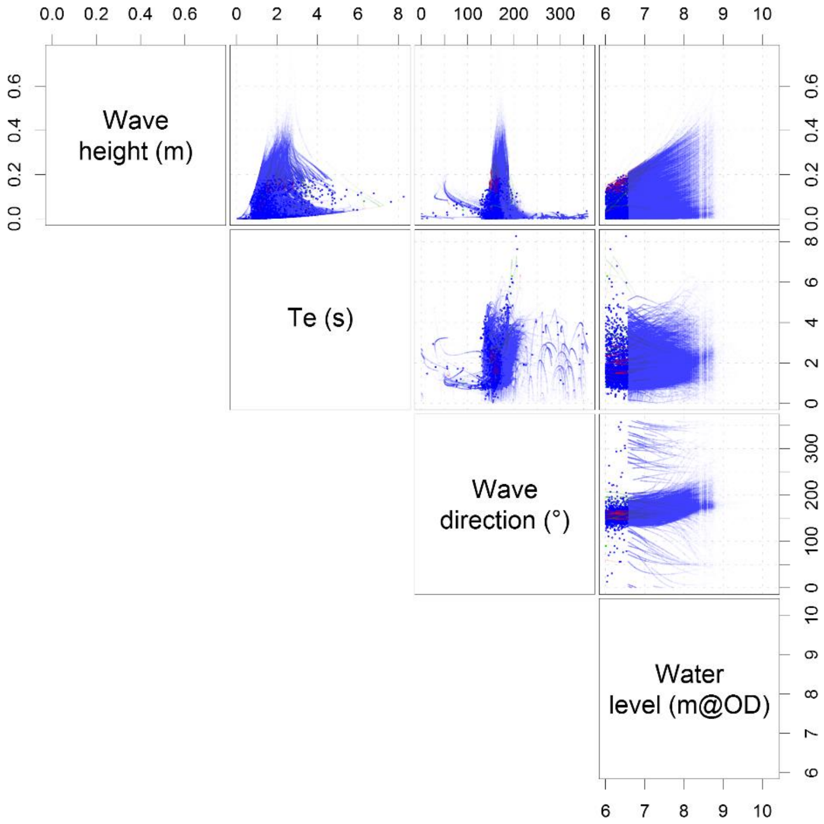




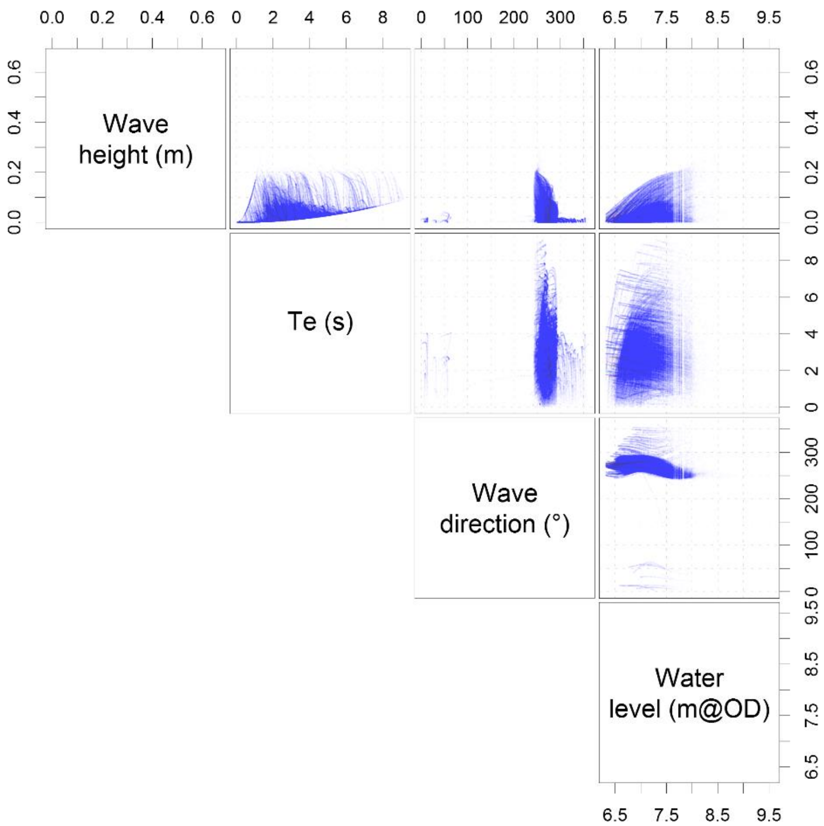
Section 29



Section 30



Section 31



Section 32

Offices at

Coleshill
Doncaster
Dublin
Edinburgh
Exeter
Glasgow
Haywards Heath
Isle of Man
Limerick
Leeds
Newcastle upon Tyne
Newport
Peterborough
Saltaire
Skipton
Tadcaster
Thirsk
Wallingford
Warrington

Registered Office
South Barn
Broughton Hall
SKIPTON
North Yorkshire
BD23 3AE

t:+44(0)1756 799919
e:info@jbaconsulting.com

Jeremy Benn Associates Ltd
Registered in England
3246693

Visit our website
www.jbaconsulting.com